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Shear strength properties of western Iowa loess

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SHEAR STRENGTH PROPERTIES OF WESTERN IOWA LOESS

by

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I. INTRODUCTION

Loess is avoided as a foundation soil because of its peculiar behavior. Terzaghi (1) described loess as one of the most treacherous foundation soils to be encountered in nature, because of its behavior after saturation.

Loess is not a rare soil but covers vast areas of the world. In the United States loessial deposits are found in Nebraska, Kansas, Wisconsin, Illinois, Tennessee, Mississippi, Idaho, Washington, and Iowa, where loess is one of the major surface soils. Loess is generally considered to be composed of silt size particles of desert or glacio-fluvial origin which have been eroded, transported, and deposited by wind action.

All foundations constructed by the civil engineer ultimately derive their support from the underlying soil or rock. A knowledge of the engineering properties of the foundation material is necessary to determine its behavior under a structural load. A thorough knowledge of the engineering properties of loess is essential to safely utilize this material as a foundation soil. The origin and deposition of loess, its mineralogical composition, and its properties are extensively described in the literature, but little information is found on the actual experiences with loess as a foundation soil.

Perhaps the most important consideration in foundation engineering is predicting the bearing capacity of a soil. Bearing capacity is defined as the largest intensity of pressure which may be applied to the soil by a structure or structural member without causing excessive settlement or danger of failure of the soil in shear. Detrimental settlement depends on the stress-deformation characteristics of the soil and is generally evaluated by a consolidation test. Shearing strength of a soil may be determined by the direct shear test or the triaxial shear test. Advantages of the latter are; independent control of the three principal stresses (two of which are equal for cylindrical samples), more uniform distribution of stresses throughout the sample, control of drainage conditions, measurement of pore pressure, and relatively accurate measurement of volume changes. The main disadvantages are the expensive apparatus and the trained technicians required for accurate testing.

The objective of this investigation was to determine how shearing strength of several loessial soils from southwestern Iowa is affected by saturation and by consolidation. The shearing strength of a soil is its ability to resist sliding along internal surfaces. Some of the numerous variables affecting shearing strength are, coefficient of internal friction, intergranular pressure, pore pressure, cohesion, pressure history of the soil, drainage conditions, consolidation pressures, void ratios, water content, and air content.

II. REVIEW OF LITERATURE

A. Description of Loessial Soils

Loess is described petrographically as a quartzose, somewhat feldspathic clastic sediment composed of a uniformly sorted mixture of fine sand, silt, and clay particles arranged in an open and cohesive fabric normally resulting in a natural dry density of seventy to ninety pounds per cubic foot. The shape of the individual granular components ranges from irregular to platy, fibrous, or prismatic. Quartz and feldspars are irregular in shape; micas are platy, hornblende, zircon, pyroxenes, and apatite are prismatic, and sillimanite is fibrous. The clay minerals, montmorillonite and illite, occur as very fine platy crystals. Generally, the montmorillonite clay partially or completely envelopes the granules as a thin shell. These clay shells probably aid in the bonding of the granules together and in bonding the grains to the clay in the matrix (2).

Loess structure is characteristically a loose arrangement of silt particles with numerous voids and vertical root-like channels (3).

The Bureau of Reclamation studied loessial soils from the Missouri River Basin in central Nebraska (4, 5, 6, 7). Gibbs and Holland (2) reported that the physical properties of most of these loessial soils were quite similar as to gradation, specific gravity, and plasticity. Most of the loessial soils were classified as silty-loess while the remainder were classified as sandy-loess or clayey-loess. The specific gravity varied between very narrow limits, averaging 2.65. A plasticity index of from 5 to 12 percent, combined with the liquid limit varying from 28 to 34 percent, was characteristic for the silty-loess. The mineral constituents were also quite similar, which suggests a similar source area. Although the proportions of quartz, montmorillonite, and minor minerals varied moderately, most of the soils behaved similarly. Some loessial soils were calcareous while others were leached free of carbonates. The Iowa Engineering Experiment Station at Iowa State University studied the physical properties of Iowa loess (8), and found it to be similar to the Nebraska loess. The loess was also found to be predominantly silty with the clay content increasing with distance from the believed source of sediments.

B. Consolidation of Loessial Soils

Any soil structure may be considered to be a skeleton of solid grains enclosing voids which may be filled with liquid, gas, or a combination of both. When the soil structure is stressed by an external load so that a gradual reduction in volume occurs, the phenomenon is referred to as consolidation. The reduction in volume is generally attributed to the escape of water and air from the pores, causing a reduction in void volume (9).

The theory of consolidation was first proposed by Terzaghi based on a stress-strain-time relationship for a saturated soil. Nine assumptions were made, including complete saturation, negligible compressibility of the soil grains and water, one-dimensional compression, one-dimensional flow of water, and the validity of Darcy's law (9). The one-dimensional consolidation test and settlement analysis were developed based on Terzaghi's theory (10).

Primary consolidation of a saturated soil is a gradual process which involves the simultaneous escape of water from the pores and the transfer of stress from the pore water to the soil structure. Where complete consolidation of a soil is delayed for reasons other than permeability, such as plastic lag, this phase of consolidation is termed secondary consolidation.

When a saturated soil is loaded, the increase in stress is initially carried largely by the pore water as hydrostatic excess pressure. As the water escapes, the stress carried by the pore water is transferred to the soil structure until the hydrostatic excess pressure is zero. The volume of the soil mass is reduced proportional to the volume of water which escapes.

For a partially saturated soil, Terzaghi's theory is not completely applicable since the compressible gas in the pores may allow appreciable reduction in the void volume without the escape of pore water. Pressures thus developed in the soil pores may be predominately from the compressed gas. Additional reduction of void volume may also occur as both air and water escape. For the lack of a reliable consolidation test for partially saturated soils, engineers continue to utilize the one-dimensional consolidation test (11).

Gibbs and Holland (2) in their investigation of loessial soils in the Nebraska-Kansas area found that the moisture content of the natural in-situ soil is normally less than twenty percent of the dry soil weight. About thirty-five percent moisture content was required for complete saturation. The Bureau of Reclamation made many one-dimensional consolidation tests of undisturbed loessial soils in the partially saturated and saturated condition. Clevenger (12) summarized the work of the Bureau with these conclusions:

1. Potential settlement of loess is governed largely by the in-situ density and highest moisture content attained by the soil.

2. At low moisture content (less than fifteen percent) natural loess will consolidate little until loads in excess of 100 psi are applied regardless of density. At higher moisture content the supporting capacity depends on the density.

3. At moisture contents from higher than twenty percent to complete saturation:
   a. Low density loess (below eighty pounds per cubic foot) settles excessively.
   b. Medium density loess (eighty to ninety pounds per cubic foot) varies in the amount of consolidation.
   c. High density loess (above ninety pounds per cubic foot) does not settle excessively due to high moisture content and the soil may be treated as ordinary silt.

4. A loessial soil consolidates about the same amount whether it is prewetted or is wetted after loading.

Olson (13) conducted one-dimensional tests on undisturbed loessial soils from southwestern Iowa and found the consolidation results similar to the Bureau's work with Nebraska-Kansas loess.

One-dimensional consolidation of saturated soils is considered acceptable for analyzing deep foundations. Shallow foundations, however, don't behave as assumed by Terzaghi because of lateral movement of the soil and horizontal drainage of water. Lateral shifting of soil is governed largely by the shear resistance of the soil and therefore not related to consolidation. The effect of horizontal drainage of water may be better characterized by a three-dimensional consolidation test such as may be performed in a triaxial test apparatus. A study of two- and three-dimensional consolidation of saturated clays has been made by several investigators (14, 15, 16, 17). The amount of consolidation or volume change was attributed to the volume of water which escaped or drained during the test. Carrillo (14) showed that three-dimensional consolidation was affected by the horizontal radial flow and vertical linear flow of water. Aboshi (17) developed a procedure for estimating total settlement by three-dimensional consolidation of saturated clays using the concept of equivalent stress ratio and equivalent drainage diameter. McKinlay (18) found that the total consolidation was not noticeably affected by radial drainage, but the rate of consolidation varied to a degree dependent on the lack of homogeneity of the clay.

Many dams and canals have experienced failure or near failure because loessial soil foundations settled excessively when saturated with water (19).

No information was found on three-dimensional consolidation of partially saturated soils.

C. Shearing Strength of Loessial Soils

We may visualize the soil structure as a compressible skeleton of solid particles enclosing voids which, in saturated soils, are filled with water, or, in partially saturated soils, with water plus air. Shear stresses can be carried only by the skeleton of solid particles, but the total normal stress on any plane is the sum of the stress carried by the solid particles and the pressure of the fluid in the void space. That portion of the total normal stress carried by the pore water is termed hydrostatic excess pressure.[1]

[1]Hereafter referred to as pore water pressure.

The effective normal stress is the difference between the total normal stress and pore pressure (20). According to the Coulomb concept, shear strength is largely determined by the frictional forces developed at the contacts between the soil particles, and maximum resistance to shear may be expressed as:

$$\tau_e = c' + (\sigma - u) \tan \phi' \qquad (1)$$

where

τ_e = shearing strength based on effective stress

c' = effective cohesion

σ = total pressure normal to the plane considered

u = pore pressure

$(\sigma - u)$ = effective normal stress to the plane considered

ϕ' = effective angle of internal friction

When the shearing strength of a soil is analyzed in terms of total stresses, the shear equation becomes:

$$\tau = c + \sigma \tan \phi \qquad (2)$$

where

τ = shearing strength based on total stress

c = apparent cohesion

σ = total pressure normal to the plane considered

ϕ = apparent angle of internal friction.

The values of the shear parameters - cohesion and angle of internal friction - are not constant soil properties but are coefficients which may vary over wide ranges for a given soil under various possible conditions of precompression, drainage, and other variables (9).

In most engineering problems relating to foundation stability, both settlement and shear analysis must be made to determine which is the controlling factor. In the shear stability analysis the engineer must decide whether to use the effective stress or the total stress analysis. The stability problem may be divided into two classes in which (1) the pore pressure is independent of the magnitude of the total stresses acting in the soil; and (2) the pore pressure depends on the magnitude of the stresses acting in the soil and the time which has elapsed since their application (20).

In the first analysis, the shear analysis would be made using effective cohesion and effective angle of internal friction. The magnitude of the total stresses on a potential slip plane may be estimated with reasonable accuracy from static analysis. Taylor (9) describes several of these methods, including the Swedish method of slices, Culmann method, sliding block method for earth dams, Prandtl solution, Fellenius solution, and Terzaghi solution. Since the pore pressure is independent of the total stress, it must be determined by flow net studies for steady seepage and depth below the water table surface for stationary ground water, or actually measured in the field.

In the second method of analysis, where the pore pressure depends on the magnitude of the total stress, the shear stability may be made using apparent cohesion and apparent angle of internal friction. Total stress analysis is applicable only when the stress likely to cause failure is imposed under conditions which allow only negligible dissipation of the excess pore pressure, this condition exists only through a combination of rapid loading and low permeability of saturated soils, as in the case of initial stability of a foundation on saturated clays.

If consolidation occurs such that the pore pressure is dissipated, the stability analysis must be made in terms of effective stress.

Clevenger (12) concluded from his summary of the Bureau's study of undisturbed drained and undrained triaxial shear tests of Nebraska-Kansas loess that:

1. Increasing the moisture content of the natural soil reduced the shearing strength. The coefficient of internal friction was unaffected by the moisture content but cohesion decreased rapidly with increases in water content.

2. Cohesion decreased with decreasing density while the coefficient of internal friction was unaffected.

3. Differences in sand and clay content appeared to exert comparatively minor influence. The coefficient of internal friction remained rather constant while cohesion increased slightly with increased clay content.

4. Prewetting the soil to near saturation reduced the shearing strength since the cohesion generally became zero. For low density, highly wetted loess the shearing strength was zero until the soil was consolidated, bringing the soil grains closer together to develop internal friction. Approximately ten pounds per square inch of effective normal stress was required to develop internal friction.

5. The coefficient of internal friction was not affected by moisture content, density, prewetting the soil, or preconsolidation of the soil. The value of tan ϕ varied from 0.60 to 0.65 for all soils and conditions of testing.

6 Cohesion was affected by moisture content, density, prewetting the soil, and preconsolidation of the soil. Cohesion of ten to twenty pounds per square inch was measured for high density loess with moisture contents less than fifteen percent, however, the introduction of additional moisture quickly reduced the cohesion.

Olson (13) investigated the shearing strength of southwestern Iowa loess using the direct shear test. The general behavior of the loess tested was similar to that reported by the Bureau of Reclamation.

III. MATERIALS

The Wisconsin loess of southwestern Iowa is believed to have been deposited by wind action from the Missouri River flood plain glacial outwash source (8).

Three Wisconsin age loessial soil samples were used in this investigation. The locations are tabulated in Table 1. The first site (49 B) was selected in the first bluff line adjacent to the Missouri River flood plain, it is near the supposed source, and the soil texture is coarse silt with only a small clay content. The second site (99 C-1) was selected near site 49 B in the recent alluvial (or secondary loess) deposits formed by a small stream draining the area. The sediments in the alluvial deposits would thus be composed of the particles eroded, transported, and redeposited by water action. The reworked loess should be similar in composition to the parent material. The third site (97 B-1) was selected some distance from the Missouri River flood plain. The loess at this site has finer texture and higher clay content.

Site 49 B is located east of U. S. Highway Alternate 30 adjacent to the Missouri River flood plain in Harrison County. The soil samples were taken from depths of 10 to 11 feet (49 B-1) and 76 to 77 feet (49 B-2) from the calcareous C horizon. The average dry densities and moisture contents for 49 B-1 and 49 B-2 were found to be 73.6 pounds per cubic foot and 7.3 percent by dry soil weight, and 88.2 pounds per cubic foot and 9.3 percent by dry soil weight, respectively The five-micron clay content was nineteen percent for 49 B-1 and eleven percent for 49 B-2. The surface soil is mapped in the Hamburg series (21).

Sample 49 B-1 was not tested in this study because of the difficulty of trimming good specimens. The soil is very friable, loose, and thickly matted with fine roots from the surface plants. So many burrowing animal holes (krotovinas) were present that a sample without large voids was extremely difficult to obtain. The few samples that were obtained and brought back to the laboratory were easily broken while trimming to the required diameter.

Site 99 C-1 is also located in Harrison County several miles from site 49 B. The samples were taken from the stream banks at a depth of seven to eight feet. The average dry density was 84.8 pounds per cubic foot with an average moisture content of 14.3 percent by dry soil weight. The secondary loess in the alluvial deposits is mapped in the McPaul series (21). The five-micron clay content was twenty-two percent, which is much higher than 49 B-2; sample 99 C-1 is more similar to 97 B-1 in clay content.

Site 97 B-1 is located in Cass County. The soil samples were taken from a depth of seven to eight feet out of a roadside cut. The soil is finer in texture and higher in clay content than 49 B-2. The average dry density was 86.1 pounds per cubic foot with an average moisture content of 23.8 percent by dry soil weight. The surface soil is mapped in the Marshall series (21).

Physical property data on the loessial soils used in this investigation are tabulated in Table 2.

Table 1. Location of sampling sites

Sample Nr.	County	Quarter section	Section	Tier	Range	Soil series	Sampling depth	Horizon[a]
49 B-1	Harrison	NE1/4	3	77N	44W	Hamburg	10 to 11 ft.	C
49 B-2	Harrison	NE 1/4	3	77N	44W	Hamburg	76 to 77 ft.	C
99 C-1	Harrison	NE 1/4	34	78N	44W	McPaul	7 to 8 ft.	C
97 B-1	Cass	SW 1/4	13	77N	35W	Marshall	7 to 8 ft.	C

[a]All samples were calcareous (unleached) except sample 99 C-1.

Table 2. Physical properties of the soil

Sample Number	49 B-2	99 C-1	97 B-1
Textural composition,[a] % dry soil weight			
Sand	1.4	0.6	0.7
Silt	87.1	77.2	73.3
Clay 5-micron	11.5	22.2	26.0
Clay 2-micron	9.5	18.0	20.2
Colloidal 1-micron	8	16	17
Textural classification[b]	silty loam	silty clay loam	silty clay loam
Predominate clay mineral, by x-ray diffraction	---Montmorillonite---		
Average dry density,[c] pcf	88.2 $\pm$ 2.1	84.8 $\pm$ 3.6	86.1 $\pm$ 3.9
Average natural moisture content,[c] % dry soil weight	9.3 $\pm$ 2.5	14.3 $\pm$ 4.2	23.8 $\pm$ 1.7
Degree of natural saturation,[c] %	28.0 $\pm$ 7.5	38.6 $\pm$ 9.8	71.1 $\pm$ 5.2
Initial void ratio[c]	0.899 $\pm$ 0.040	0.962 $\pm$ 0.082	0.905 $\pm$ 0.075
Liquid limit, % dry soil weight	28.7	31.0	34.9
Plasticity index, % dry soil weight	2.1	5.1	11.1

[a]Sand - 2.0 to 0.074 mm., silt 0.074 to 0.005 mm., clay less than 0.005 mm.

[b]Textural classification based on the Bureau of Public Roads system.

[c]The $\pm$ entry is the standard deviation from the arithmetic mean of the 12 specimens tested.

IV. TESTING PROCEDURE

A. Obtaining Test Specimens

Special care was taken in obtaining and preparing the undisturbed soil specimens required for this investigation.

A steep hillside, vertical stream bank, and a vertical highway cut were the respective sites of samples 49 B-2, 99 C-1, and 97 B-1. The previously noted sampling depth was measured vertically from the ground surface immediately above or close to the sampling site. A small cave was then dug horizontally into the face of the site just above the proposed depth of sampling, and an attempt was made to strip all of the exposed soil which may have been altered by water, freezing and thawing, or contaminated by roots or materials eroded from above. Using the cave floor as the top of the sample, a pedestal of soil about four inches in diameter by eight inches in height was carved with a large knife. A four and one-half inch diameter by eight and one-half inch high ice cream container, heavily coated with paraffin, was gently slipped over the pedestal. The pedestal was cut off at the bottom, the container plus soil turned over, and the excess soil trimmed level with the top. The container cover was placed and sealed tightly with masking tape for prevention of moisture loss, and each container was marked with laboratory identification numbers. All samples were returned to the laboratory and stored in a humidity room at a constant temperature of about 77 degrees Fahrenheit and near 100 percent relative humidity.

The cylindrical specimen size selected for testing was 2.8 inches in diameter by 5.6 inches in length. The following procedure was used to prepare the required specimens:

1. The samples obtained from the field were trimmed to a cylinder of exactly 2.8 inches in diameter and approximately six inches in length using the trimming tool illustrated in Figure 1. The trimming tool is a vertical-axis soil lathe using a rapidly spinning wire brush as a cutter.
2. The trimmed specimen was removed from the soil lathe and wrapped with two layers of plastic (Saran) wrap, each layer being sealed tightly with pressure sensitive tape. Specimens were then wrapped with an outer cover of aluminum foil.
3. The trimmed specimens were marked with laboratory identification numbers and again placed in the constant temperature and humidity room until time of testing.

B. Consolidated-Undrained Triaxial Shear Test

1. Description of the test

The triaxial shear test is conducted on a cylindrical specimen which is subjected to an axial load while the specimen is confined by a constant lateral pressure. The soil specimen is encased in a rubber membrane which acts as a confining surface when air or liquid is introduced into the cell to a specified pressure. Ends of the specimen bear on porous plates. Failure of the specimen is caused by the application of an axial load.

Figure 1. Trimming tool

The trimming tool operates on the same principle as a lathe. The cutting tool is a copper wire brush spinning at 1725 rpm. The soil platform rotates at 36 rpm.

The consolidated-undrained test with pore pressure measurement is one type of triaxial shear test; it is described by Bishop and Henkel (20). In this test the soil specimen is allowed to consolidate under a cell pressure of known magnitude, the three principal stresses thus being equal. Drainage is allowed in the consolidation phase with volume change-time relationship measured during the test by collecting the volume of drained water and/or air. Primary consolidation of the soil is completed when the total volume change for the specific cell pressure applied becomes constant. Secondary consolidation would not affect volume measurement since it is not related to permeability. The soil specimen is then sheared under undrained conditions allowing pore pressure to develop while maintaining the same cell pressure used during the consolidation phase. The axial load is applied at a uniform rate of strain until the specimen fails. The pore pressure is measured to allow the shearing strength to be analyzed by the effective stress principle. The consolidated-undrained shear test was used in this investigation as it best simulates the field condition of prewetting and preconsolidation of a loessial soil by ponding water or injection of water into the strata to induce rapid settlement before placement of a structure.

Total stress analysis of partially saturated soil is not recommended by Bishop and Henkel (20) because the apparent cohesion is dependent on the pore pressure developed by compressed gas rather than the pore water. The effects of saturation and preconsolidation on shearing strength would be analyzed best by effective stress.

2. Apparatus

The triaxial shear test machine used was a modified Soiltest Model T-118, illustrated in Figure 2. The axial load was applied by a variable speed electric drive through a calibrated proving ring to the top of the soil specimen, the magnitude of the axial load being indicated by deflection of the proving ring. The vertical deflections of the specimen were measured with a dial gauge extensometer. The cell fluid used was compressed air, with the cell pressure controlled by a diaphragm regulator to within 0.33 psi. Pore pressure developed in the soil specimen was measured automatically with a Karol Warner Model 51-PP pore pressure device, Figure 3. By adjusting the mercury-water interface in the hand-pump to some pre-selected reference level, the pore pressure is measured at constant volume throughout the test since volume change in the pore pressure line is reduced to zero. The volume change of the soil specimen during the consolidation phase of the test was measured using calibrated gas and liquid collection burettes. All volumes were collected at atmospheric pressure and room temperature, which unfortunately could not be controlled.

3. Test

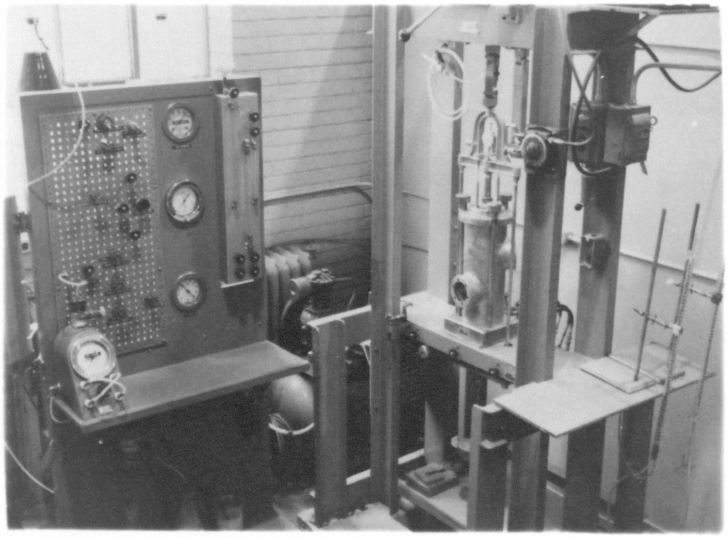
The consolidated-undrained test was performed in the following manner:

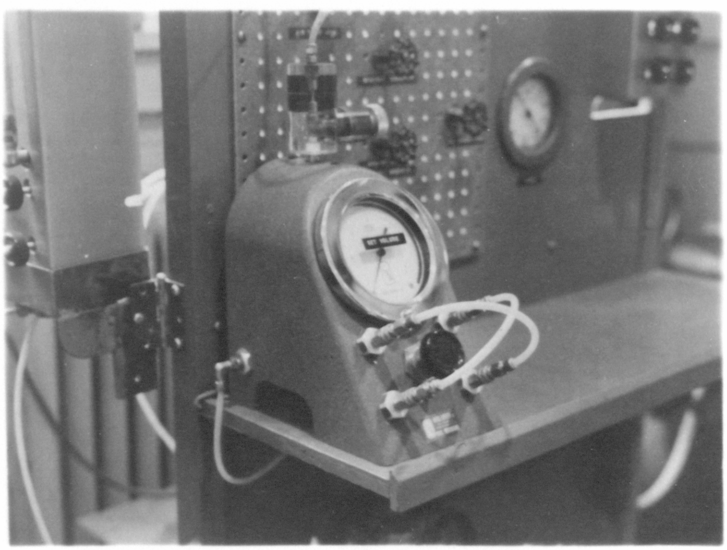
1. The 2.8 inch diameter specimen was obtained from the humidity room and trimmed to near 5.6 inches height. The height, diameter, and weight were measured. Moisture contents were determined from the soil trimmings.

Figure 2. Modified Soiltest Model T-118 triaxial shear testing machine (right) and cell control panel (left)

Figure 3. Karol-Warner Model 51-PP pore pressure device

This instrument allows the pore pressure to be measured at constant volume by maintaining the volume change in the pore pressure line to zero.





2. The specimen was placed in the triaxial cell sealed in the rubber membrane.
3. Water was introduced to the bottom of the specimen to prewet the soil to saturation. A slight suction was applied to the top of the specimen to assist in drawing the water through the soil and displacing the air from the voids. The soil was saturated for about 18 to 20 hours.[1]
4. Cell pressure was applied to consolidate the specimen. Drainage was allowed and the volume of air and water expelled from the soil was measured at specified time intervals. The specimens were consolidated for an arbitrary period of 24 hours to insure that complete primary consolidation was achieved.
5. Drainage was not allowed during the shear phase of the test. The pore pressure device was calibrated to record the average pressure developed on the top and bottom porous plates.
6. The axial load was applied at 0.007 inch per minute of vertical deflection of the soil specimen.[2]
7. The axial load from proving ring deflection and the pore pressure developed were read every 0.025 inch of vertical sample deformation until failure occurred.

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[1]Step 3 was omitted for soil specimens tested in the natural condition.

[2]The rate of strain was selected to allow sufficient time for the pore water to migrate to the end plates and the pore pressure to fully develop in the saturated porous end plates as recommended by Akroyd (11).

Twelve specimens were tested for each of the three soils studied in this investigation. Six specimens of each soil were tested at their natural moisture content and the remaining six specimens were saturated prior to testing. In each case, cell pressures of 10 to 60 pounds per square inch were used.

V. RESULTS AND DISCUSSION ON CONSOLIDATION

Plots of the soil void ratio versus the log of consolidating pressures are shown in Figures 4, 5, and 6.[1]

The data necessary to construct an e-log p curve for a soil are normally obtained from a single incrementally loaded specimen, allowing each load to remain until compression has practically ceased (or the volume change becomes constant).

In this study the e-log p data were obtained using one specimen for each of the consolidating pressures. Difficulty was encountered by the non-uniformity of the soil specimens. The data were normalized by calculating the percent change in void ratio for each consolidation pressure and relating the consolidated void ratio to the average initial void ratio ($\bar{e}_o$) of the specimens tested.[2]

An additional problem was encountered in that the reduction in volume of the soil mass is not proportional to the total volume of water plus air at atmospheric pressure expelled from the pore space of a partially saturated soil; i.e., the pressure of the air may be greater than the pressure of the pore water, in part due to the compression of the pore air by the surface tension of the surrounding water film (22). In addition to entrapped gas bubbles, certain amounts of air and other gases always exist in solution in the pore water; a definite amount of

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[1]Hereafter referred to as the e-log p curve(s).

[2] $\bar{e}_o$: arithmetic average of the initial void ratios of the twelve specimens tested.

dissolved air is possible for a given temperature and pressure (9). When the compressed air is expelled, it expands to atmospheric pressure obeying Boyle's and Henry's gas laws. However, not all of the air in the pores of a partially saturated soil exists at a pressure greater than atmospheric. Alpan (23) described consolidation of a partially saturated soil to occur in two steps: (1) upon application of the load there is a rapid isotropic compression of the gas in the pores due to the compressibility of the grain structure; (2) this is followed by drainage of the pore fluid from the soil until the gradient causing the flow ceases to operate.

Three dimensional consolidation of the partially saturated soils was analyzed by assuming each of three sets of conditions: First, the reduction in volume of the soil mass was considered proportional only to the volume of water expelled, second, the volume reduction was considered proportional to the total volume of water plus air expelled at atmospheric pressure; and third, the volume change was considered proportional to the volume of water plus the volume of air as if corrected by Boyle's and Henry's gas laws to pore air pressure existing before the consolidation load was applied.

The solid curves A and B in Figures 4, 5, and 6 were computed for the consolidation of samples at natural moisture content. Curve A was calculated assuming that the volume of the soil mass is reduced only by the volume of water expelled from the pore space. The pore air is considered to have no effect on the volume change of the soil mass (first case). Curve B was calculated assuming that the volume of the soil mass is reduced by the sum of the volume of water plus the volume of air at atmospheric pressure expelled from the pore space. The pore air is considered to exist at atmospheric pressure (second case). When the consolidation load is applied, the pore air is rapidly compressed only if the soil structure is compressed. The compressed air then quickly expands through the soil to atmospheric pressure because of its high permeability. Thus, the pore air is considered to reduce the volume of the soil mass by the identical volume of pore air expelled at atmospheric pressure.

The shaded area between the two solid lines, A and B, represents the limits of the volume reduction of a partially saturated soil attributed to the volume of water plus the corrected volume of pore air (third case). If the pore air exists at some pressure greater than atmospheric and has some effect on the volume reduction of the soil mass, the actual e-log p curve for a partially saturated soil would plot in this shaded area. The volume of air collected at atmospheric pressure must be corrected by some parameters to reflect the volume reduction of the soil mass attributed to pore air.

The dashed lines C and D were calculated the same as A and B, respectively, for a soil which was wetted before the consolidation load was applied, and the cross-hatched area between the two dashed lines represents the limits of the third case. Complete saturation of the soil probably was not obtained, Bishop and Henkel (20) reported that full saturation of partially saturated clayey soils was not obtained even after using de-gassed water with a small hydraulic head for a month or more, whereas the saturation period in the present test was only about 18 to 20 hours.

Comparison of the results shown in Figure 4 with one-dimensional consolidation data reported by Gibbs and Holland (2) for a similar loessial soil from the Nebraska-Kansas area suggests that the consolidation of the partially saturated soils should be analyzed according to assumptions of the third case; i.e., their consolidation data would plot in the shaded or cross-hatched areas depending on the moisture condition of the soil. Thus, good approximation of the consolidation of a partially saturated soil could be obtained by assuming case one and decreasing the calculated void ratio by five to ten percent to account for the effects of pore air.

The data obtained in this study appear to substantiate the consolidation process described by Alpan (23) for a partially saturated soil, and further indicate that some of the pore air exists at a pressure higher than atmospheric pressure. One further conclusion may be made: the pore air does not appreciably affect the consolidation of a partially saturated soil. That is, since the actual consolidation (Gibbs and Holland's data for a similar loessial soil) is closely approximated by curves A and C in the three e-log p curves, the air expelled at atmospheric pressure must have occupied a small volume at high pressure. The pore air pressure attributed to surface tension effects of water have been reported to be as high as several hundred atmospheres for highly colloidal soils (24).

The consolidation characteristics of samples 49 B-2 and 97 B-1 were identical to the behavior described by Clevenger (12) for similar loessial soils. Sample 49 B-2 being of medium density and low moisture content was highly resistant to consolidation in its natural state but consolidated rapidly when saturated. Sample 97 B-1, having a higher clay content and moisture content than 49 B-2, was less resistant to consolidation in its natural condition and when saturated also consolidated rapidly. Both samples 49 B-2 and 97 B-1 released a large volume of pore air at atmospheric pressure. Sample 99 C-1, a secondary loessial soil, behaved similarly to 49 B-2, but the volume of pore air expelled at atmospheric pressure was much less. The reworked loess structure is different from the primary loess structure in that the clay hulls around the grains are absent for the most part which may account for the reduced surface tension effect on pore air.

For time-settlement relationships from three dimensional consolidation of partially saturated soils, some additional information is required. The vertical deflection of the specimen must be measured to relate volume change to vertical settlement. The effects of radial drainage of water and air and the change in permeability with changes of porosity and saturation and its effect on the coefficient of consolidation and the dissipation function must also be considered.

The consolidation data for the three loessial soils are tabulated in Tables 3, 4, and 5 in the Appendix.

Figure 4. Void ratio - log pressure diagram for sample 49 B-2, loess near the source area

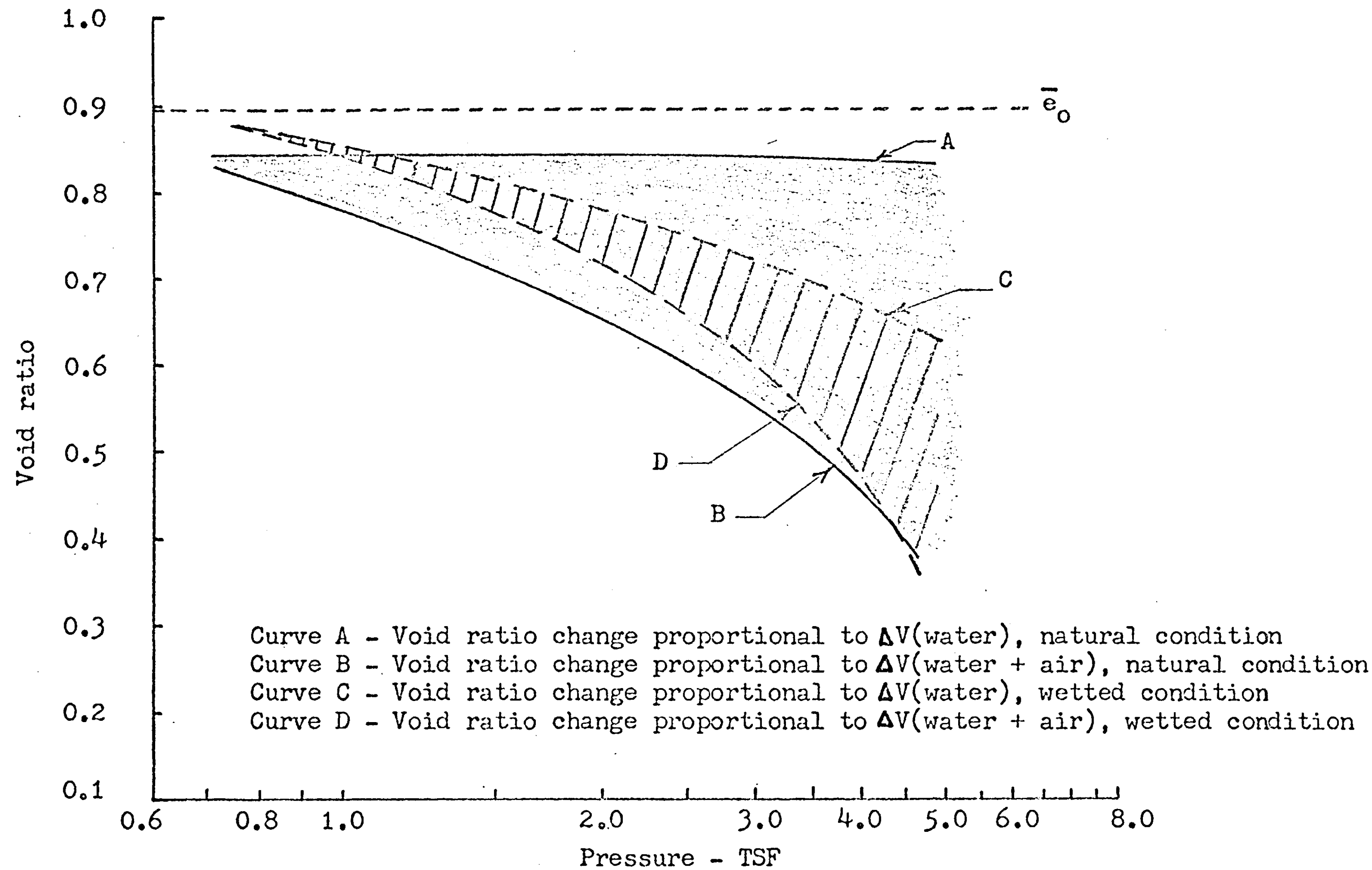

1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
Void ratio
0.6
0.8
1.0
2.0
3.0
4.0
5.0
6.0
8.0
Pressure - TSF
$\bar{e}_o$
A
B
C
D
Curve A - Void ratio change proportional to ΔV(water), natural condition
Curve B - Void ratio change proportional to ΔV(water + air), natural condition
Curve C - Void ratio change proportional to ΔV(water), wetted condition
Curve D - Void ratio change proportional to ΔV(water + air), wetted condition

Figure 5. Void ratio - log pressure diagram for sample 99 C-1, secondary loess near the source area

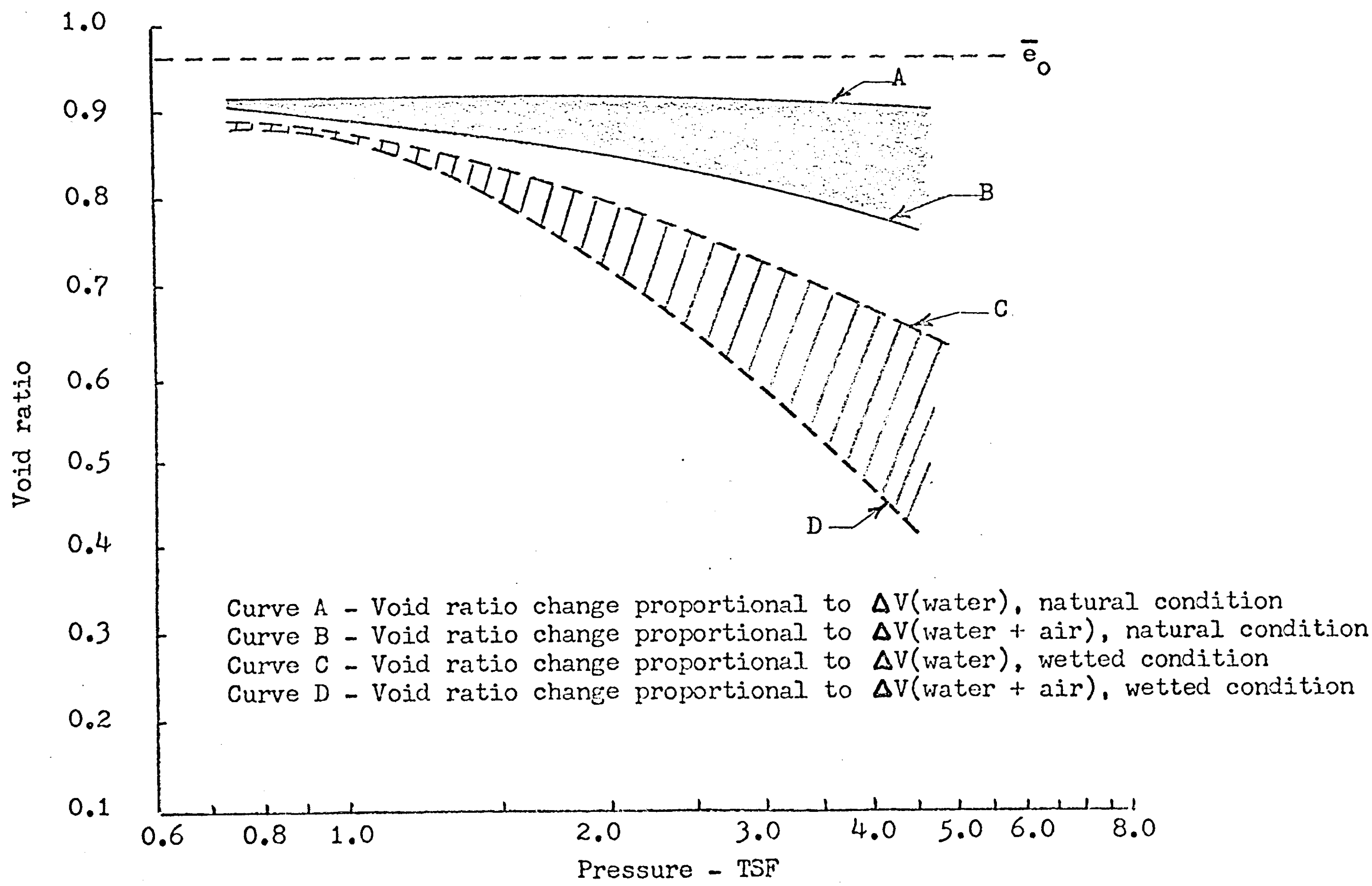
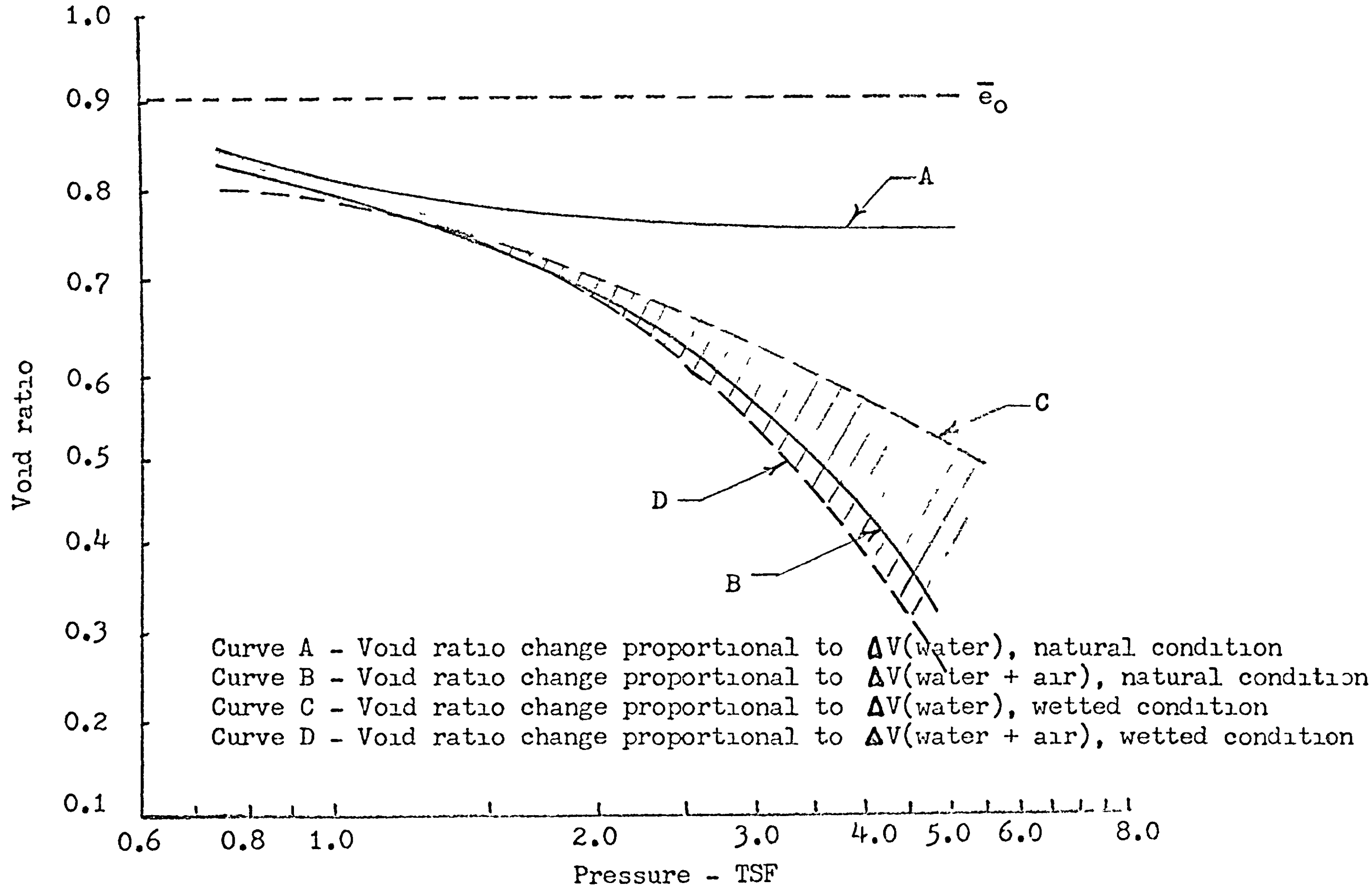

Void ratio
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
$\bar{e}_o$
A
B
C
D
0.6
0.8
1.0
2.0
3.0
4.0
5.0
6.0
8.0
Pressure - TSF
Curve A - Void ratio change proportional to ΔV(water), natural condition
Curve B - Void ratio change proportional to ΔV(water + air), natural condition
Curve C - Void ratio change proportional to ΔV(water), wetted condition
Curve D - Void ratio change proportional to ΔV(water + air), wetted condition

Figure 6. Void ratio - log pressure diagram for sample 97 B-1, loess some distance from source area



1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
Void ratio
$\bar{e}_o$
A
B
C
D
Curve A - Void ratio change proportional to ΔV(water), natural condition
Curve B - Void ratio change proportional to ΔV(water + air), natural condition
Curve C - Void ratio change proportional to ΔV(water), wetted condition
Curve D - Void ratio change proportional to ΔV(water + air), wetted condition
0.6
0.8
1.0
2.0
3.0
4.0
5.0
6.0
8.0
Pressure - TSF

VI. RESULTS AND DISCUSSION ON SHEARING STRENGTH

The relation of consolidation behavior to shearing strength is probably the most important structural property of loessial soils to be considered by engineers. Several methods have been used to prepare the soil as a foundation (19). These include prewetting and applying a surcharge to induce rapid consolidation prior to construction, or injection of a bentonite slurry, under pressure, to increase the consolidation resistance and shearing strength.

An evaluation of the effect on shearing strength induced by prewetting (to near saturation) and consolidating loessial soils was undertaken in this study.

A. Stress-Strain Relationship

Graphs showing the axial stress-axial strain relationship for the three southwestern Iowa loessial soils subjected to three-dimensional consolidation before the shear test are shown in Figures 7, 8, 9, 10, 11, and 12. The axial stress was corrected for pore water pressure and evaluated in terms of effective stress.

The effective axial stress-axial strain curves[1] of the soils tested were of three types: (1) axial stress increases steadily with axial strain, (2) axial stress increases to a maximum value and then remains constant with increasing strain, and (3) axial stress increases to a maximum peak and then decreases with increasing strain. The stress-strain behavior is probably related to the rearrangement of the grains

[1]Hereafter referred to as stress-strain curve(s).

Figure 7. Effective axial stress-axial strain curves for sample 49 B-2, natural condition

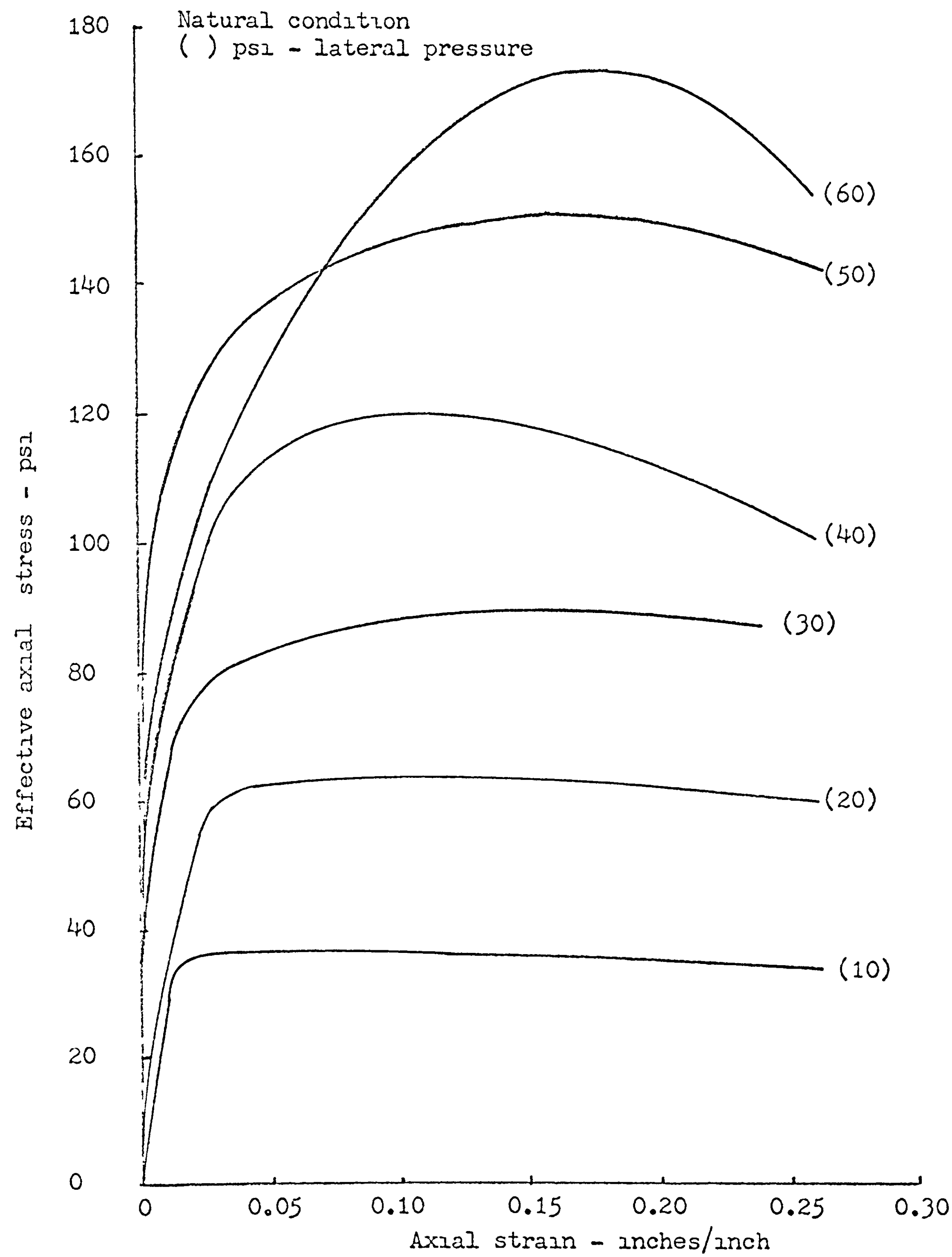

Natural condition
( ) psi - lateral pressure
Effective axial stress - psi
Axial strain - inches/inch
0
20
40
60
80
100
120
140
160
180
0
0.05
0.10
0.15
0.20
0.25
0.30
(60)
(50)
(40)
(30)
(20)
(10)

Figure 8. Effective axial stress-axial strain curves for sample 49 B-2, wetted condition

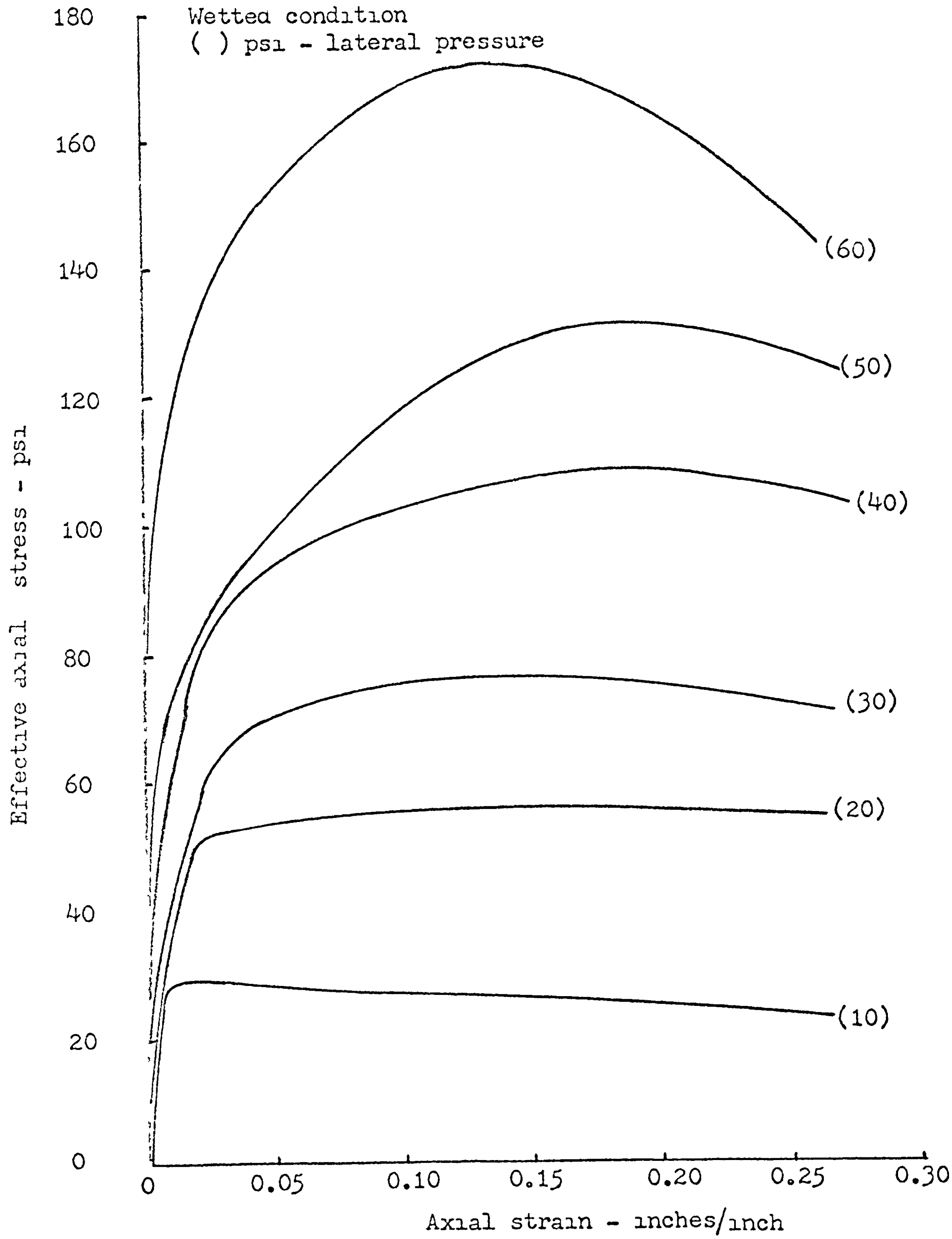

Wettea condition
( ) psı - lateral pressure
180
160
140
120
100
80
60
40
20
0
Effective axıal stress - psı
(60)
(50)
(40)
(30)
(20)
(10)
0
0.05
0.10
0.15
0.20
0.25
0.30
Axıal straın - ınches/ınch

Figure 9. Effective axial stress-axial strain curves for sample 99 C-1, natural condition

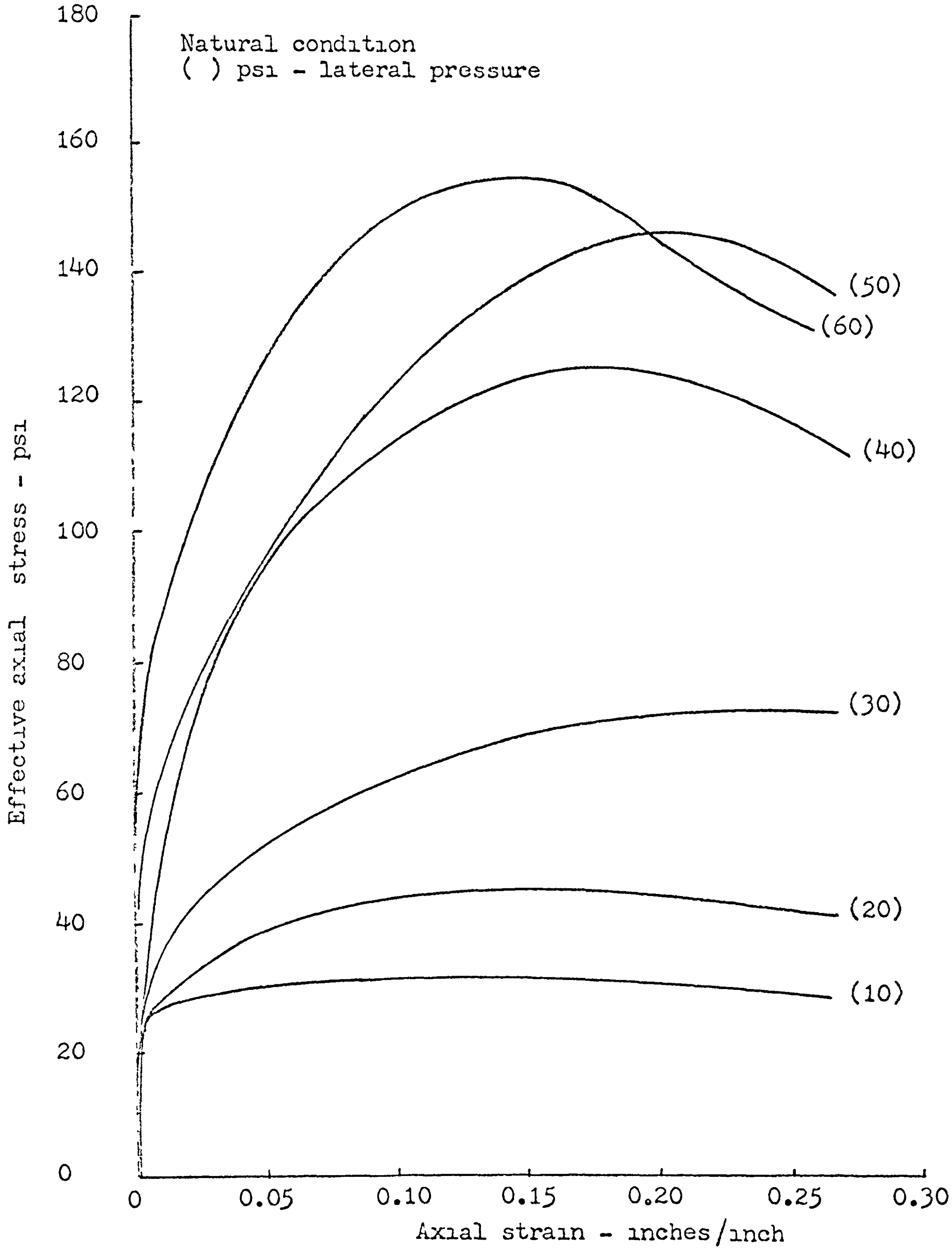

Natural condition
( ) psi - lateral pressure
Effective axial stress - psi
Axial strain - inches/inch
0
20
40
60
80
100
120
140
160
180
0
0.05
0.10
0.15
0.20
0.25
0.30
(50)
(60)
(40)
(30)
(20)
(10)

Figure 10. Effective axial stress-axial strain curves for sample 99 C-1, wetted condition

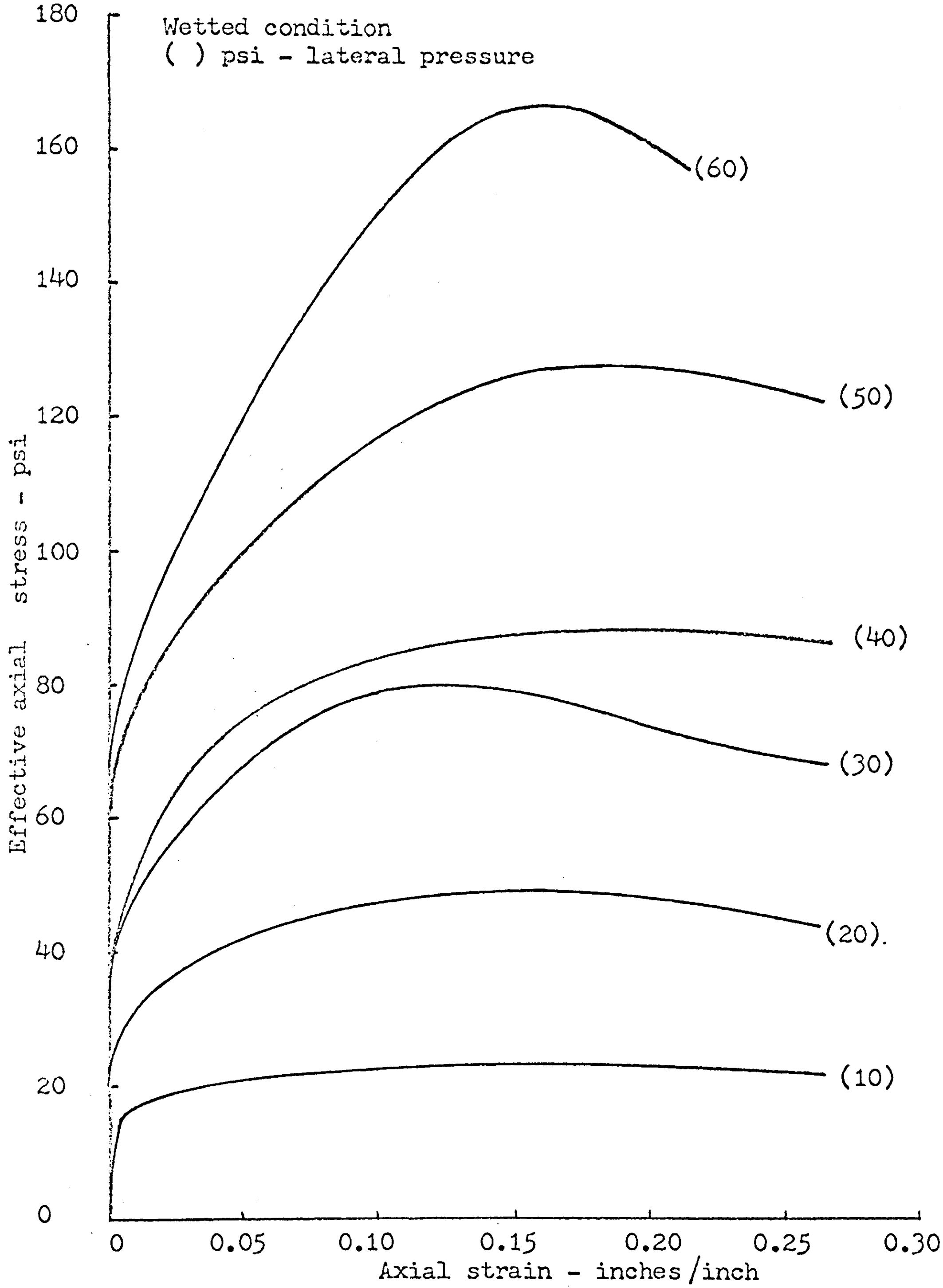
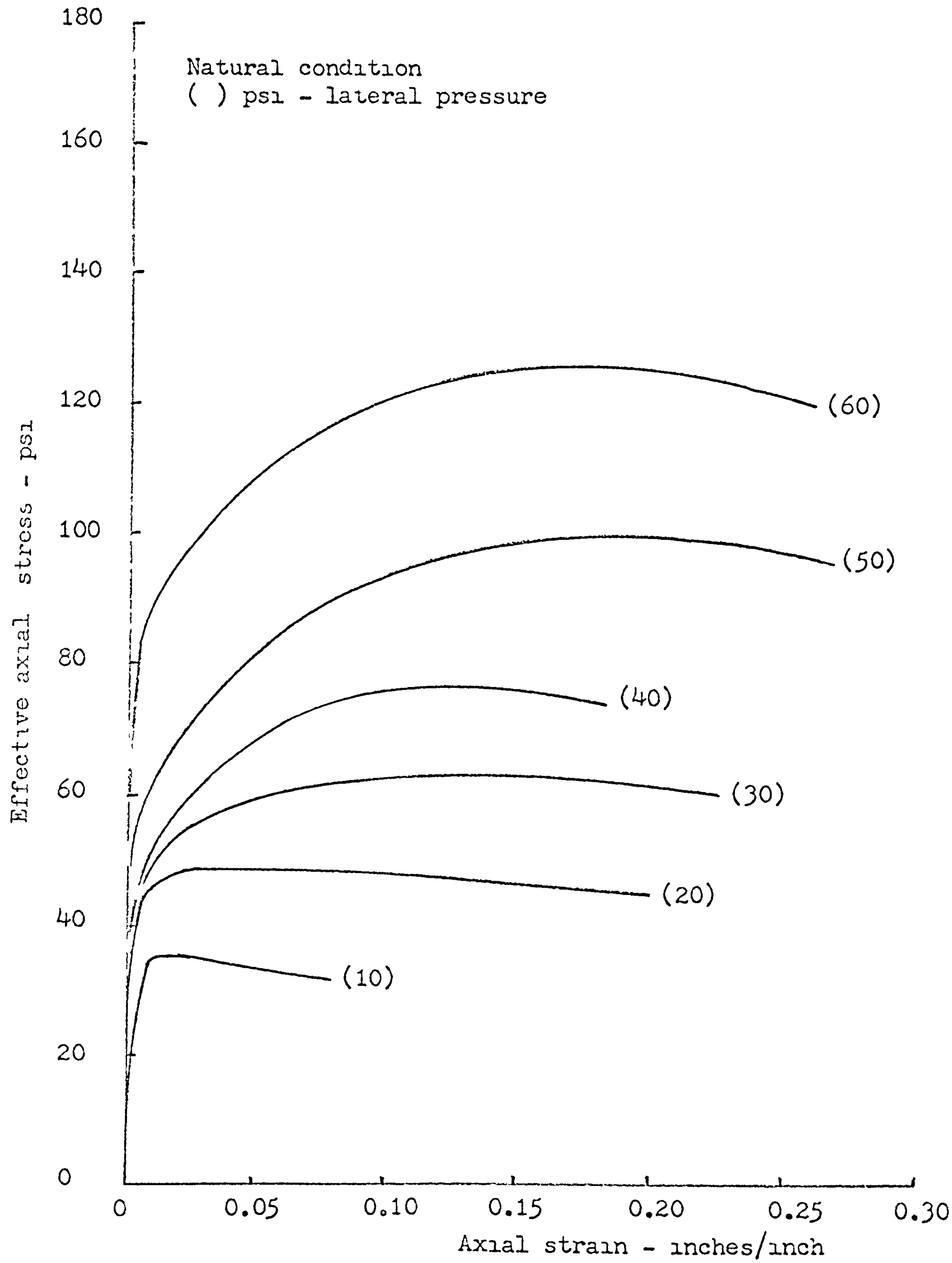
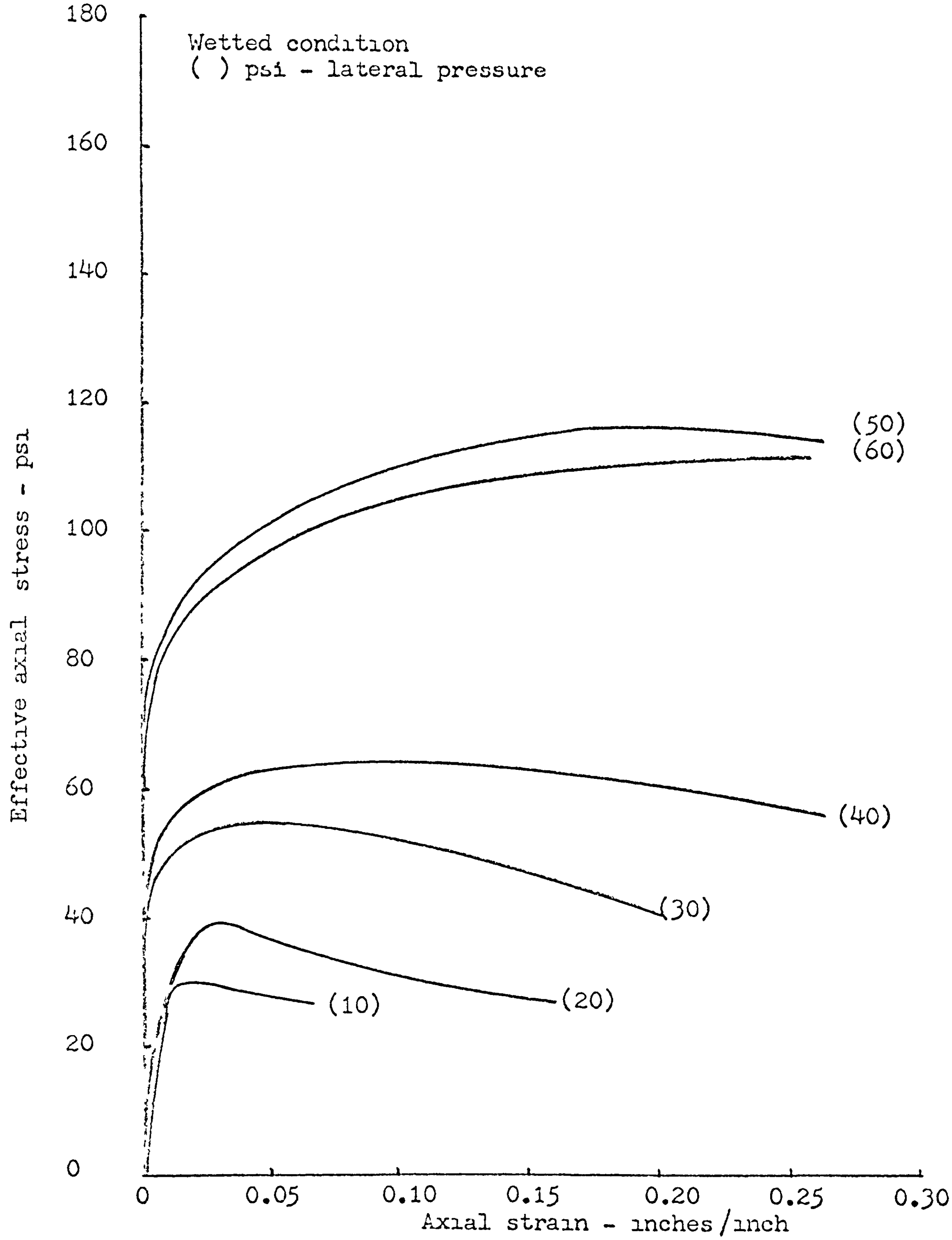

Wetted condition
( ) psi - lateral pressure
180
160
140
120
100
80
60
40
20
0
Effective axial stress - psi
(60)
(50)
(40)
(30)
(20)
(10)
0
0.05
0.10
0.15
0.20
0.25
0.30
Axial strain - inches/inch

Figure 11. Effective axial stress-axial strain curves for sample 97 B-1, natural condition



Natural condition
( ) psi - lateral pressure
Effective axial stress - psi
180
160
140
120
100
80
60
40
20
0
(60)
(50)
(40)
(30)
(20)
(10)
0
0.05
0.10
0.15
0.20
0.25
0.30
Axial strain - inches/inch

Figure 12. Effective axial stress-axial strain curves for sample 97 B-1, wetted condition


Wetted condition
( ) psi - lateral pressure
Effective axial stress - psi
Axial strain - inches/inch
0
20
40
60
80
100
120
140
160
180
0
0.05
0.10
0.15
0.20
0.25
0.30
(50)
(60)
(40)
(30)
(20)
(10)

in the soil structure induced by the stress change. The axial strain may be the result either of consolidation or of shifting of material laterally, the latter is largely governed by shearing resistance. Under a foundation the pressures at any depth may be calculated using the Boussinesq equation (9).

Generally, soils which were confined with low lateral pressure were quickly stressed to a maximum value with little strain or pore pressure. The maximum effective stress developed probably reflected the intrinsic strength of the consolidated soil structure. Further increases in strain caused the radial stresses to exceed the confining pressure, and axial strain and lateral bulging increased without an increase in stress.

Higher lateral pressures prevented lateral displacement of the soil and probably caused the soil structure to rearrange, allowing pore pressure to develop. This mechanism would be characterized by axial stress increasing steadily with axial strain. As higher lateral pressures were applied, the axial stress kept increasing with axial strain until failure was by shear rather than by bulging, also characterized by a rapid decrease of axial stress at failure.

The consolidated southwestern Iowa loessial soils all exhibited similar stress-strain curves for both the natural and wetted conditions. The magnitude of the maximum effective stress of the soil was related to its texture, clay content, and moisture content. The most resistant soil to axial deformation was the low clay content, coarse textured sample, 49 B-2. The secondary loessial soil 99 C-1 was a little less resistant to axial deformation than its parent, 49 B-2, possibly because of its slightly finer texture and higher clay content. Soil 97 B-1 was the least resistant to axial deformation, which may be attributed to its high clay content, moisture content, and fine texture.

Increasing the moisture content to near saturation reduced the axial deformation resistance for all three soils. The soil structure may be weakened by the softening of the clay hull surrounding the grains or the loss of cohesion of the clay in the soil matrix by the increased moisture content.

Careful examination of the stress-strain curves showed that the consolidated loessial soils cannot be strained beyond fifteen percent axial strain without plastic deformation or shear occurring.

B. Coefficient of Internal Friction

Mohr diagrams were constructed to determine the shearing strength parameters of cohesion and coefficient of internal friction. The shearing strengths were determined using both the effective stress and the total stress analysis. Mohr diagrams for each of the three loessial soils based on effective stress are shown in Figures 13, 14, and 15, and based on total stress in Figures 16, 17, and 18. The shear envelopes were drawn using calculated values of cohesion and coefficient of internal friction from least squares fitting of data (25).

The effective coefficient of internal friction ($\tan \phi'$) for sample 49 B-2 was found to be 0.576 in the natural condition and 0.569 in the wetted condition.

The $\tan \phi'$ for sample 99 C-1 in the natural and wetted conditions was found to be 0.568 and 0.549, respectively. The variation in $\tan \phi'$

between 49 B-2 and 99 C-1 is not significant. Since sample 99 C-1 was derived from the sediments eroded from the area in which 49 B-2 was obtained, this similarity is partially expected. The reworked loess structure appears to have little or no effect on tan ϕ'.

Sample 97 B-1 did not develop the tan ϕ' exhibited by both 49 B-2 and 99 C-1 until a minimum effective normal stress was exerted on the soil structure. Once the minimum effective normal stress was exceeded, the grains were brought into closer contact which allowed tan ϕ' to attain those values obtained for 49 B-2 and 99 C-1. This behavior of tan ϕ' is shown by the break in the shear envelope in Figure 15.[1] Both the natural moisture content and near saturated soils showed this break. For the natural condition, tan ϕ' was 0.264 until the effective normal stress exceeded 50 psi, at which point tan ϕ' increased to 0.619. For the wetted condition, tan ϕ' was 0.320 until the effective normal stress exceeded 40 psi, at which point tan ϕ' increased to 0.567.

Sample 97 B-1 is a primary loessial soil having a high clay content. Petrographic analysis of a similar primary loess has shown that the larger grains are surrounded with a thin shell of montmorillonite clay (3). These coated particles are thus dispersed through a clay matrix. The tan ϕ' cannot be fully developed until these larger grains are brought into closer contact, thus increasing the friction between them. The increase in tan ϕ' to maximum value probably occurred when the developed effective normal stress exceeded the force necessary to

[1]Mohr circle for lateral pressure of 60 psi for the wetted condition was rejected in part because the dry density of the specimen was 8.6 pcf less than the average dry density of the soil.

shear the interface surface between the tightly adhered clay hull and the clay matrix which allowed the grains to come into closer contact. Increasing the moisture content to saturation softened the clay, thus reducing the effective normal stress necessary to develop maximum tan ϕ'.

Sample 99 C-1 also has a high clay content similar to 97 B-1; however, it did not exhibit a break in the shear envelope. A possible explanation may be the difference in the soil structure between the secondary and primary loess. Petrographic analysis of secondary loess has shown that the clay hulls are for the most part absent (3), thus tan ϕ' may develop without overcoming the resistance offered by clay hull-matrix interface. Sample 49 B-2 has the clay hulls but not the clay matrix.

The variation in tan ϕ' among the three loessial soils in the natural and wetted conditions was not significant once tan ϕ' was fully developed. Saturation reduced tan ϕ' by only 0.01 to 0.05, thus moisture content appears to have little effect on tan ϕ'.

The apparent coefficient of internal friction (tan ϕ), like tan ϕ', did not vary with density, moisture content, or air content once the maximum tan ϕ was developed; however, tan ϕ was found to be a slightly lower value than tan ϕ'. The average maximum tan ϕ for the three loessial soils tested was 0.544. High clay content primary loess, 97 B-1, did not develop this maximum tan ϕ until the total normal stress was exceeded. The average value of the initial tan ϕ of 97 B-1 was 0.217.

Consolidation of the soil prior to shear testing tended to minimize the volume reduction of the soil mass during the application of the axial load, thus it would be reasonable to assume that the soil structure would compress very little and the axial load would largely be distributed as intergranular stress. For the soils tested in their natural moisture condition, the maximum pore pressures developed were 3.5, 7.2, and 12.7 psi for samples 49 B-2, 99 C-1 and 97 B-1, respectively. Saturation increased the maximum pore pressures to 5.1, 9.5, and 14.5 psi, respectively.

The rate at which pore air and pore water pressure would develop depends on the compressibility of the soil structure and the compressibility of the pore fluid.

Sample 49 B-2 is a low natural moisture content, low clay content loess and was found to be the most resistant to consolidation and axial deformation of the three soils tested. In the natural condition, the maximum pore pressure developed was 3.5 psi which increased to 5.1 psi when the soil was saturated. The lower pore pressures developed in the natural condition may be attributed to the small volume change and the compressibility of the pore air. Saturation reduced the consolidation and axial deformation resistance and also reduced the pore air content. As the soil mass volume was reduced by the application of the axial load, the incompressibility of the pore water allowed higher pore pressure to develop.

Sample 97 B-1 is a high natural moisture content, high clay content soil and was found to be the least resistant to consolidation and axial deformation of the three soils tested. In the natural moisture condition, the maximum pore pressure developed was 12.7 psi which increased to 14.5 psi when the soil was saturated. These higher pore pressures developed may be attributed to the greater volume change and the incompressibility of the pore water.

Sample 99 C-1 is the intermediate soil as to moisture content and clay content and the maximum pore pressures developed were between those values developed for 49 B-2 and 97 B-1. Therefore, as the clay content of a loessial soil increases, the resistance to volume reduction of the soil mass for an applied load decreases which allows a higher pore pressure to develop. Saturating a soil further increases the developed pore pressure because of the additional reduction of resistance to volume change, and the stress change is absorbed by the incompressible water instead of the pore air which has been removed.

A composite diagram of the shear envelopes of Figures 13, 14, and 15 (based on effective stress analysis) is presented in Figure 19 to illustrate the small variation of tan ϕ' of the three loessial soils. The tan ϕ' varied from 0.54 to 0.62 with the average tan ϕ' being 0.575. Sample 97 B-1 did not develop the average tan ϕ' until a minimum effective normal stress was exerted on the soil structure. The average tan ϕ' below the minimum effective normal stress was 0.292.

The variation of tan ϕ was also small with an average value of 0.544 once the maximum tan ϕ was developed. Sample 97 B-1 had an average tan ϕ of 0.217 until the minimum total normal stress was exceeded.

The consistent values of tan ϕ', regardless of moisture content, density, or clay content, was described by Clevenger (12) for Nebraska-Kansas loessial soils. Clevenger's reported values of 0.60 to 0.65 are slightly higher than those values obtained for the Iowa loess in this study. Possible explanation may be the difference in source area. The effect of clay content on tan ϕ' was not reported by Clevenger.

Olson (13) also found consistent values of tan ϕ' for southwestern

Iowa loessial soils, but his reported values of 0.44 to 0.46 were much lower than values obtained in this study. Olson determined the shear strength by the direct shear test with maximum normal loads of 28 psi.

Summaries of the triaxial shear test data are tabulated in Tables 6, 7, and 8 in the Appendix.

C. Cohesion

The effective cohesion (c') for sample 49 B-2 was found to be 2.25 psi in the natural condition and 0.37 psi in the wetted condition.

The c' for sample 99 C-1 in the natural and wetted conditions was found to be 1.2 psi and 0.7 psi, respectively. Sample 99 C-1 was derived from the sediments eroded from the area in which 49 B-2 was obtained, but its clay content was similar to sample 97 B-1.

The c' for sample 97 B-1 in the natural and wetted conditions was found to be 8.3 psi and 4.3 psi, respectively. Sample 97 B-1 is a high clay content soil and the higher value of c' would be expected.

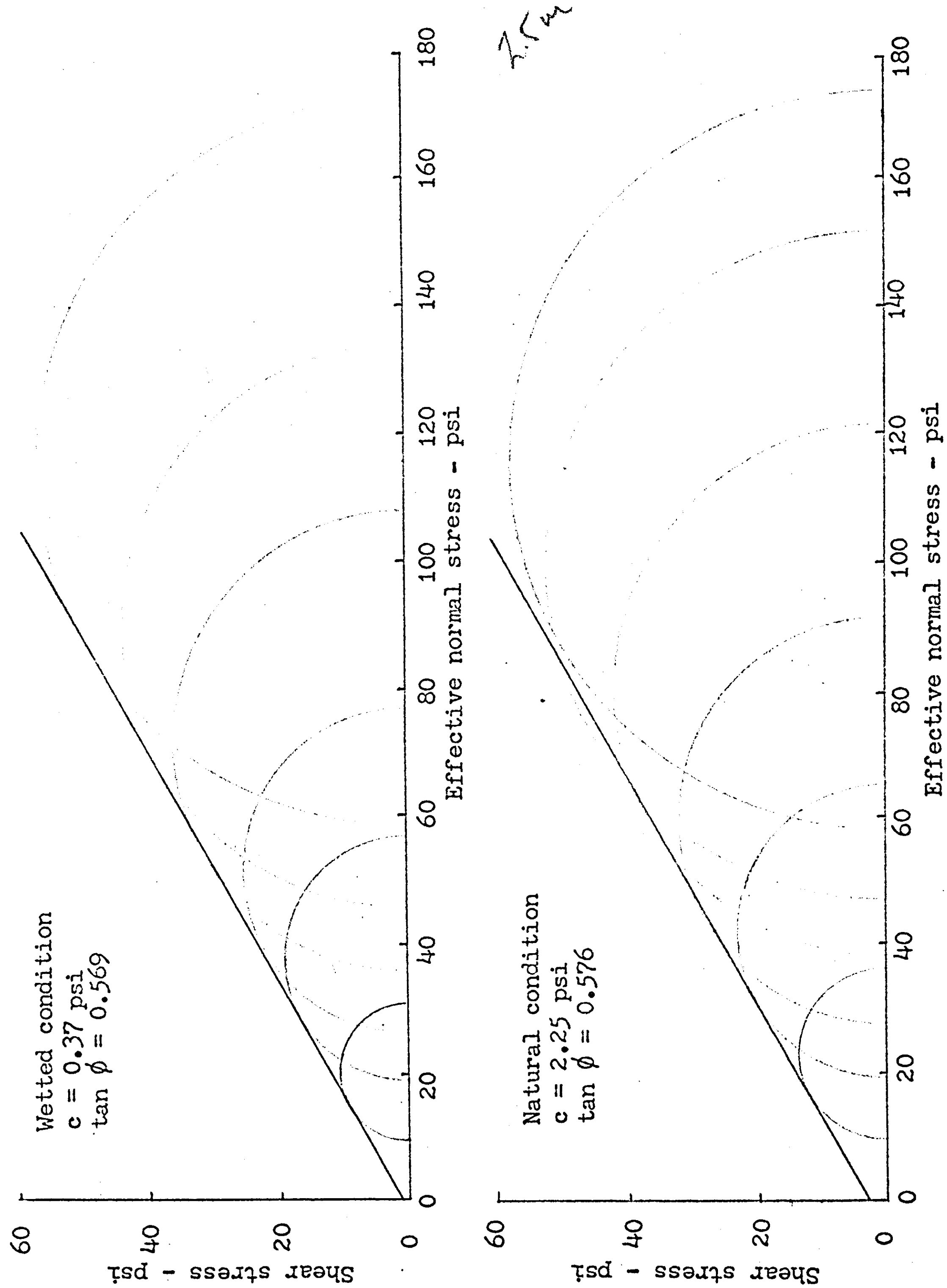
The secondary loessial soil, 99 C-1, similar to 97 B-1 in clay content, developed less c' than either 49 B-2 or 97 B-1. The cohesion of loessial soils may thus be largely attributed to the clay hull-matrix interface resistance with minor contribution to cohesion by the matrix clay.

The c' of the loessial soils was reduced considerably by increasing the water content of the soil to saturation. The c' developed at high moisture content and saturation would probably be unreliable and, therefore, the effective cohesion for saturated loessial soils should be considered as zero.

The apparent cohesion (c) for sample 49 B-2 was found to be 2.41 and zero psi in the natural and wetted conditions, respectively. The c for sample 99 C-1 was zero psi for both the natural and wetted conditions. The c for sample 97 B-1 was 8.6 psi in the natural condition and 6.0 psi in the wetted condition.

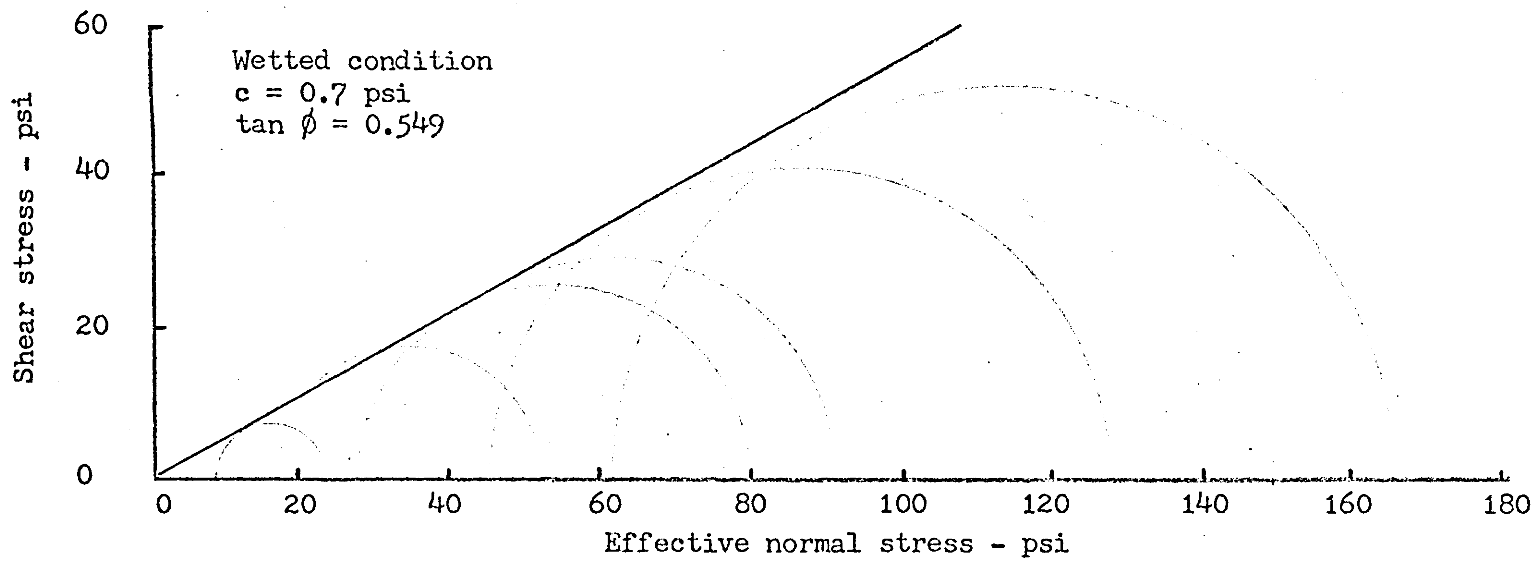
For the natural moisture condition, the apparent cohesion ranged from zero to slightly greater than the observed effective cohesion for each of the three loessial soils. Saturating the loessial soils reduced the apparent cohesion. The high clay content primary loess lost only a small part of its apparent cohesion when saturated, thus behaving as a clayey soil.

Figure 13. Mohr diagrams for sample 49 B-2 based on effective stress analysis



Wetted condition
c = 0.37 psi
tan ϕ = 0.569
Natural condition
c = 2.25 psi
tan ϕ = 0.576
Shear stress - psi
Effective normal stress - psi
0
20
40
60
80
100
120
140
160
180

Figure 14. Mohr diagrams for sample 99 C-1 based on effective stress analysis


Wetted condition
c = 0.7 psi
tan ϕ = 0.549
Shear stress - psi
60
40
20
0
0
20
40
60
80
100
120
140
160
180
Effective normal stress - psi

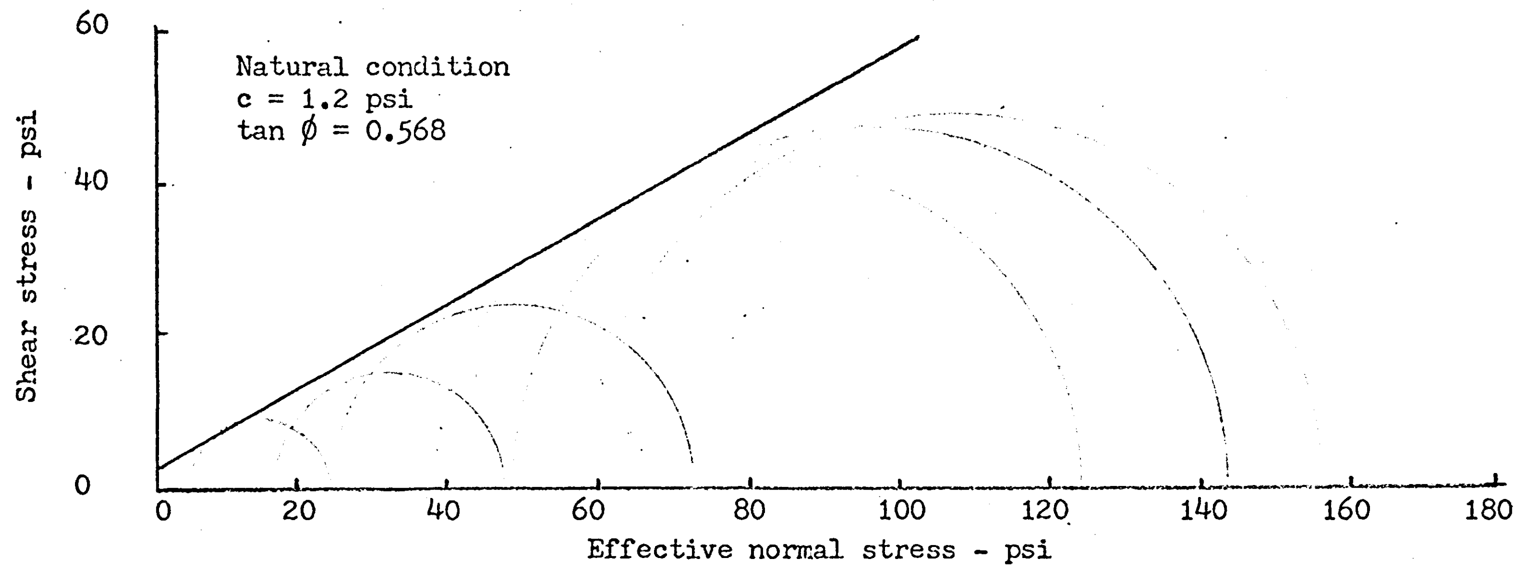

Natural condition
c = 1.2 psi
tan ϕ = 0.568
Shear stress - psi
60
40
20
0
0
20
40
60
80
100
120
140
160
180
Effective normal stress - psi

Figure 15. Mohr diagrams for sample 97 B-1 based on effective stress analysis

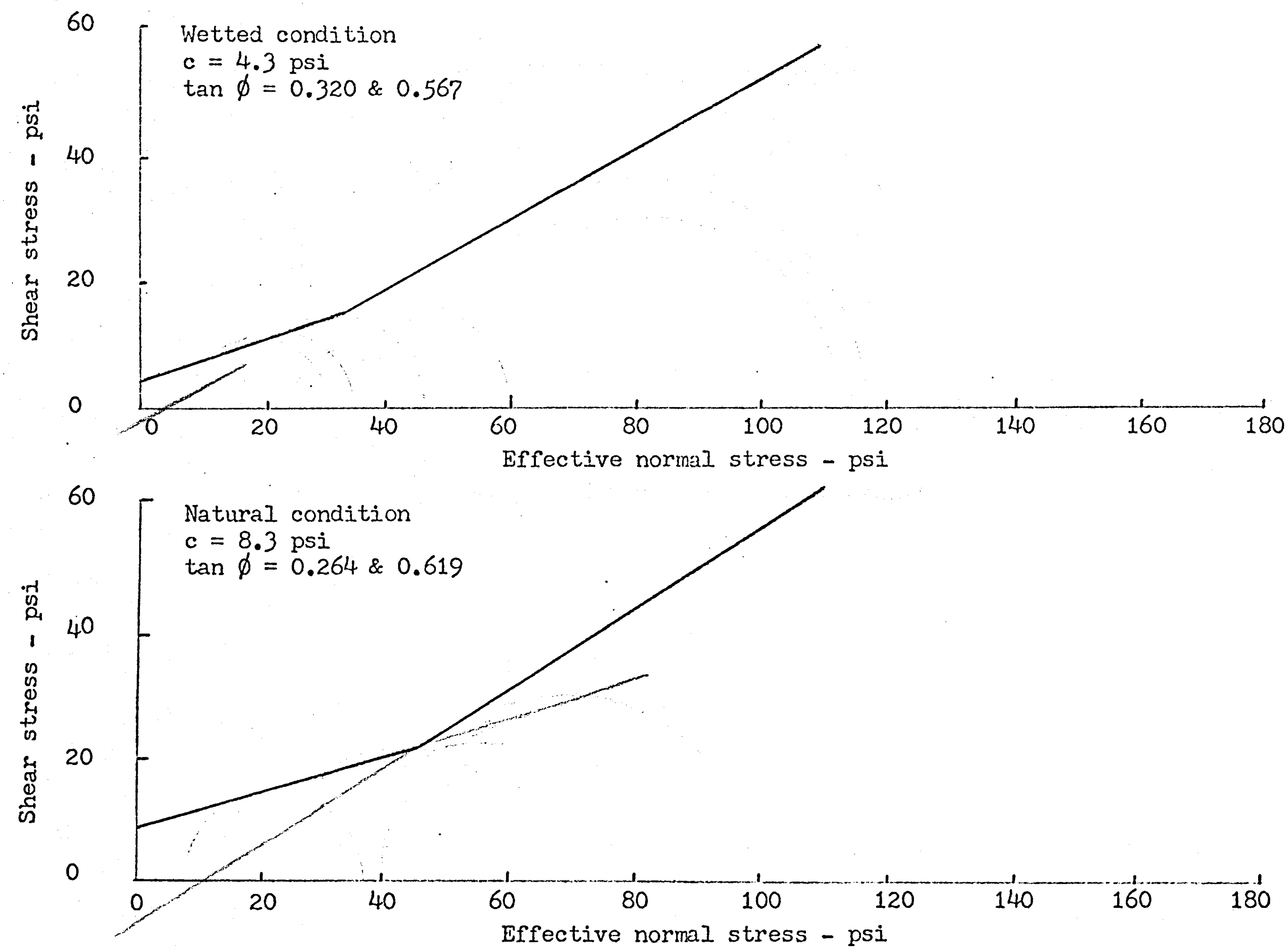

Wetted condition
c = 4.3 psi
tan ø = 0.320 & 0.567
Shear stress - psi
0
20
40
60
0
20
40
60
80
100
120
140
160
180
Effective normal stress - psi
Natural condition
c = 8.3 psi
tan ø = 0.264 & 0.619
Shear stress - psi
0
20
40
60
0
20
40
60
80
100
120
140
160
180
Effective normal stress - psi

Figure 16. Mohr diagrams for sample 49 B-2 based on total stress analysis

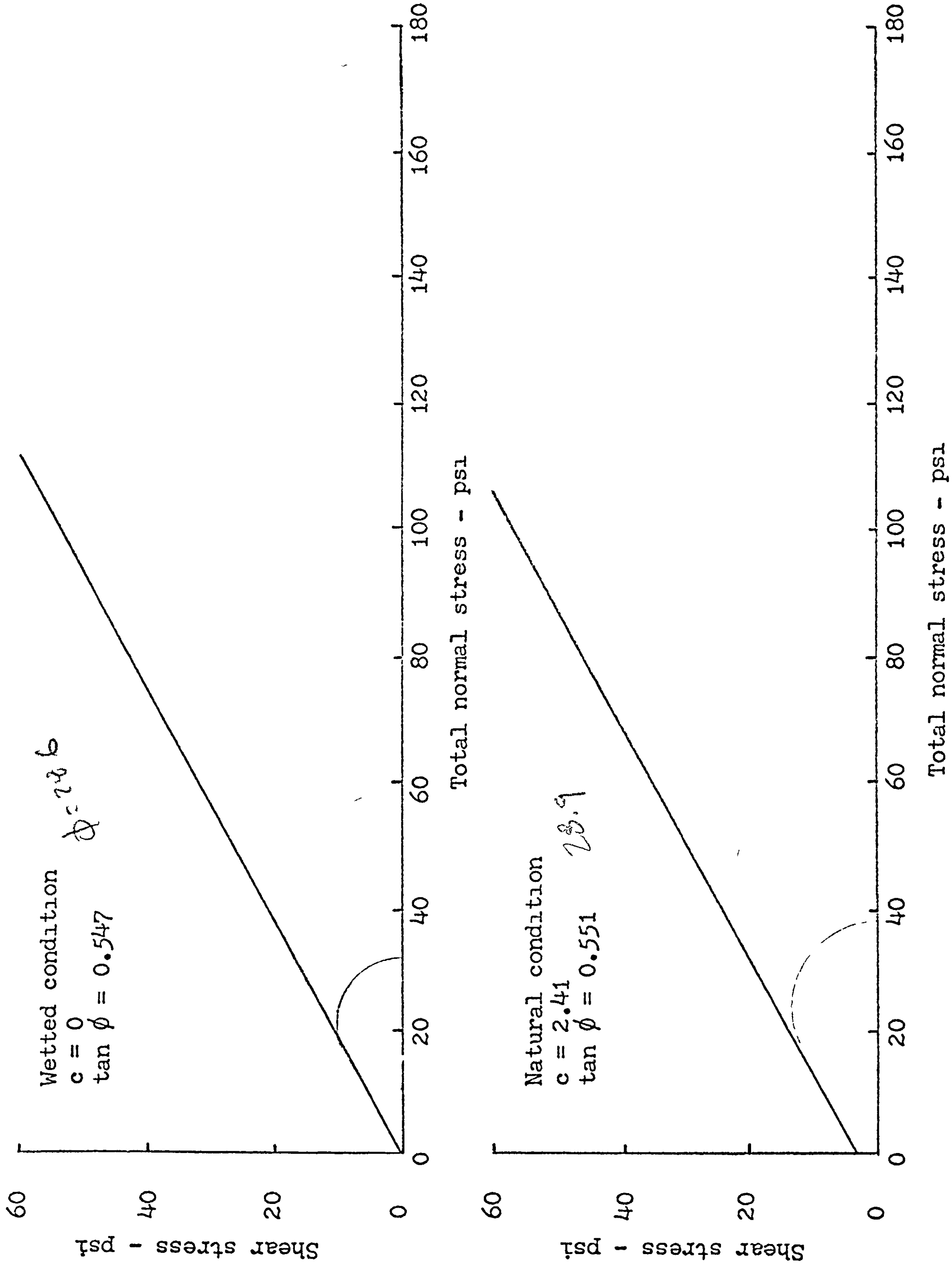

Wetted condition
c = 0
tan ϕ = 0.547
ϕ = 28.6
Natural condition
c = 2.41
tan ϕ = 0.551
28.9
Shear stress - psi
Total normal stress - psi
0
20
40
60
80
100
120
140
160
180

Figure 17. Mohr diagrams for sample 99 C-1 based on total stress analysis

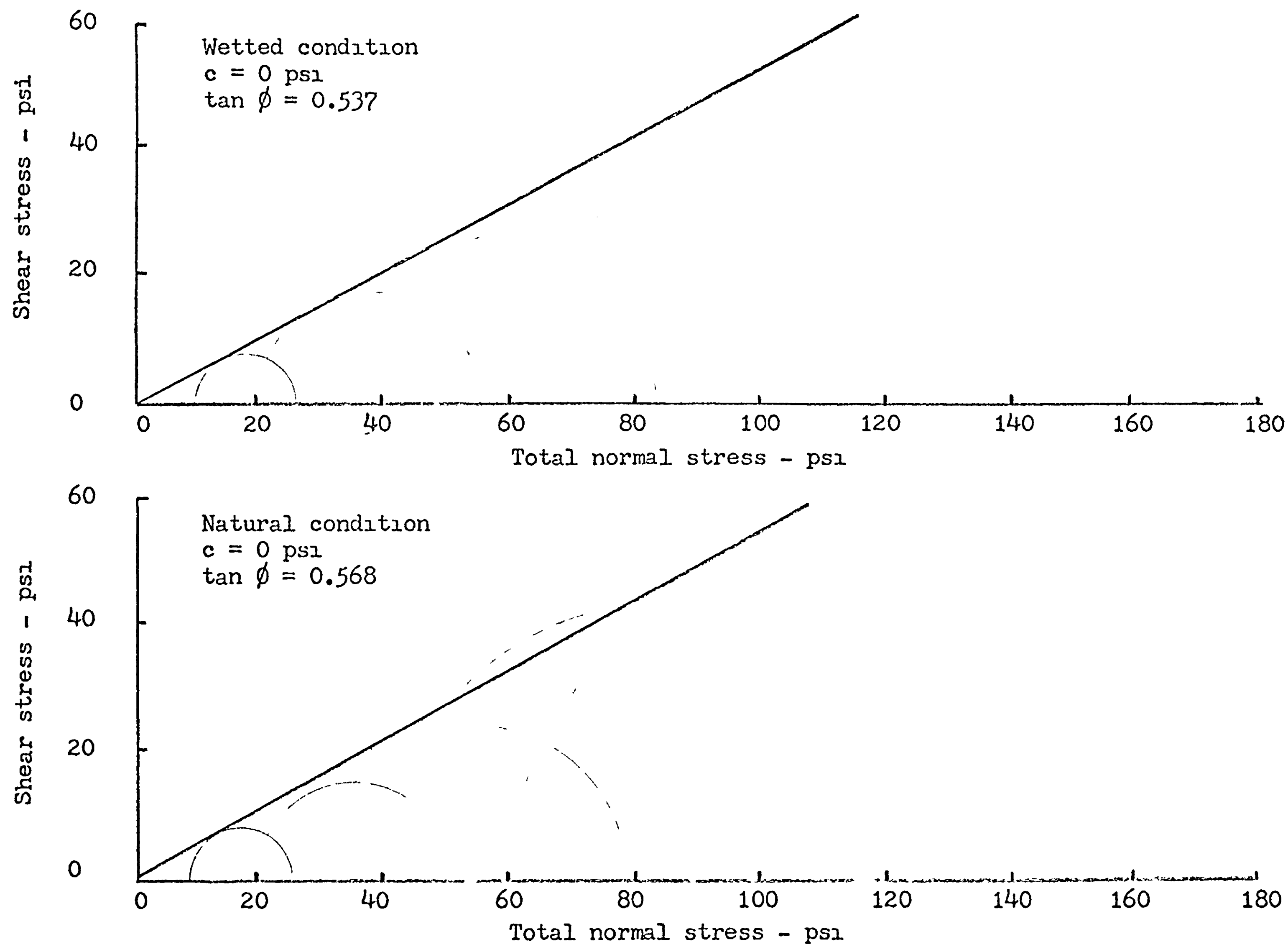

Wetted condition
c = 0 psi
tan ϕ = 0.537
Shear stress - psi
60
40
20
0
0
20
40
60
80
100
120
140
160
180
Total normal stress - psi
Natural condition
c = 0 psi
tan ϕ = 0.568
Shear stress - psi
60
40
20
0
0
20
40
60
80
100
120
140
160
180
Total normal stress - psi

Figure 18. Mohr diagrams for sample 97 B-1 based on total stress analysis

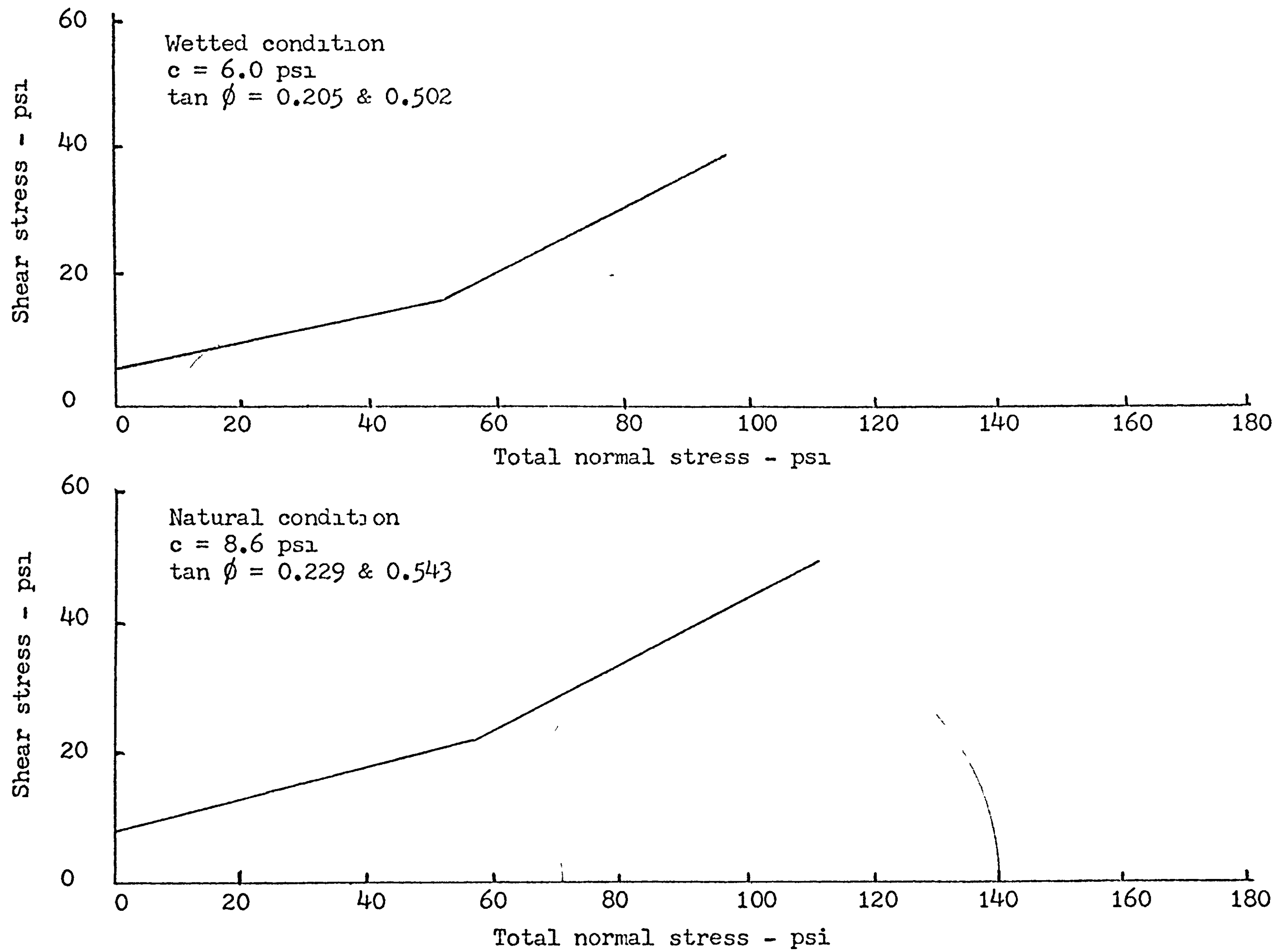
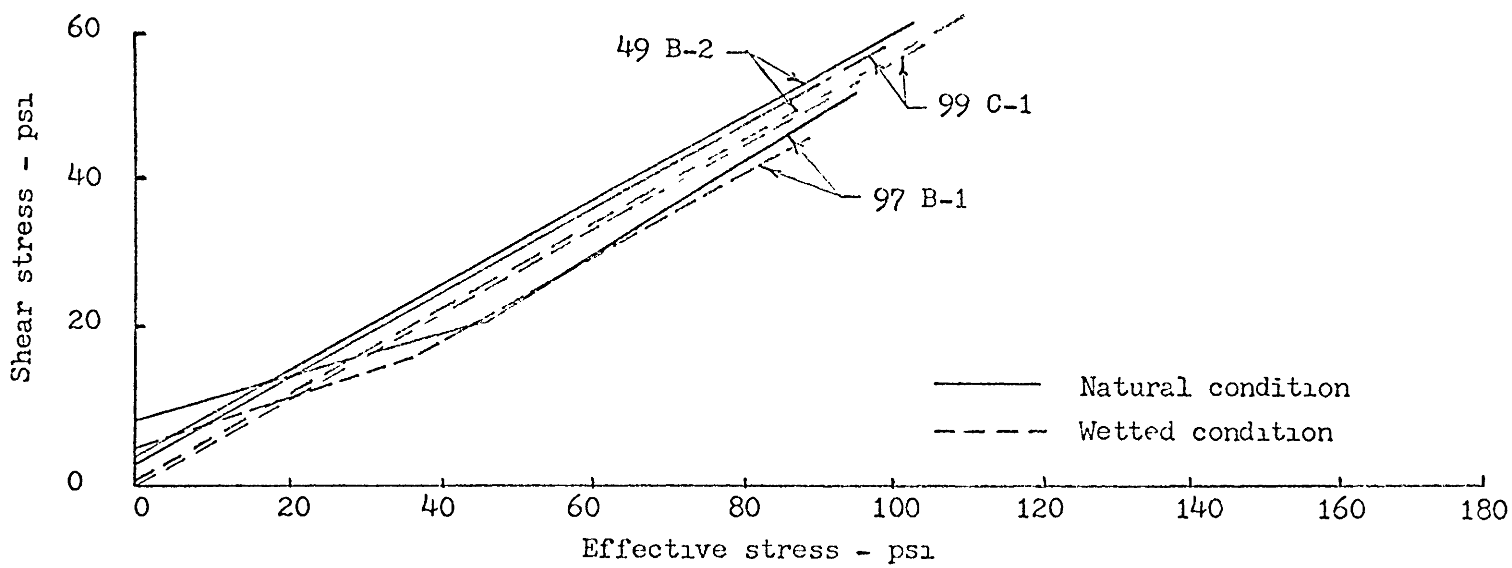

Wetted condition
c = 6.0 psi
tan ϕ = 0.205 & 0.502
Shear stress - psi
Total normal stress - psi
Natural condition
c = 8.6 psi
tan ϕ = 0.229 & 0.543
Shear stress - psi
Total normal stress - psi

Figure 19. Shear envelopes for samples 49 B-2, 99 C-1, and 97 B-1 based on effective stress analysis



60
40
20
0
Shear stress - psi
0
20
40
60
80
100
120
140
160
180
Effective stress - psi
49 B-2
99 C-1
97 B-1
Natural condition
Wetted condition

VII. SUMMARY AND CONCLUSION

The consolidation behavior of partially saturated and near saturated loessial soils was determined by three-dimensional consolidation in the triaxial cell measuring the volume of air and water expelled from the specimen upon the application of the cell pressure. The volume of air expelled was collected and measured at atmospheric pressure and room temperature. Several difficulties were encountered in the consolidation analysis. These problems were:

1. To determine the relationship of the total volume of water and air collected at atmospheric pressure to the actual volume change of the soil specimen consolidated by a cell pressure of known magnitude.
2. To determine the final void ratio for each consolidation pressure applied.
3. To construct an e-log p diagram for a soil using six specimens, each consolidated by a different cell pressure, instead of a single specimen incrementally loaded. The initial void ratio of the six specimens varied in such a manner that a good e-log p diagram could not be obtained.
4. To determine the relationship between the vertical deflection and the volume change after consolidation.
5. To determine the settlement-time relationship for a specific loading.

These problems were analyzed and these partial solutions are presented:

1. Comparison of the three-dimensional consolidation data with

one-dimensional consolidation data found in the literature for similar loessial soils appears to indicate that the volume change of the soil mass is largely attributed to the volume of water expelled and that the volume of pore air expelled contributes very little to the volume change of the soil mass.

2. A good approximation of the final void ratio obtained after consolidation by a pressure of known magnitude is calculated assuming that the volume change of the soil is equal to the volume of water drained, and then reducing this calculated value by 5 to 10 percent to account for the effect of pore air. As the moisture content of the soil increases to saturation, this correction for pore air should be reduced until no correction is made for a fully saturated soil.

3. An acceptable e-log p diagram may be obtained by determining an average initial void ratio ($\bar{e}_o$) for the tested specimens and relating the percent change of void ratio for each consolidation pressure to $\bar{e}_o$, thus normalizing the void ratio for each consolidating pressure.

4. The relationship between the vertical deflection and volume change was not obtained because of insufficient data.

5. The settlement-time relationship for a specific loading was not obtained because of insufficient data.

The consolidation behavior of loessial soils determined by three-dimensional consolidation can only be expressed in qualitative terms since the analysis is based on incomplete data and hypotheses. Some of these general observations of the consolidation behavior of loess are:

1. Potential settlement of medium density loess with low natural moisture content was small up to about 5 TSF of pressure. The consolidation resistance of loessial soils decreased as the moisture content and clay content increased. Secondary loess consolidated similarly to primary loess and was also affected by moisture and clay content.
2. Wetting the loessial soil to near saturation induced a more rapid consolidation and greatly reduced the consolidation resistance.

An evaluation of the shearing strength of loessial soils wetted (to near saturation) and consolidated was investigated using the consolidated-undrained triaxial shear test with pore pressure measurements. Although the following remarks on shearing strength are applicable only for the particular southwestern Iowa loessial soils tested, the similarity of loessial soils throughout the world, suggests broader interpretation of the findings.

1. The consolidated primary loessial soil with low moisture and clay content (49 B-2) developed a tan ϕ' of 0.576. Tan ϕ' was not affected significantly by saturation since it was only reduced slightly to 0.569.
2. The consolidated secondary loess (99 C-1) also exhibited similar values of tan ϕ' of 0.568 and 0.549 in the natural and wetted conditions, respectively. The reworked loess structure appears to have little effect on tan ϕ'.

3. The consolidated primary loess with high clay content (97 B-1) developed a tan ϕ' of 0.264 until the effective normal stress exceeded 50 psi, at which point tan ϕ' increased to 0.619. This change of tan ϕ' was also shown for the saturated condition, in that the initial tan ϕ' developed was 0.320 until the effective normal stress exceeded 40 psi, at which point tan ϕ' increased to 0.567.
4. Tan ϕ' of the three loessial soils tested did not vary significantly with density, moisture content, or air content once the maximum value was developed. The average value of the maximum tan ϕ' for the southwestern Iowa loessial soils was 0.575. High clay content primary loessial soil (97 B-1) did not develop this maximum value until the minimum effective normal stress was exceeded. The average initial tan ϕ' of 97 B-1 was 0.292.
5. Tan ϕ, like tan ϕ', did not vary with density, moisture content, or air content among the three loessial soils tested once the maximum value was developed. The average value of tan ϕ was 0.544 which is slightly lower than the average tan ϕ'. High clay content primary loess (97 B-1) did not develop this maximum tan ϕ until the minimum total normal stress was exceeded. The initial value of tan ϕ of 97 B-1 averaged 0.217.
6. The consolidated loessial soils did not develop high pore pressures during the test because the stress change was largely absorbed as intergranular stress. Primary loess

with low moisture and low clay content developed the lowest pore pressures while the high moisture and high clay content loessial soils developed the highest pore pressures. The resistance to volume change was further reduced by saturating the soil, allowing still greater pore pressures to develop. The highest pore pressure measured was 14.5 psi for the saturated high clay content primary loessial soil.

7. The consolidated loessial soil with low natural moisture content and low clay content (49 B-2) developed a c' of 2.25 psi which was reduced to 0.37 psi when the soil was saturated.
8. The consolidated secondary loess (99 C-1) developed a c' of 1.2 psi in the natural condition and 0.7 psi in the wetted condition.
9. The consolidated primary loess of high clay content (97 B-1) developed a c' of 8.3 and 4.3 psi in the natural and wetted conditions, respectively.
10. Primary loessial soils of high clay content have some cohesion in the natural condition. Low clay content and secondary loessial soils have little or no cohesion. Saturation of loessial soils quickly reduced the cohesion (c and c') to low values or zero. The cohesion measured for saturated soils would be highly unreliable and the cohesion should be considered as zero for foundation analyses.

Prewetting and consolidation of loessial soil foundations eliminates the engineering problem of excessive settlement under structural loads by inducing settlement before construction. The low clay content and secondary loessial foundations which have been prewetted and consolidated may be evaluated using values of 0.575 for tan ϕ' and zero for c'. The prewetting and consolidation of a high clay content primary loessial soil is more difficult in that a surcharge sufficient to exert the minimum effective normal stress must be applied to develop the maximum average tan ϕ' of 0.575. If the surcharge is insufficient to develop this value of tan ϕ', the foundation must then be evaluated using the lower value of 0.292. Even for a high clay content soil, the value of c' for a saturated condition must be taken as zero.

VIII. LITERATURE CITED

1. Terzaghi, K. Discussion [of Holtz, W. G. and Gibbs, H. J. Consolidation and related properties of loessial soils]. American Society for Testing Materials Special Technical Publication 126: 30-32. c1952.

2. Gibbs, H. J. and Holland, W. Y. Petrographic and engineering properties of loess. U. S. Bureau of Reclamation Engineering Monograph 28. 1960.

3. U. S. Bureau of Reclamation. Petrographic examination of loessial material; Ashton pile testing program; Missouri River Basin Project. U. S. Bureau of Reclamation Petrographic Laboratory Report Pet-94. 1952.

4. U. S. Bureau of Reclamation. Laboratory studies on foundation material (loess); Spring Creek Siphon site; Cambridge Canal; Missouri River Basin Project, Nebraska. U. S. Bureau of Reclamation Earth Materials Laboratory Report EM-254. 1950.

5. U. S. Bureau of Reclamation. Laboratory studies on structure foundation material (loess); Courtland Canal; Bostwick Division; Missouri River Basin Project, Nebraska-Kansas. U. S. Bureau of Reclamation Earth Materials Laboratory Report EM-251. 1950.

6. U. S. Bureau of Reclamation. Laboratory tests of loess material for the foundations and embankments of the railroad relocation at Trenton Dam; Frenchman-Cambridge Division; Missouri River Basin Project, Nebraska. U. S. Bureau of Reclamation Earth Materials Laboratory Report EM-232. 1950.

7. U. S. Bureau of Reclamation. Report of loess research studies for Ashton Pile Testing Program; Lower Platte River Area; Missouri River Basin Project, Nebraska. U. S. Bureau of Reclamation Earth Materials Laboratory Report EM-278. 1951.

8. Davidson, D. T. and associates. Geological and engineering properties of Pleistocene materials in Iowa. Iowa State University of Science and Technology Engineering Experiment Station Bulletin 191. 1960.

9. Taylor, D. W. Fundamentals of soil mechanics. New York, John Wiley and Sons, Inc. 1948.

10. Gibbs, H. J. Estimating foundation settlement by one-dimensional consolidation tests. U. S. Bureau of Reclamation Engineering Monograph 13. 1953.

11. Akroyd, T. N. W. Laboratory testing in soil engineering. London, Soil Mechanics, Ltd. 1957.

12. Clevenger, W. A. Experiences with loess as foundation material. American Society of Civil Engineers Soil Mechanics and Foundation Division Journal 82, SM3: 1025-1 - 1025-26. 1956.

13. Olson, G. R. Direct shear and consolidation tests of undisturbed loess. Unpublished M.S. thesis. Ames, Iowa, Library, Iowa State University of Science and Technology. 1958.

14. Carrillo, N. Simple two- and three-dimensional cases in theory of consolidation of soils. Journal of Mathematics and Physics 21, No. 1: 1-5. 1942.

15. Biot, M. A. General theory of three-dimensional consolidation. Journal of Applied Physics 12: 155-164. 1941.

16. Silveira, I. Da. Consolidation of a cylindrical clay sample with external radial flow of water. International Conference on Soil Mechanics and Foundation Engineering, 3rd, Zurich, 1953, Proceedings 1: 55-56. 1953.

17. Aboshi, H. and Monden, H. Three dimensional consolidation of saturated clays. International Conference on Soil Mechanics and Foundation Engineering, 5th, Paris, 1961, Proceedings 1: 559-562. 1961.

18. McKinlay, D. G. A laboratory study of consolidation in clays with particular reference to conditions of radial pore water drainage. International Conference on Soil Mechanics and Foundation Engineering, 5th, Paris, 1961, Proceedings 1: 225-228. 1961.

19. Holtz, W. G. and Hilf, J. W. Settlement of soil foundations due to saturation. International Conference on Soil Mechanics and Foundation Engineering, 5th, Paris, 1961, Proceedings 1: 673-679. 1961.

20. Bishop, A. W. and Henkel, D. J. The measurement of soil properties in the triaxial test. 2nd ed. London, Edward Arnold (Publishers), Ltd. 1962.

21. Simonson, R. W., Riecken, F. F., and Smith, G. D. Understanding Iowa soils. Dubuque, Iowa, William C. Brown Company. 1952.

22. Bishop, A. W. and Donald, I. B. The experimental study of partly saturated soil in the triaxial apparatus. International Conference on Soil Mechanics and Foundation Engineering, 5th, Paris, 1961, Proceedings 1: 13-21. 1961.

23. Alpan, I. The dissipation function for unsaturated soils. International Conference on Soil Mechanics and Foundation Engineering, 5th, Paris, 1961, Proceedings 1: 3-5. 1961.

24. Hewes, L. I. and Oglesby, C. H. Highway engineering. New York, John Wiley and Sons, Inc. 1954.

25. U. S. Bureau of Reclamation. Earth manual. Washington, D. C. U. S. Government Printing Office. 1960.

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X. APPENDIX

Table 3. Summary of consolidation results for sample 49 B-2

Lab test Nr.	Initial specimen data: Dry density pcf	Moisture content %	Void ratio	Volume ft^3	Volume change: ΔV_{H_2O} ft^3	ΔV_{air} ft^3	ΔV_{total} ft^3	Normalized wrt $\bar{e}_o$ [a]: e1 based on V_{H_2O}	e1 based on V_{total}
Natural condition									
II-10-N	85.0	14.3	0.969	0.02034	0.00056	0.00008	0.00064	0.902	0.899
II-20-N	87.9	9.4	0.901	0.02049	0.00051	0.00100	0.00151	0.852	0.760
II-30-N	92.5	11.7	0.832	0.01913	0.00062	0.00204	0.00266	0.758	0.564
II-40-N	88.2	7.2	0.898	0.02010	0.00049	0.00299	0.00348	0.855	0.570
II-50-N	91.5	10.5	0.828	0.01994	0.00059	0.00430	0.00489	0.775	0.380
II-60-N	86.6	6.4	0.927	0.02035	0.00049	0.00415	0.00464	0.880	0.489
Wetted condition									
II-10-S	87.1	6.2	0.919	0.02020	0.00028	- -	0.00028	0.01992	0.898
II-20-S	89.1	9.2	0.876	0.02034	0.00144	0.00019	0.00133	0.01901	0.755
II-30-S	86.8	9.4	0.927	0.02070	0.00172	0.0016	0.00288	0.01782	0.662
II-40-S	89.8	8.0	0.862	0.01867	0.00156	0.00098	0.00254	0.01613	0.610
II-50-S	86.7	6.6	0.931	0.02012	Leaky burette				
II-60-S	87.4	12.7	0.912	0.02035	0.00158	0.00412	0.00570	0.01455	0.379
Average	88.2	9.3	0.899[b]						

[a]Normalized with respect to $\bar{e}_o$.

[b] $\bar{e}_o$: arithmetic average of the initial void ratios of the twelve specimens tested.

Table 4. Summary of consolidation results for sample 99 C-1

Lab test Nr.	Initial specimen data: Dry density pcf	Moisture content %	Void ratio	Volume ft^3	Volume change: ΔV_{H_2O} ft^3	ΔV_{air} ft^3	ΔV_{total} ft^3	Normalized wrt $\bar{e}_o$ [a]: e_1 based on V_{H_2O}	e_1 based on V_{total}
Natural condition									
III-10-N	75.8	20.5	1.169	0.01992	0.00050	0.00013	0.00063	1.115	1.101
III-20-N	81.2	16.8	1.026	0.01999	0.00050	0.00013	0.00063	0.975	0.962
III-30-N	81.5	17.1	1.018	0.02019	0.00034	0.00039	0.00123	0.934	0.895
III-40-N	85.9	6.6	6.914	0.01978	0.00056	0.00100	0.00156	0.860	0.763
III-50-N	86.4	9.0	0.903	0.02008	0.00053	0.00071	0.00124	0.853	0.786
III-60-N	83.6	11.9	0.969	0.02141	0.00051	0.00135	0.00186	0.922	0.798
Wetted condition									
III-10-S	83.8	18.6	0.967	0.02034	0.00081	- -	0.00081	0.889	0.889
III-20-S	84.6	18.6	0.944	0.02050	0.00124	0.00016	0.00190	0.827	0.812
III-30-S	89.4	13.8	0.841	0.02062	0.00142	0.00057	0.00199	0.714	0.663
III-40-S	87.7	9.4	0.875	0.02044	0.00260	0.00210	0.00470	0.636	0.444
III-50-S	87.1	11.6	0.866	0.02034	0.00267	0.00236	0.00503	0.640	0.421
III-60-S	80.9	17.2	1.033	0.02009	0.00312	0.00192	0.00504	0.717	0.523
Average	84.8	14.3	0.962[b]						

[a]Normalized with respect to $\bar{e}_o$.

[b] $\bar{e}_o$: arithmetic average of the initial void ratios of the twelve specimens tested.

Table 5. Summary of consolidation results for sample 97 B-1

Lab test Nr.	Initial specimen data				Volume change			Normalized wrt $\bar{e}_o$[a]	
	Dry density pcf	Moisture content %	Void ratio	Volume ft^3	ΔV_{H_2O} ft^3	ΔV_{air} ft^3	ΔV_{total} ft^3	e_1 based on V_{H_2O}	e_1 based on V_{total}
Natural condition									
I-10-N	90.0	22.4	0.824	0.02070	0.00074	0.00014	0.00088	0.773	0.760
I-20-N	78.2	25.4	0.890	0.02070	0.00137	0.00063	0.00200	0.767	0.710
I-30-N	85.2	22.0	0.942	0.02060	0.00141	0.00235	0.00376	0.811	0.588
I-40-N	88.0	22.0	0.880	0.02070	- -	- -	- -	- -	- -
I-50-N	87.9	23.2	0.880	0.02050	0.00135	0.00414	0.00549	0.762	0.382
I-60-N	87.5	23.6	0.890	0.02060	0.00060	0.00419	0.00479	0.830	0.449
Wetted condition									
I-10-S	88.7	23.0	0.862	0.02022	0.00099	- -	0.00099	0.772	0.772
I-20-S	88.4	23.4	0.873	0.020772	0.00141	0.00016	0.00157	0.730	0.714
I-30-S	88.5	23.6	0.869	0.02058	0.00140	0.00112	0.00252	0.742	0.641
I-40-S	84.8	34.5	0.949	0.02015	0.00316	0.00105	0.00421	0.643	0.544
I-50-S	88.8	23.8	0.860	0.02020	0.00222	0.00246	0.00468	0.658	0.432
I-60-S	77.5	28.5	1.128	0.02140	0.00348	0.00202	0.00550	0.780	0.578
Average	86.1	23.8	0.905[b]						

[a]Normalized with respect to $\bar{e}_o$.

[b] $\bar{e}_o$: arithmetic average of the initial void ratios of the twelve specimens tested.

Table 6. Summary of consolidated-undrained triaxial shear results for sample 49 B-2

	Initial specimen data				Test values at failure				
					Total stress		Effective stress		
	Dry density pcf	Moisture content %	Degree of saturation %	Void ratio	σ_1 psi	σ_3 psi	u psi	$\bar{\sigma}_3$ psi	$\bar{\sigma}_1$ psi
Natural condition									
II-10-N	85.0	14.3	39.5	0.969	37.2	10	0.4	9.6	36.2
II-20-N	87.9	9.4	27.9	0.901	65.3	20	1.3	18.7	64.0
II-30-N	92.5	11.7	38.9	0.832	92.7	30	1.9	28.1	90.8
II-40-N	88.2	7.2	21.3	0.898	122.6	40	2.1	37.9	120.5
II-50-N	91.5	10.5	34.0	0.828	156.0	50	3.5	46.4	152.5
II-60-N	86.6	6.4	19.7	0.927	177.1	60	3.3	56.7	173.8
Wetted condition									
II-10-S	87.1	6.2	18.1	0.919	31.0	10	0.4	9.6	30.6
II-20-S	89.1	9.2	28.0	0.876	58.4	20	1.6	18.4	56.3
II-30-S	86.8	9.4	27.3	0.927	80.7	30	4.3	27.5	76.3
II-40-S	89.8	8.0	24.8	0.862	112.7	40	2.9	37.1	109.8
II-50-S	86.7	6.6	19.3	0.931	138.2	50	5.1	44.9	133.1
II-60-S	87.4	12.7	37.3	0.912	175.4	60	2.5	57.5	172.9
Average	88.2	9.3	28.0	0.899					

Table 7. Summary of consolidated-undrained triaxial shear results for sample 99 C-1

	Initial specimen data				Test values at failure				
					Total stress		Effective stress		
	Dry density pcf	Moisture content %	Degree of saturation %	Void ratio	σ_1 psi	σ_3 psi	u psi	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi
Natural condition									
III-10-N	75.8	20.5	46.0	1.169	25.1	10	3.5	21.5	6.5
III-20-N	81.2	16.8	43.2	1.026	50.5	20	4.2	46.3	15.8
III-30-N	81.5	17.1	77.3	1.018	79.1	30	6.4	72.9	23.6
III-40-N	85.9	6.6	19.1	0.914	128.4	40	3.6	124.9	36.4
III-50-N	86.4	9.0	26.3	0.903	150.3	50	5.0	145.3	45.0
III-60-N	83.6	11.9	32.3	0.969	158.5	60	3.4	155.8	56.6
Wetted condition									
III-10-S	83.8	18.6	50.0	0.967	26.0	10	2.7	22.9	7.3
III-20-S	84.6	18.6	51.9	0.944	52.0	20	2.5	49.5	17.5
III-30-S	89.4	13.8	43.3	0.841	83.0	30	3.7	79.1	26.3
III-40-S	87.7	9.4	28.3	0.875	99.2	40	9.5	90.1	30.5
III-50-S	87.1	11.6	34.4	0.866	135.6	50	7.6	128.1	42.4
III-60-S	80.9	17.2	43.8	1.033	167.7	60	1.3	166.4	58.7
Average	84.8	14.3	38.6	0.962					

Table 8. Summary of consolidated-undrained triaxial shear results for sample 97 B-1

	Initial specimen data				Test values at failure				
					Total stress		Effective stress		
	Dry density pcf	Moisture content %	Degree of saturation %	Void ratio	σ_1 psi	σ_3 psi	u psi	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi
Natural condition									
I-10-N	90.0	22.4	70.9	0.824	36.9	10	0.7	36.1	9.3
I-20-N	78.2	25.4	60.5	0.890	52.9	20	5.4	46.6	19.6
I-30-N	85.2	22.0	61.8	0.942	70.0	30	7.2	62.8	22.8
I-40-N	88.0	22.0	66.1	0.880	83.6	40	6.1	77.2	33.9
I-50-N	87.9	23.2	69.9	0.880	111.3	50	11.0	99.6	39.0
I-60-N	87.5	23.6	70.4	0.890	139.6	60	12.7	126.1	47.3
Wetted condition									
I-10-S	88.7	23.0	70.7	0.862	30.8	10	0.8	29.6	9.2
I-20-S	88.4	23.4	73.8	0.873	42.6	20	8.1	32.3	11.9
I-30-S	88.5	23.6	72.0	0.869	61.2	30	13.1	45.0	16.9
I-40-S	84.8	34.5	96.3	0.949	74.5	40	14.5	58.9	25.5
I-50-S	88.8	23.8	73.5	0.860	123.0	50	6.5	116.5	43.5
I-60-S	77.5	28.5	67.0	1.128	121.1	60	9.1	112.0	50.9
Average	86.1	23.8	71.1	0.905					