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# Direct shear and consolidation tests of undisturbed loess

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DIRECT SHEAR AND CONSOLIDATION TESTS

OF UNDISTURBED LOESS

by

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# I. INTRODUCTION

All engineering structures have one thing in common: the use of soil or rock as their ultimate support. In addition, some of them have soil as an integral part of the structure itself. Therefore soils are an important part of engineering construction.

Due to inferior structural properties, some soils have not been extensively used as support for large or important structures. Since in the past their use has been generally avoided, no attempt was made to study the structural properties of these soils. However, in the future as land becomes scarce these soils will out of necessity have to be used. First, however, their structural properties and how these properties vary with other soil variables must be studied. Loess, which covers large areas of the middle west, is one of these soils whose structural properties have not yet been extensively studied.

Although there is some disagreement as to the definition of loess, in this report it is considered a soil predominantly silt in grain size and aeolion in manner of deposition. The majority of the work on the undisturbed properties of loess has been done by the Bureau of Reclamation. Although this work on loess includes the whole Missouri River Basin, it centers in Nebraska where most of the projects on loess have been built. The Engineering Experiment Station at Iowa State College has conducted extensive studies on the physical properties of loess, but this report is the first study of its undisturbed structural properties.

The main object of this initial study was to examine undisturbed properties of loess and learn what, if any, correlation exists between them and other properties such as density, moisture content, gradation, depth, and clay content. A second objective, also important, was to find out what actual stresses loess can resist.

Loess subjected to stresses from an external load may fail in one of two ways:

1. Bearing capacity failure. In this type of failure the soil is unable to support the load without actual destruction of the soil structure. An example is when a section of an earth embankment slips along a curved surface and slides down.
2. Detrimental settlement. This is the condition where a soil consolidates excessively and/or unequally so that the structure cannot operate satisfactorily.

In the case of a bearing capacity failure the soil fails in shear by sliding along an internal surface. A suitable test for determining a soils strength against this type of failure is the direct shear test. The consolidation test may be used to evaluate resistance of a soil to settlement.

# II. REVIEW OF LITERATURE

## A. Shearing Strength of Loess

The shearing strength of a soil is its ability to resist sliding along internal surfaces. There is, however, no one value of shearing strength for a given soil, but rather a wide variation of values due to differences in moisture content, density, and the degree of consolidation. The normally desired laboratory procedure is to attempt to test the soil at the weakest condition in which it could or will exist in the field. This proves difficult, however, since it is extremely hard to reproduce field conditions in the laboratory.

The direct shear and the triaxial test are the two common laboratory methods for determining shear strength. In a direct shear test a constant load is applied normal to the shearing plane and another force is applied parallel to this plane. This latter force is increased until the specimen fails. The maximum force that the specimen resists, divided by the cross sectional area, is the shearing strength for that soil under that normal loading and under the conditions of moisture content, density, and degree of consolidation that prevailed throughout the test. By performing a series of these tests under similar conditions, but with a different normal load each time, a graph can be constructed with the shearing stress as the ordinate and the normal stress as the abscissa. The curve thus obtained will depict Coulomb's formula:

$$S = C + N \tan \phi ,$$

in which

$S$ = shearing stress

$C$ = apparent cohesion

$N$ = normal stress

$\phi$ = angle of shearing resistance of the soil.

Some authors feel Coulomb's formula presents an oversimplification of shearing stress conditions in cohesive soils. Lambe states that the cohesion of a soil is not a constant soil property but is a function of the load carried by the soil structure (7). He visualizes that cohesion is a maximum when the normal force is zero and then decreases in value as the normal force increases to the preconsolidation load on the soil. At this point the cohesion is zero and the shear envelope changes slope. The new slope of the line gives the actual friction angle $\phi$ of the soil under the test conditions.

In the triaxial test the specimen must be cylindrical. The cylindrical surface is covered by a rubber membrane, and a fluid pressure is applied to the membrane and usually kept constant while the axial load is increased until failure. The axial load divided by the cross sectional area is the maximum principal stress, while the fluid pressure, acting normal to the axial load, is the minimum principal stress. Neither of these stresses is, however, the shearing stress. To evaluate the shearing stress one must revert to applied mechanics and apply Mohr's theory. By plotting both principal stresses along the abscissa and using their difference as the diameter, a Mohr circle can be constructed depicting the stresses in the sample. By using different fluid pressures several Mohr circles can be constructed. If one draws a common tangent to these circles it is possible to graphically represent Coulomb's formula as it appears on the previous page. Although the direct shear and triaxial test are different methods of obtaining shearing stresses, a good correlation exists between the two (7).

Bureau of Reclamation workers have written several articles on the shearing strength of loess (12, 13, 14, 15, 16 and 17) but Clevenger (2) recently summarized their work on loess and gave the following conclusions on shearing strength:

1. Differences in the sand and clay content of the loess have only a minor effect on the shearing strength.

2. The moisture content and density at the time of testing control the shearing strength.

3. Shear envelopes are generally parallel to each other, indicating a constant internal friction.

4. Wetted, low density specimens have almost zero shearing strength until the effective normal load reaches 10 psi.

31°-33°

5. Tan $\phi$ usually varies between 0.60 and 0.65. Cohesion is zero for wetted, low density loess, and between 10 and 20 psi for loess at natural moisture contents.

The above conclusions were based on the results of triaxial shear tests of loess from central Nebraska.

## B. Consolidation of Loess

When a soil decreases in volume due to an external load, the soil is said to be consolidating and the phenomenon is known as consolidation. This decrease in volume could, according to Taylor (9), be attributed to three possible factors:

1. Compression of the solid matter.
2. Compression of water and air within the voids.
3. Escape of water and air from the voids.

It can be accurately assumed that neither the water nor the solid matter is compressible. Therefore, if a soil is in the saturated state, it is possible to conclude that the decrease in soil volume is equal to the volume of water forced out of the non-capillary voids. In the case of a partly saturated soil the decrease in volume is due to both water and air being forced out of the voids and by entrapped air being compressed. Since this second case is very complicated, present day theory considers only the first or saturated condition, which is usually the case for clays.

The previous mentioned consolidations are generally referred to as primary consolidation, whereas a plastic deformation of the soil under a constant load is called secondary consolidation. This secondary consolidation occurs much later in the soil and is usually far less in magnitude than primary consolidation.

The theory of consolidation, first proposed by Terzaghi (9), is based on a stress-strain-time relationship for the primary consolidation of saturated soils. His theory also assumes that primary consolidation causes only vertical drainage of the pore water and is a one-dimensional compression. This is the case in the laboratory when the consolidation test is run, but it isn't always true in the field. The weights of buildings cause compressions at shallow depths that are definitely three-dimensional, while in a deeply buried strata or under large fills they are essentially one-dimensional (9).

Because of its unusual nature, loess doesn't always fit the assumptions of the Terzaghi theory. Due to its high permeability loess seldom occurs saturated in the field, and in most instances where this would occur it is doubtful that loess would adequately support anything, except possibly minor structures, without excessive settlement. Clevenger (2) stated the following general opinions on consolidation of loess:

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

2. At low moisture contents (15% or less) natural loess will support the normally assigned loads for silty soil regardless of density. At high natural moisture (above 20%) the supporting capacity depends on the density.

3. At high natural moisture or if saturated:

    a. Low density loess (below 80 p.c.f.) settles excessively.

    b. Medium density loess (80 - 90 p.c.f.) varies in consolidation.

    c. High density loess (above 90 p.c.f.) does not settle excessively due to moisture and can be treated as ordinary silt.

4. A loess soil consolidates about the same whether it is pre-wetted or is wetted after loading.

Peck and Ireland (8) disagree with some of Clevenger's conclusions. They feel that there is no distinct correlation between structural strength and density, grain size, or penetration resistance for loess deposits throughout this country. They also have data showing that a grain elevator did not settle excessively although the natural moisture content was twenty-three percent, or well above the 15% maximum of Clevenger. Peck and Ireland also suggest that when loess is used as support for a foundation, the soil should be prevented from increasing its moisture content and that the design should then be based on the natural moisture content. If this is done, Terzaghi's theory of consolidation cannot be applied, although consolidation curves can still be of value. The consolidation curves can be related to final primary consolidation only, since time intervals do not apply. Peck and Ireland suggest a method to estimate the allowable soil pressure on loess; the break in the e-log p (void ratio <u>vs</u> logarithm of pressure) consolidation curve is taken as an ultimate load value, and a safety factor is applied based on judgment. They also suggest, as an alternate method, using the load causing a settlement of one-half inch for a one foot square loading plate.

Holtz and Gibbs (5) stated that two things cause loess to break down and consolidate: load and moisture. Under moderate or light loads, moisture is of great importance, while under heavy loads moisture is of less importance. They also noted that in some cases, loess of low density underwent considerable consolidation even though it was at a low natural moisture.

## C. Other Properties of Loess

Permeability is another distinctive property of loess. In general, permeability of low density loess is quite high and decreases in value as the density increases. Permeability of loess is also far greater in the vertical direction due to tubular rootlike holes which predominate in that direction (4). Terzaghi (10) is of the opinion that the permeability of loess can be accurately studied only by the use of air, since water would cause a breakdown of its structure.

Another outstanding property of loess is its very feeble resistance to erosion in both the natural and compacted state. Clevenger (2) feels that this can be minimized by using slopes as steep as possible ($\frac{1}{4}$ : 1 in many cases). Holtz and Gibbs (5) suggest slopes of one-fourth to one for heights up to thirty-five feet, one-half to one up to fifty-five feet, and three-fourths to one for higher slopes.

When loess is being used as a foundation material in embankment construction, three possible procedures have been suggested by the Bureau of Reclamation (4).

1. Partial or complete removal of loess in the foundation.
2. Saturation of the loess foundation soils by ponding or with well points prior to construction in order to induce the maximum settlement during construction.

3. Construction of the embankments with the materials at such a moisture content as to render them plastic, so they will rupture and conform to the foundation as settlement takes place.

Another construction procedure to improve the properties of loess is silt injection or grouting. The Corps of Engineers used this successfully in construction of a dam in Nebraska. A slurry of 95% loess and 5% bentonite was pumped under pressure into the loess. The purpose of this grouting was to reduce consolidation and prevent formation of cavities caused by differential consolidation. There is also the possibility of using some form of soil stabilization to improve in-place loess. So far the most successful method is based on injection or saturation with sodium silicate.

## III. SOILS

Five Wisconsin age loess samples from western Iowa were used in this investigation. The first three (49 B-1, 49 B-2, and 49 B-3) were taken at different depths from the same site. This location along with a fourth (85 B) and a fifth (97 B) lie approximately in a line parallel to what had been conjectured to be the prevailing winds at the time of deposition (6). Along this line, as the distance from the Missouri River floodplain increases, the density and clay content of loess increase, while the mean particle size and thickness of the deposit decrease. (Therefore, by taking samples along this line parallel to the prevailing winds, correlations, if they exist, with the above variables can be found.)

The first three samples, at location 49 B, were taken at different depths so that density (actually preconsolidation) variations could be studied independent of the other variables. Cut 49 B is located just east of U.S. Highway Alternate 30 in Harrison County adjacent to the Missouri River floodplain. It is a relocation of the type locality of the Loveland loess (3), which is an older pre-Wisconsin buried loess. Location 49 B-1 samples were taken at a depth of from 10 to 11 feet, 49 B-2 samples were from 76 to 77 feet, and 49 B-3 samples were from 134 to 135 feet. Their corresponding average densities of 74.3 pcf, 79.5 pcf, and 84.0 pcf show a large variation in density due to the increasing preconsolidation with depth. One peculiar fact was that sample 49 B-1 had a noticeably higher clay content than samples 49 B-2 or 49 B-3. This is contrary to theory and was not due to a soil profile development. Surface soil would be mapped in the Hamburg series, a Regosol with no zone of clay accumulation. Sample 49 B-3 is from Farmdale (early Wisconsin) loess separated from the overlying loess by a weak paleosol. Its composition is very similar to 49 B-2.

B.S. it
It's Loveland
Loe

Samples from 85 B, which were taken at a depth of from 10 to 11 feet, show higher clay contents and a higher density (78.5 pcf) than location 49 B for a comparable depth. This was expected since location 85 B is in the eastern part of Harrison County. The soil series at this location is the Ida.

Samples from location 97 B show still higher clay contents and a further increase in density, to 80 pcf. The increase in density would probably have been slightly larger if the samples could have been again taken from a depth of 10 to 11 feet, but due to the limited thickness of loess, the samples were from a depth of 7 to 8 feet. Location 97 B is in Cass County, and the soil series is the Marshall. Loess was not sampled farther east in western Iowa since it occurs only as thin deposits on the tops of hills.

Data on the loess soils can be found in Tables 1 and 2.

Table 1. Location of samples

| Sample no. | County | Section | Tier | Range | Soil series | Sampling depth, ft. | Horizon[a] |
|---|---|---|---|---|---|---|---|
| 49 B-1 | Harrison | 3 | 77 N | 44 W | Hamburg | 10 - 11 ft. | C |
| 49 B-2 | Harrison | 3 | 77 N | 44 W | Hamburg | 76 - 77 ft. | C |
| 49 B-3 | Harrison | 3 | 77 N | 44 W | Hamburg | 134 - 135 ft. | C |
| 85 B | Harrison | 29 | 78 N | 41 W | Ida | 10 - 11 ft. | C |
| 97 B | Cass | 13 | 77 N | 35 W | Marshall | 7 - 8 ft. | C |

[a]All samples were of calcareous, unleached loess

Loveland loess? [illegible] [illegible]

Table 2. Physical properties of soil

| Sample no. | 10-11 ft<br>49 B-1 | 76-77 ft<br>49 B-2 | 134-135 ft.<br>49 B-3 | 10-11 ft.<br>85 B | 7-8 ft.<br>97 B |
|---|---|---|---|---|---|
| Textural comp. % by weight[a] | | | | | |
| Sand | 1.0 | 1.1 | 1.3 | .6 | 1.6 |
| Silt | 78.7 | 84.5 | 83.7 | 73.5 | 68.4 |
| Clay 5$\mu$ | 20.3 | 14.4 | 15.0 | 25.9 | 30.0 |
| Clay 2$\mu$ | 16.2 | 12.3 | 13.2 | 21.3 | 25.0 |
| Colloidal 1$\mu$ | 14.0 | 11.3 | 12.2 | 19.3 | 23.1 |
| Textural classification[b] | Silty clay loam | Silty loam | Silty loam | Silty clay loam | Silty clay |
| Predominate clay Mineral from X-ray diffraction | Montmorillonite | | | | |
| Carbonates from D. T. A.[c] | 8.3% | 16.2% | 11.5% | 4.6% | 4.5% |
| Average density in pcf. | ~~64.3~~ 74.3 | 79.5 | 84.0 | 78.6 | 80.6 |

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

[b]Textural classification based on the Bureau of Public Roads system except that sand and silt sizes are separated by no. 200 sieve (0.074 mm.)

[c]Based on Ca $Co_3$

# IV. TESTING PROCEDURE

## A. Obtaining Test Specimens

In order to test truly undisturbed soil, special care must be taken in obtaining and preparing the soil samples. Cardboard containers 6 3/4 inches in diameter by 6 1/2 inches high were used to transport the soil samples from the field back to the laboratory. However, before being filled with samples, the cardboard containers and their covers were coated on the inside with paraffin wax to prevent moisture loss from the samples.

Vertical or near-vertical cuts were selected for sampling and the profiles were cleaned off and described. Sampling depths were chosen for samples to be from calcareous parent material. A small cave was then dug about two feet horizontally into the face of the cut just above the proposed depth of sampling. The samples were cut from the material at the rear of the cave to minimize alteration by freezing and thawing or contamination by material eroded down from above. Using the cave floor as the sample top, carving was then done downward to leave a pedestal cut to the inside dimensions of a cardboard container. The container was gently forced down over the pedestal, which was then cut off at the bottom, turned over, and trimmed flush. The cover was then marked to show the sample location, and placed on the container. After being returned to the laboratory, the containers and covers were sealed together with paraffin wax. The samples were then stored to await use.

When a test was to be run, one of the desired soil sample containers was cut open and the soil removed. Actual size test specimens were then carved from the large sample. Usually three or four could be obtained from one cardboard container. Test specimens to be used in the direct shear test were 2 1/2 inches in diameter by about 1 1/5 inches high. Consolidation test specimens were made 2 1/2 inches in diameter by 1 inch high. The test samples not for immediate use were stored in a new cardboard container or in a moisture room (95% relative humidity).

## B. Direct Shear Test

### 1. Apparatus

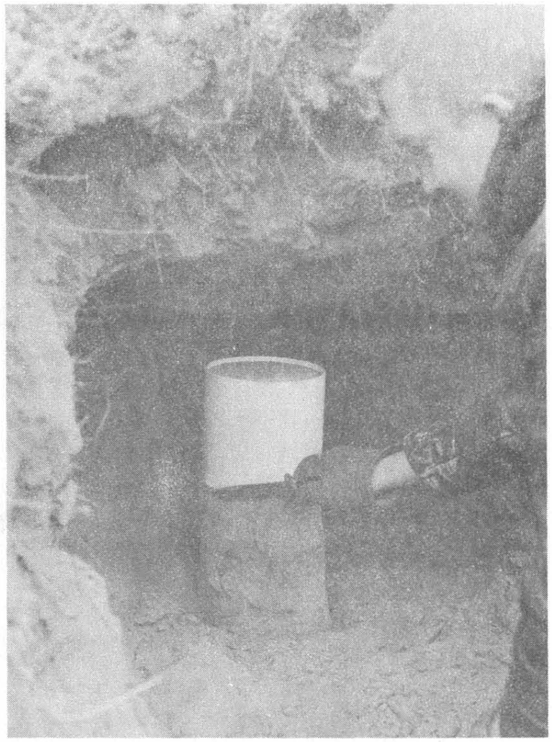
There are two methods for running a direct shear test:

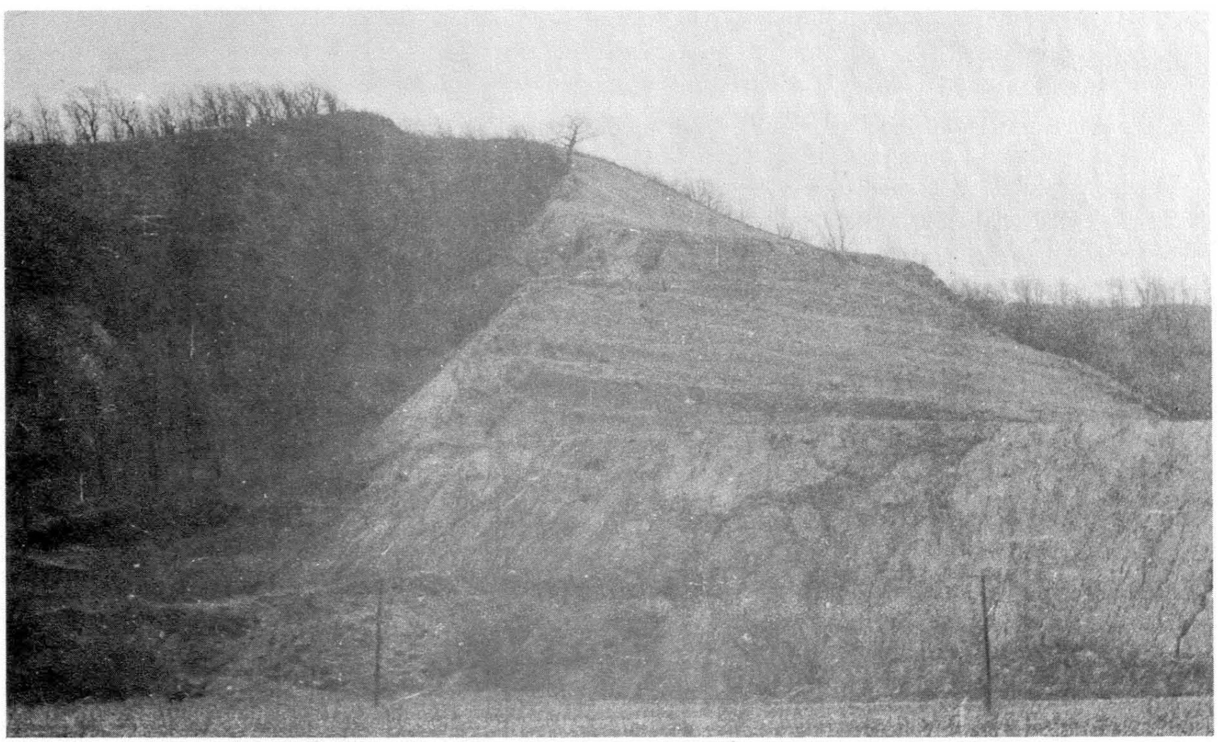
1. By increasing the shearing force at a given rate.
2. By increasing the shearing displacement at a given rate.

The first is called a stress-controlled unit, while the second is called a strain-controlled unit. The apparatus used here was of the strain-controlled type.

Figure 1. Final carving of the loess pedestal prior to forcing down the cardboard container

Figure 2. Location 49 B in Harrison County





In the apparatus used, an electric motor causes the lower half of the shear box to move horizontally outward, the rate of the displacement being controlled by a gear box and transmission. The upper half is held in place by a horizontal arm and yoke connected to a calibrated proving ring. An extensometer, in recording the proving rings deflections, gives dial readings from which the shearing resistance can be calculated. The actual horizontal displacements of the sample are measured by a second extensometer. The normal force is applied through a lever system the same as in the consolidation test.

2. Test

Three types of direct shear tests can be run in the laboratory:

1. A quick test where the specimen is neither allowed to consolidate or drain,
2. An intermediate test where the specimen is allowed to consolidate but can not drain, and
3. A slow test where the specimen can both consolidate and drain.

The actual time interval required for each of the above tests varies with the permeability of the soil tested. For a complete discussion of the above tests refer to W. Lambe (7).

A slow test was run for the data presented in this report. The test was performed with the following steps:

1. The sample was placed in the shear box.

*2. Water was added to the sample until it became saturated.

3. The normal load was applied.

4. A two hour interval was allowed so that the soil sample could consolidate.

**5. The shearing force was applied with a rate of displacement of 0.02 inches per minute.

6. Readings were taken every 15 seconds for the first 2 1/2 minutes, and then every 30 seconds until failure.

7. The sample was removed and its moisture content measured.

## C. Consolidation Test

### 1. Apparatus

There are two methods for loading consolidation specimens: by a jack loading device where the load is measured by

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*Step 2 was omitted when the sample was not tested in the saturated state.

**The rate of shearing displacement was faster than that normally used in the slow test but was believed to be satisfactory due to the high permeability of loess.

a platform scale, or by a lever system where the load is applied by hanging known weights. There are also two types of soil containers (7) in which the sample can be placed: a fixed-ring container where the samples movement relative to the container is downward, and a floating-ring container where compression occurs toward the middle of the sample, from both the top and bottom. The apparatus used is shown in Figure 3. It has a fixed-ring container and a lever loading system such that the actual load on the sample is ten times the weight of the load hung on the end of the lever arm.

2. Test

The test was performed in the following manner:

1. The sample was placed in the fixed-ring container with porous plates above and below it.

*2 a. Water was added to the soil sample in order to saturate it.

b. Slightly moist cotton was placed around the outside of the top porous plate.

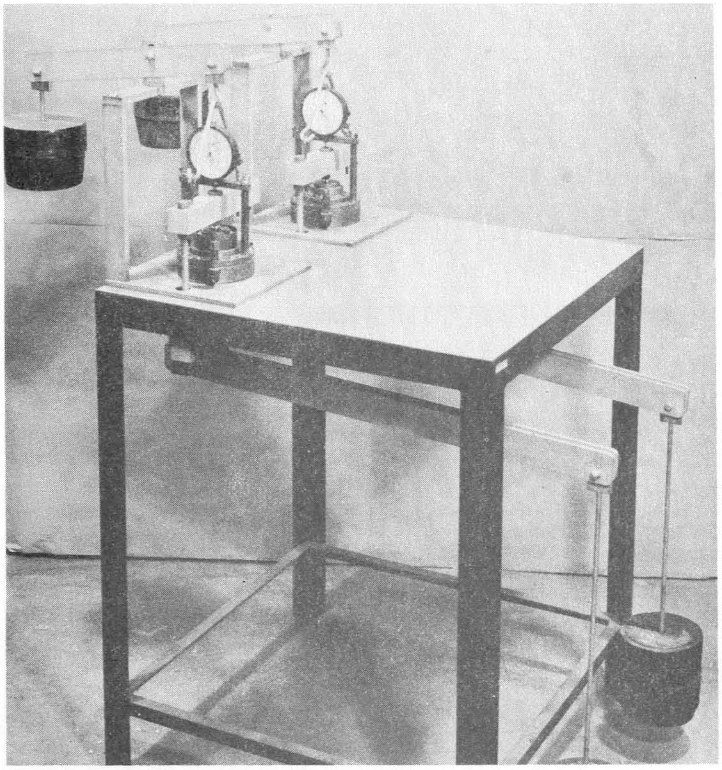
3. The initial dial reading was recorded and the first increment of load was applied.

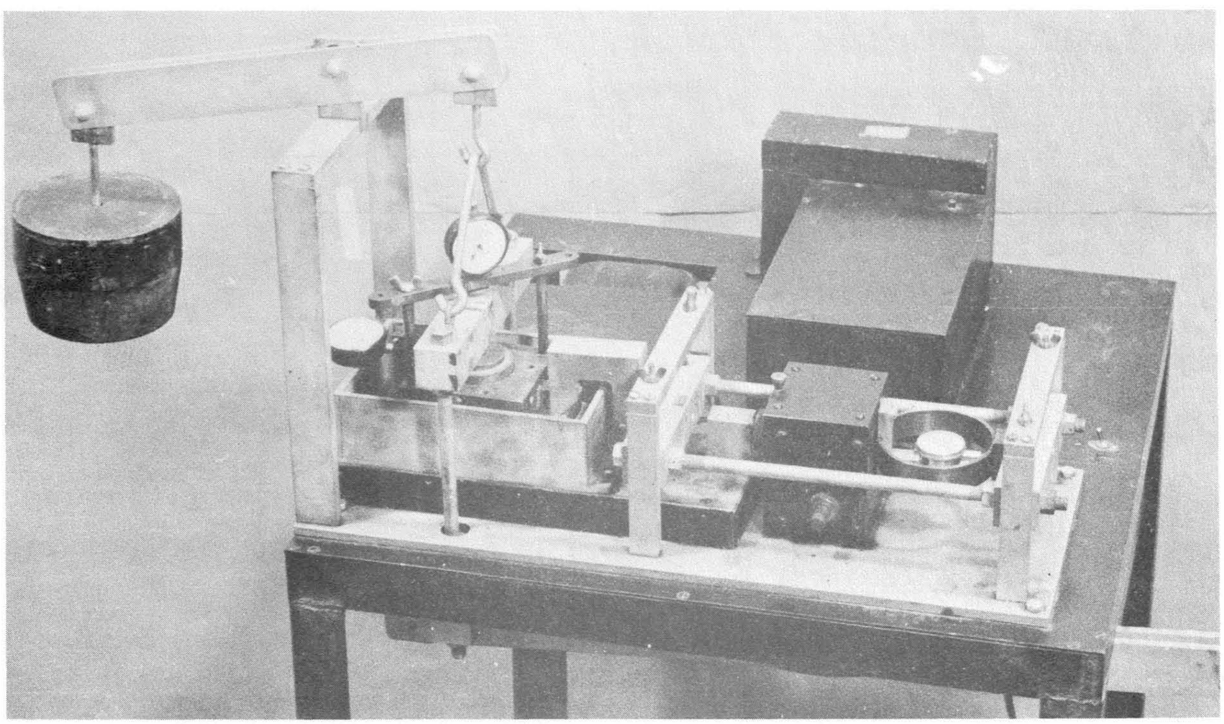
4. Dial readings were taken about every half hour to determine when the primary consolidation for that loading was nearing completion. When the rate of

*In testing saturated samples step 2b was omitted, while for samples tested at moisture contents below saturation step 2a was omitted.

Figure 3. Consolidation apparatus

Figure 4. Direct shear apparatus





consolidation became so slow that further consolidation would be negligible (less than about 0.0003 inches per hour), a final reading was recorded on the data sheet. The time required was two hours or less for light loads and from four to six hours for the heavy loads.

5. The next increment of load was added, and step 4 repeated. This was continued until the final increment of load was added. Table 3 gives the increments of loading.

6. The specimen was completely removed, placed in a container, weighed, and placed in the oven. The dry density and final moisture content were then determined.

Table 3. Load increments

| Weight on pan | Load on sample |
|---|---|
| 500 gms. | .161 tons/sq. ft. |
| 1,000 " | .323 " |
| 2,000 " | .646 " |
| 3,150 " | 1.015 " |
| 6,300 " | 2.030 " |
| 12,600 " | 4.060 " |
| 25,200 " | 8.120 " |
| 50,400 " | 16.240 " |

# V. RESULTS AND DISCUSSION ON SHEARING STRENGTH

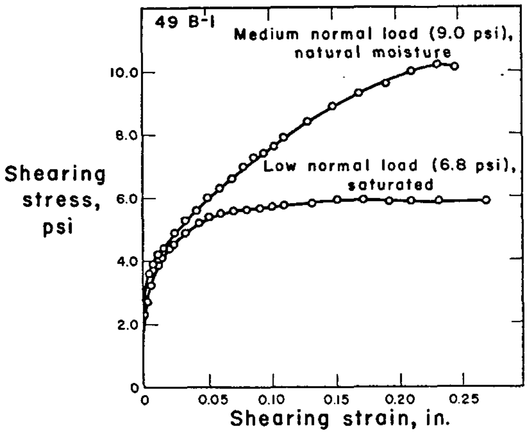
## A. Stress-Strain Relationship

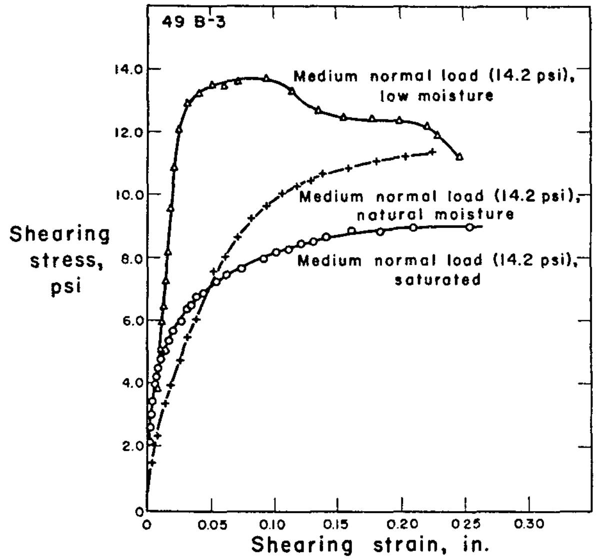
Graphs showing the stress-strain relationship for the five loess soils are shown in Figure 5. There are three types of stress-strain curves associated with western Iowa loess: (1) curves where the shearing stress continues to increase until failure, (2) curves where the shearing stress increases to a maximum and remains approximately at the same level until failure, and (3) curves where the shearing stress increases to a maximum peak and then decreases until failure.

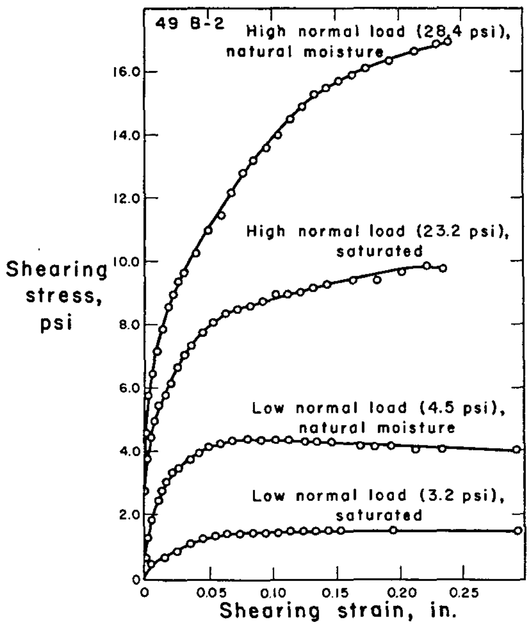
The first of these curves, where shearing stress increases until failure, occurs most often, and if the need arises to designate one curve as most typical, it would be this one. This curve is obtained from either saturated or natural moisture content loess that is tested at medium or high normal loads.

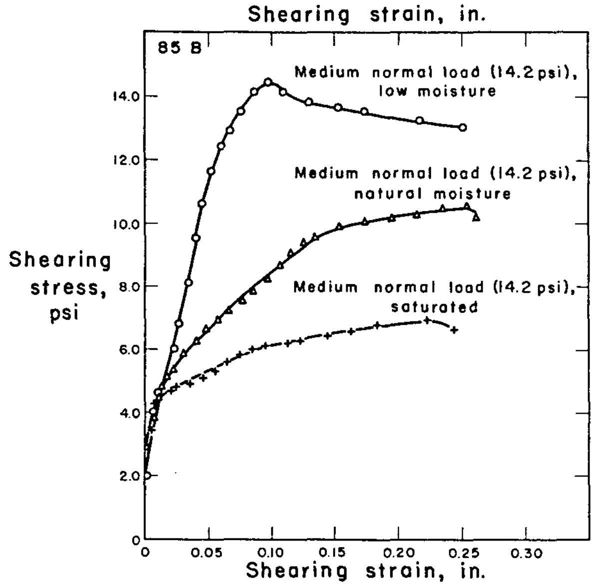
The second type of curve, where shearing stress levels off, occurs when the loess is tested with low normal loads. The third type of curve is found only with a certain moisture condition. Here samples are tested at such low moisture contents that there is a sharp increase in strength (as shown by samples from 49 B-3 and 85 B in Figure 7). In this type of curve shearing stress reaches a maximum and then decreases

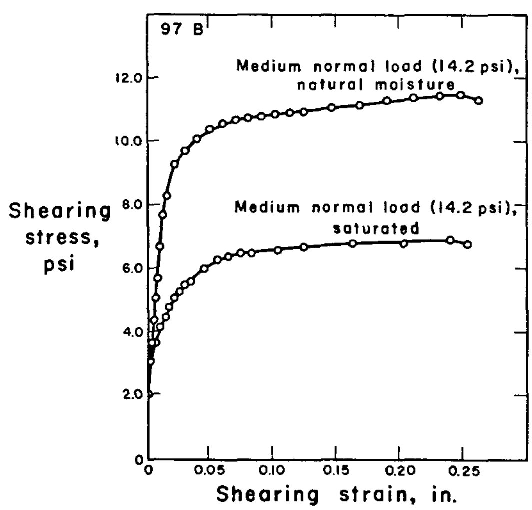
Figure 5. Typical stress-strain curves for western Iowa loess


49 B-1
Medium normal load (9.0 psi), natural moisture
Low normal load (6.8 psi), saturated
Shearing stress, psi
Shearing strain, in.


49 B-3
Medium normal load (14.2 psi), low moisture
Medium normal load (14.2 psi), natural moisture
Medium normal load (14.2 psi), saturated
Shearing stress, psi
Shearing strain, in.


49 B-2
High normal load (28.4 psi), natural moisture
High normal load (23.2 psi), saturated
Low normal load (4.5 psi), natural moisture
Low normal load (3.2 psi), saturated
Shearing stress, psi
Shearing strain, in.


85 B
Medium normal load (14.2 psi), low moisture
Medium normal load (14.2 psi), natural moisture
Medium normal load (14.2 psi), saturated
Shearing stress, psi
Shearing strain, in.


97 B
Medium normal load (14.2 psi), natural moisture
Medium normal load (14.2 psi), saturated
Shearing stress, psi
Shearing strain, in.

to an ultimate value before failure, and if the ultimate value rather than the maximum value is entered on the plot there is no sharp increase in strength. This decrease in shearing strength with increasing strain also occurs under certain conditions in other soils.

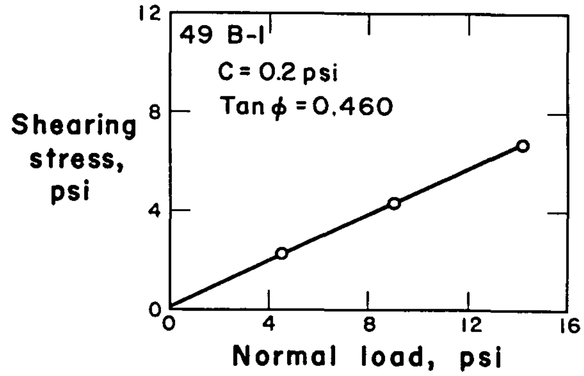
### B. Angle of Shearing Resistance

Shearing strength envelopes of the five saturated loess soils plus the shear envelope for 49 B-2 tested at 14.5% moisture are shown graphically in Figures 6 and 7.

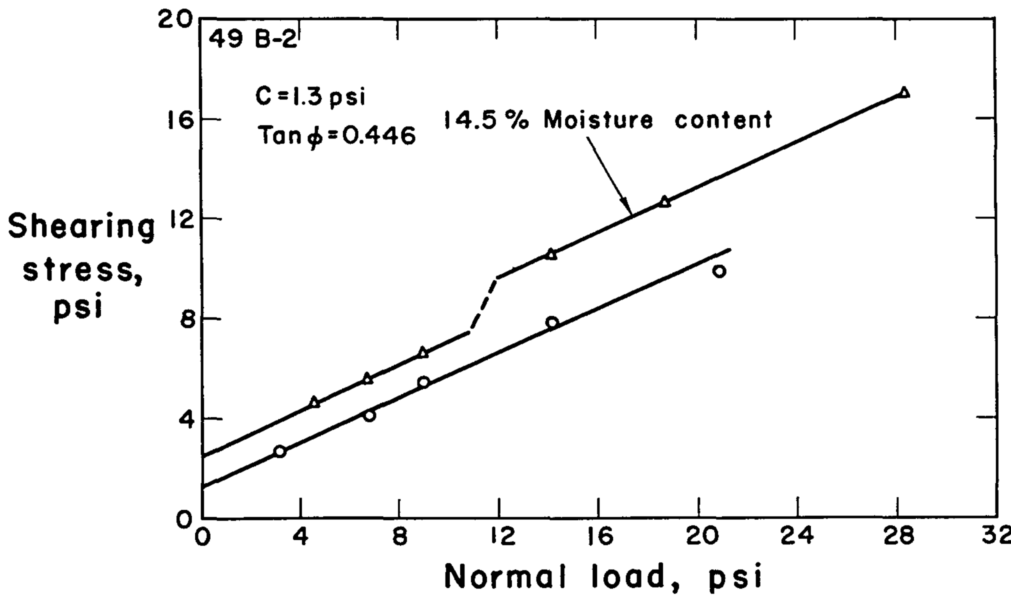
An important characteristic of these shear envelopes is the consistent value of tan $\phi$. Tan $\phi$ varies from 0.44 to 0.46, a variation which could be wholly experimental. This important strength property of the western Iowa loess is therefore independent of other such properties as density, clay content, and preconsolidation. It also appears, by examining the shear envelope for 49 B-2 tested at 14.5% moisture, that moisture content has no effect on the value of tan $\phi$. It should be pointed out however that the shear envelope for 49 B-3 changed slope under higher normal loads and gave a higher value of tan $\phi$.

Clevenger, in his report on the Bureau of Reclamations work with loess (2), also found that the shear envelopes for loess are parallel and thus give approximately equal values

Figure 6. Shear envelopes for samples 49 B-1, 49 B-2, and 49 B-3




$c = 0.0144$ T/ft²
$\phi = 24° 43'$
c = 1.4 kPa




$\phi = 24°$
z = 76

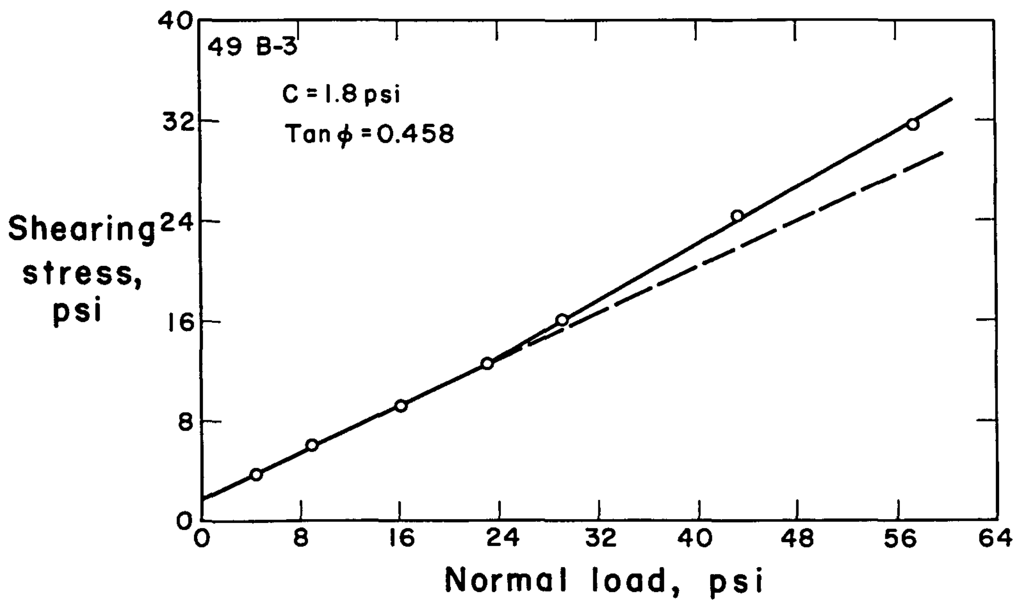
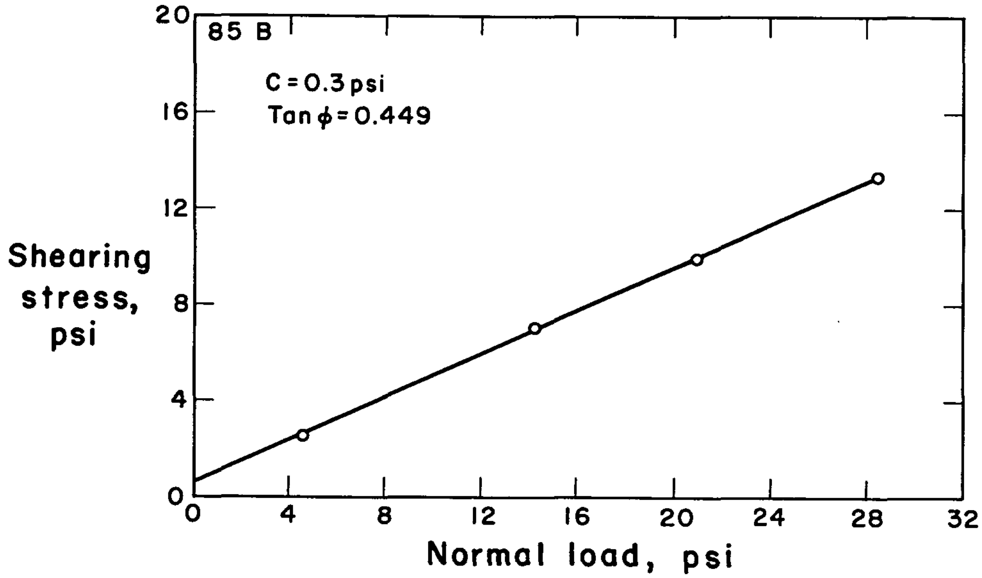
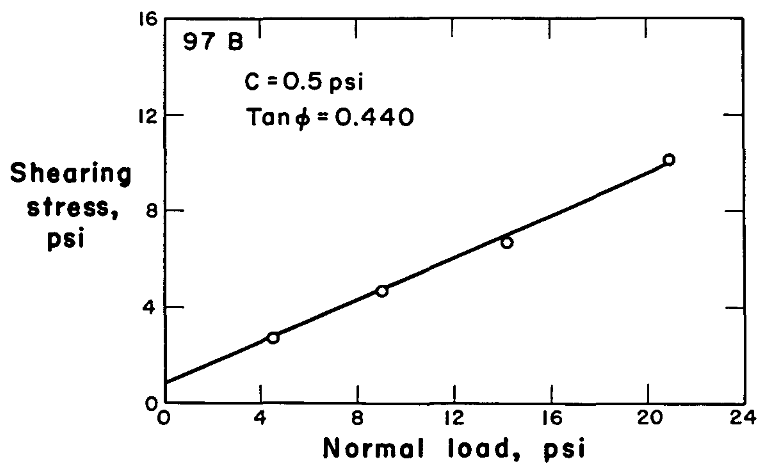
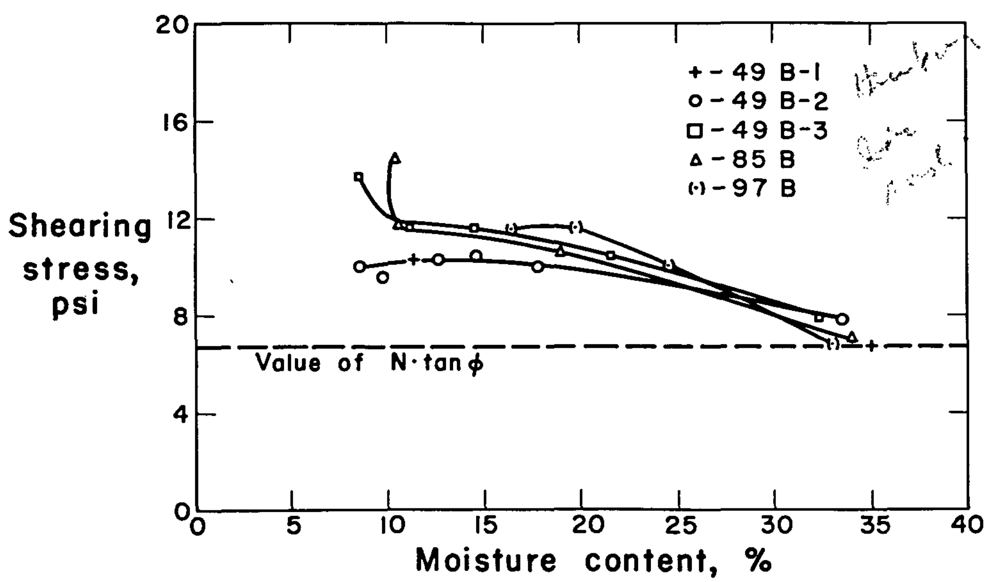


Figure 7. (a) and (b) Shear envelopes for samples 85 B and 97 B. (c) Moisture content *vs.* shearing stress curves for the five loess soils


85 B
C = 0.3 psi
Tan φ = 0.449
Shearing stress, psi
Normal load, psi
0 4 8 12 16 20 24 28 32
0 4 8 12 16 20


97 B
C = 0.5 psi
Tan φ = 0.440
Shearing stress, psi
Normal load, psi
0 4 8 12 16 20 24
0 4 8 12 16


+ - 49 B-1
o - 49 B-2
□ - 49 B-3
Δ - 85 B
(·) - 97 B
Value of N·tan φ
Shearing stress, psi
Moisture content, %
0 5 10 15 20 25 30 35 40
0 4 8 12 16 20

of tan $\phi$, but he stated that tan $\phi$ varies from 0.60 to 0.65. This discrepancy is probably due to one or possibly more of the following reasons:

1. Clevenger's data is based primarily on loess from central Nebraska which was derived from a different source than the loess in western Iowa. Therefore, there could be a physical difference in the loess.
2. Clevenger's data was based on triaxial tests, while these tests were by direct shear.
3. The rate of direct shearing displacement was too fast to allow drainage.
4. Clevenger measured his values of tan $\phi$ with higher normal loads, which correspond to the upper portion of the shear envelope of 49 B-3.

The last possibility seems the most probable. An article on the properties of loess at the Cambridge Canal in Nebraska, where tests were run with normal loads comparable to those used here, gave a value of 0.40 for tan $\phi$ (13). Also, the upper portion of the shear envelope for 49 B-3 gives a value of tan $\phi$ very nearly equal to the values found by Clevenger.

Clevenger also reported a lack of shearing strength for saturated low-density (below 80 pcf) loess if the normal load was less than 10 psi. As can be seen by the graphs, there was no appreciable deviation from the shear envelope for low density loess tested at low normal loads, even though some tests were run with normal loads as low as 4 psi.

The break in the slope of a shear envelope, as for sample 49 B-3, is usually believed to indicate the point where the normal load is equal to the natural field preconsolidation loading. For samples 49 B-3 the preconsolidation load is calculated as follows:

| | |
|---|---|
| Approximate average dry density in cut above 49 B-3 | = 80 pcf |
| Approximate highest average value of moisture content in cut | = 20% |
| Wet density | = 96 pcf |
| Height of cut above 49 B-3 | = 135 ft. |
| Preconsolidation load | = 96x135 |
| " " | = 12,960 psf |
| " " | = 90 psi |

The actual break occurred at a normal load of about 24 psi, which is equivalent to a depth of only 36 feet. This difference probably indicates that location 49 B-3 had never been saturated in the field. Since, as will be explained in the discussion on consolidation, high moisture contents greatly increase the consolidation of loess, it is thought that by not being saturated the loess at 49 B-3 was never preconsolidated to the potential afforded by the weight of the above material. In running the tests in the laboratory, however, the samples were saturated, and this allowed the full consolidation possible for the applied normal loads. This phenomenon and its possible application will be discussed later.

## C. Cohesion

Values of cohesion (referred to in this discussion as the value of shearing stress for a normal loading of zero) for these five loess soils bring out some important relationships in loess. Regardless of the soil tested, the saturated value of cohesion was very low, below 5 psi in each case. As the clay content increases, as shown by going from samples at 49 B-1 to 85 B to 97 B, there is only a slight and insignificant increase in cohesion. It therefore would appear that clay content has no effect on the cohesion of saturated loess of western Iowa. Along with this increase in clay content, there is a simultaneous increase in density as the distance of the loess from it's source increases. Obviously then, either this increase in density also has no effect on the values of cohesion, or the density change offsets the change due to clay content. The latter is believed unlikely since normally in soils an increase in density would cause an increase in shearing strength, and an increase in clay content would (in a preconsolidated soil) cause an increase in cohesion.

Since this did not occur, it is thought that most of the additional clay went into the voids in the loose structure of the loess as filler material and did not contribute to the binding of the loess structure.

However, density increases due to preconsolidation caused a more appreciable increase in cohesion. This effect is shown by samples from 49 B-1, 49 B-2, and 49 B-3. Values of cohesion increase from practically zero to 1.3 psi and finally to 1.8 psi as the depth of sampling increased to 135 feet. Actually, as was previously mentioned, the loess was only preconsolidated to a portion of its potential. Nevertheless the cohesion did show an increase. Had the loess been saturated in the field so that it could have been completely preconsolidated, the increase in cohesion probably would have been many times greater than it actually was.

Preconsolidation can then definitely be established as a factor influencing cohesion. The reason behind this increase in cohesion is related to the structure of the loess. It is generally accepted that thin clay coatings cover the loosely packed silt particles and give loess its bonding strength. In preconsolidating the loess, there is a partial breakup of the loess structure and the silt particles are forced closer together. This gives a larger surface area of contact between the particles and thus increases the force holding them together.

A series of samples from location 49 B-2 were tested at a moisture content of about 14.5%. The values of shearing stress in this case were considerably higher than for samples tested saturated. However, tan $\phi$ did not change in value, and the entire increase in shearing strength was due to an increase in cohesion. The shear envelope for these tests is plotted in two segments, each parallel to the other, with the break occurring at a normal load of about 11 or 12 psi. This break is related to the theory, discussed under consolidation, that load and/or moisture cause an initial breakdown of the loess structure. If the moisture content is high, the loess will immediately break down under a very small load, while if the moisture content is low this breakdown will occur at a higher loading. For 49 B-2 samples, which were tested saturated, the initial breakdown occurred at all values of normal loading. However, 49 B-2 samples tested at a 14.5% moisture content did not initially break down until the normal load reached about 12 psi. This load corresponds to the dotted portion of the shear envelope which is essentially an increase in cohesion. The cause of the sudden increase in cohesion is believed due to the sudden decrease in void ratio. This decrease in void ratio then causes an increase in cohesion similar to that caused by preconsolidation.

D. Moisture Content

Graph (c) in Figure 7 shows the increase in shearing strength with a decreasing moisture content. All five soils show a substantial increase in shearing strength as the moisture content decreases from saturation, or about 33% moisture, to about 20% moisture. The shearing strength then remains relatively constant while the moisture content decreases to about 10%, where, although not enough samples were tested to experimentally prove it for all five soils, there was a sharp increase in strength. Since tan $\phi$ remains constant, all these variations in shearing strength are due to cohesion, the value of which is shown by the horizontal line on the graph. As the soil decreases in moisture there is a thinning of the water films which in turn causes an increase in bonding. However, at very low moisture contents, ionic cohesion or coherence is primarily responsible for cohesion, since many of the water films are broken or not touching all points of contact (1). Therefore, it is apparent that cohesion should increase with decreases in moisture content.

Although all five soils increased in cohesion with drying, a greater increase was observed for the soils with higher clay contents. The sharp increase in strength at very low moisture contents also seems to vary with clay content, as it occurs at a higher moisture content for soils with high clay contents. Although the mechanism of this sharp increase is not known, it may be related to the moisture content at which ionic forces cause an increase in cohesion. The fact that the soils were initially wet and then dried produces maximum contact between particles, which in turn causes high ionic cohesion or coherence in the dried states.

### E. Design Values

Probably the most important factor to consider in estimating a design value for the shearing strength of western Iowa loess is that tan $\phi$ is a constant, a representative value for tan $\phi$ being 0.45. The plateau in the shearing strength <u>vs</u>. moisture content curve is another important factor. If, in designing a structure on loess, provisions can be made to prevent the loess from ever becoming saturated, or if the loess is so situated and of a high enough permeability that it will not become saturated, the plateau value of shearing strength is a reasonable design value. Since it is extremely doubtful if loess could be prevented from exceeding a 10% moisture content, the shearing strength values above the plateau are unrealistic design values and should not be used.

If the design value is to be based on a moisture content in the plateau range, one direct shear test would give enough information for estimating this value. By using the formula $S = C + N \tan \phi$, and substituting the value obtained in the test for S and using tan $\phi$ as 0.45, the value of C can be found.

The value of S could then be determined for any value of N. If the value of S is to be based on saturation, no testing is required because C is insignificant, an exception being if the soil was preconsolidated, where again one test would give the value of cohesion. The above statements apply only to the loess in western Iowa and where the normal loads are below the break in the shear envelope curve.

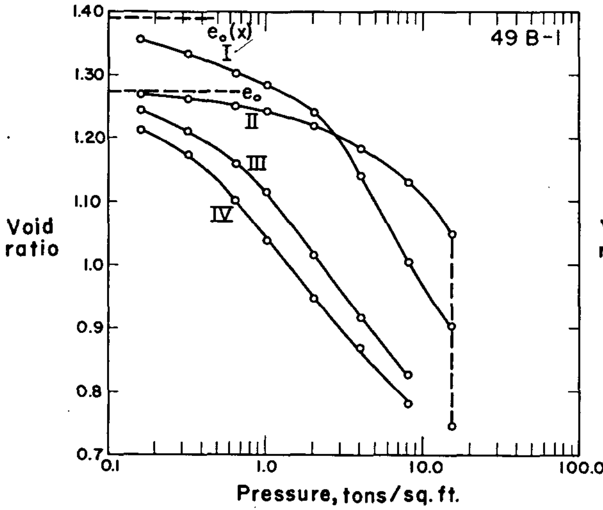
## VI. RESULTS AND DISCUSSION ON CONSOLIDATION

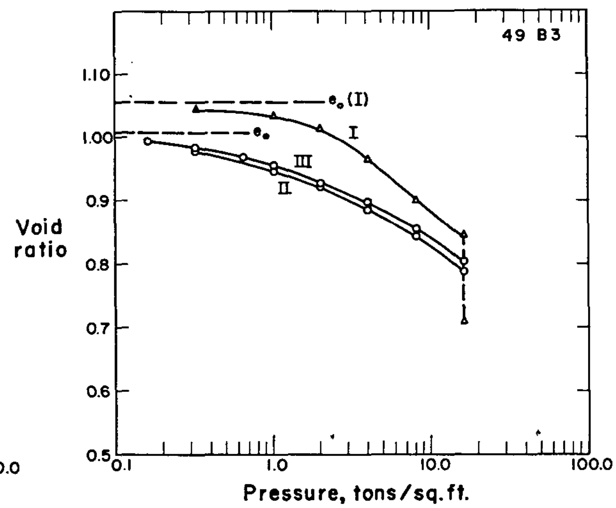
Plots of the soil void ratio *vs.* the log of consolidation pressure, commonly called e-log p curves, are shown in Figure 8. Some authors suggest the load at the break in the e-log p curve (the point of maximum curvature) as a value by which the consolidation resistance of different loess soils can be compared. This point was too obscure to accurately distinguish for the soils tested here. Instead, the loads corresponding to a 3% and a 5% reduction in volume were used. The volume reduction was based on the reduction of the total volume and for a 1 inch sample would then be 0.03 inches and 0.05 inches respectively. The load values corresponding to these volume reductions do not represent suggested design values but are used only as a means to compare the different loess samples. Actual design values should be based on the load corresponding to the break in the e-log p curves, if one exists, or the load corresponding to the maximum allowable settlement depending on the type of structure to be built. A safety factor should then be applied to this value. Table 4 gives the loads for the above volume changes along with the density and moisture content of the samples.

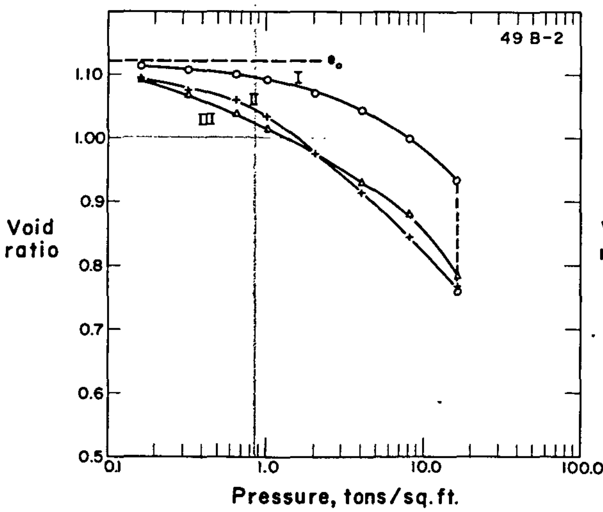
### A. Low Density Loess

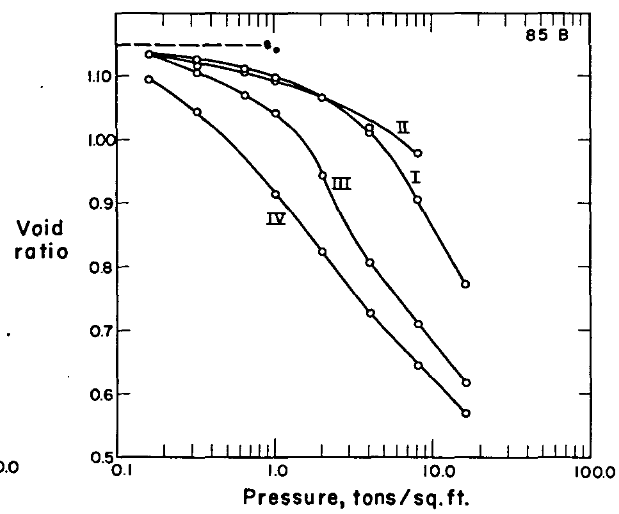
Location 49 B-1 gives a good general picture of low density loess. Three of the samples (II, III, IV) tested had a density

Figure 8. Void ratio *vs.* log pressure curves for the five loess soils


49 B-1
$e_o(x)$
$e_o$
I
II
III
IV
Void ratio
Pressure, tons/sq. ft.


49 B3
$e_o$ (I)
$e_o$
I
II
III
Void ratio
Pressure, tons/sq. ft.


49 B-2
$e_o$
I
II
III
Void ratio
Pressure, tons/sq. ft.


85 B
$e_o$
I
II
III
IV
Void ratio
Pressure, tons/sq. ft.

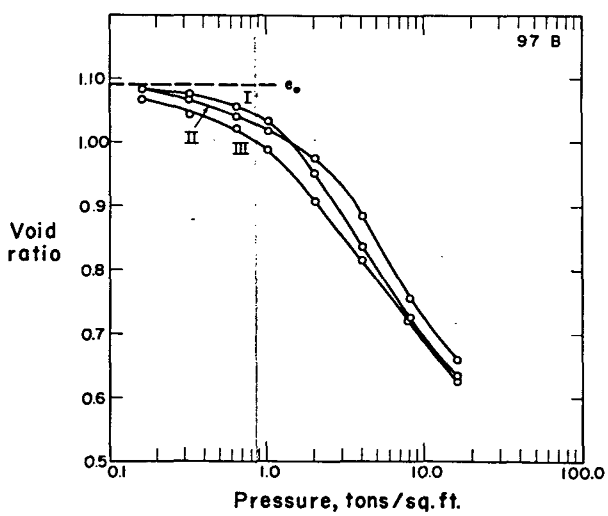

97 B
$e_o$
I
II
III
Void ratio
Pressure, tons/sq. ft.

Table 4. Consolidation test data

| Sample no. | | Actual density | Density used to figure e | Gravimetric Moisture content | Load for given volume reductions 3% | 5% |
|---|---|---|---|---|---|---|
| 49 B-1 | I | 70.5 p.c.f. | 70.5 p.c.f. | 4.9% | .49 T/sq.ft. | 1.42 T/sq.ft. |
| " | II | 74.4 " | 74.3 " | 6.1% | 2.85 " | 5.80 " |
| " | III | 74.3 " | 74.3 " | 14.6% | .36 " | .65 " |
| " | IV | 74.2 " | 74.3 " | x Saturated(33.5%) | .19 " | .38 " |
| 49 B-2 | I | 79.5 " | 79.5 " | 8.5% | 3.15 " | 6.5 " |
| " | II | 79.6 " | 79.5 " | 16.2% | .65 " | 1.27 " |
| " | III | 79.2 " | 79.5 " | x Saturated(29.5%) | .42 " | .98 " |
| 49 B-3 | I | 82.2 " | 82.2 " | 8.0% | 2.85 " | 4.65 " |
| " | II | 84.1 " | 84.0 " | 11.7% | 1.10 " | 2.85 " |
| " | III | 83.9 " | 84.0 " | x Saturated(30.1%) | 1.35 " | 3.25 " |
| 85 B | I | 78.5 " | 78.6 " | 10.3% | 1.025 " | 3.30 " |
| " | II | 78.9 " | 78.6 " | 11.5% | 1.40 " | 2.85 " |
| " | III | 78.6 " | 78.6 " | 24.8% | .47 " | .99 " |
| " | IV | 78.3 " | 78.6 " | x Saturated(32.5%) | .185 " | .33 " |
| 97 B | I | 80.7 " | 80.6 " | 21.1% | 1.01 " | 1.60 " |
| " | II | 80.6 " | 80.6 " | 22.8% | .90 " | 1.80 " |
| " | III | 80.5 " | 80.6 " | x Saturated(31.3%) | .58 " | 1.05 " |

x reported values of saturation too low (most only 90% saturated) of about 74.3 pcf, and their calculated void ratios are based on this average value. The other sample (I), for some unknown reason, had a density of only 70.5 pcf, and its e-log p curve was based on this value. This variation between the two densities is quite large and clearly shows the effect of density on consolidation. Although sample I was tested at an extremely low moisture content, its consolidation was quite high. The loads corresponding to a 3% and a 5% volume reduction were 0.44 tons/sq. ft. and 1.42 tons/sq. ft. Had reductions in these values been made (due to a higher natural moisture content in the field), and a safety factor applied, the allowable design load for this loess would have been almost zero. It seems plausible then that unless some form of stabilization such as silt injection or preconsolidation is used, loess with a density of about 70 pcf or lower is unsuited for supporting a structure without excessive settlement.

A second sample (II), from location 49 B-1 was tested at about the same moisture content, but having a higher density it had a much greater resistance to consolidation. The load values, at volume reductions of 3% and 5% are 2.85 tons/sq. ft. and 5.8 tons/sq. ft., or in each case more than 4 times the load values of sample I. However, when the moisture content is increased to 14.6%, a moisture content about or slightly above the moisture content that could be maintained as a maximum in the field, the strength is reduced to practically nothing. This soil exhibits a rapid decrease in consolidation resistance as the moisture content increases to about 15%. From here the resistance decreases only slightly until the sample reaches saturation. It therefore seems that although at very low moisture contents the soil is capable of sustaining loads without excessive settlement, it loses this strength too rapidly when wet to be of much use as a supporting medium for a large structure.

### B. Preconsolidation of Low Density Loess

Locations 49 B-1, 49 B-2, and 49 B-3 clearly show the effect of preconsolidation on the future consolidation of loess. However, due to the irregularities of the curves there is no clear point of maximum curvature or in some instances no straight portion of the curve, and it was impossible to locate the actual preconsolidation load with any degree of accuracy. Location 49 B-2 was substantially stronger than 49 B-1 regardless of the moisture content at which they were compared. At high moisture contents the corresponding loads were twice as high for 49 B-2, which had a density of 79.5 pcf compared to 74.3 pcf for 49 B-1. 49 B-2 sample III tested saturated, had approximately the same consolidation, less resistance at light loads and more resistance at heavy loads, as sample II which was tested at only 16.2% moisture. This means that moisture content has virtually no effect on consolidation of location 49 B-2 unless the moisture falls somewhere below 16%. There is also less total effect from moisture in location 49 B-2 than in that from 49 B-1 samples, which had no preconsolidation.

Location 49 B-3 shows the effect of further preconsolidation and in two of the samples tested (II and III) the density was increased to about 84.0 pcf, while another sample (I) had a density of only 82.1 pcf. Although the density had been increased, there was no significant change in the resistance of the lowest moisture content sample when compared to a 49 B-2 sample at the same moisture. However, as the moisture content increased, the resistance of 49 B-3 became twice that of 49 B-2. Also to be noted is that sample III of 49 B-3, which was tested saturated, had slightly higher load values than sample II which was tested at only 11.7% moisture. It becomes apparent then that moisture content loses its importance on load values except at very low moisture contents.

By examining values of all three locations at once, a trend is apparent. The increase in density from 70.5 pcf to 74.3 pcf shows an increase in strength for low moisture contents but as the density further increases to 84.5 pcf there is no further significant increase in resistance to consolidation.

However for high or even medium moisture contents, from saturation down to around 15% moisture, the load values quadruple as the density changes from 74.3 pcf to 84.5 pcf. The point above which moisture content is no longer important in affecting these load values also changes, decreasing down to below 11.7% for the highest density.

Both location 49 B-2 and 49 B-3 have a high enough resistance to consolidation to satisfactorily support a structure, although in the case of 49 B-2 there would be the necessity of preventing the moisture content from exceeding about 15%. For 49 B-3 the load values are high, even when saturated, and it would not be worth while to try to reduce the field moisture content unless it could be kept below 10%.

An interesting point concerning location 49 B is that the final void ratio of all the samples (49 B-1, 49 B-2, and 49 B-3) tested was about the same. This shows that the initial portion of the curve for the preconsolidated samples is actually a recompression curve and when the load exceeds the preconsolidation load the curves are similar. This corresponds to the theory of consolidation as related to preconsolidated clays.

C. Medium Density Loess

Location 85 B, with a density of about 78.5 pcf, shows some difference in consolidation values compared to 49 B-1.

Although low moisture load values corresponding to the 3% and 5% volume reductions are similar for the two soils, the effect of increasing the moisture content is a gradual loss in consolidation resistance as the moisture content increases to saturation, whereas in 49 B-1 the effect of moisture was almost fully realized by the time the moisture content reached 15%. However, by the time saturation is reached, the load values of 85 B and 49 B-1 are again similar. The effect of an increased density and clay content, related to distance from the loess source, is only to have the decrease in consolidation resistance more gradual as the moisture content increases and therefore have higher load values at the intermediate moisture contents (about 12% to 25% moisture).

Location 97 B, showing a further increase in density and clay content, was tested only at higher moisture contents. Since this soil's clay content is quite high, it is doubtful if this soil could exist in the field at low moisture contents. Location 97 B seems to show a general increase in resistance to consolidation throughout the moisture contents tested. This seems especially true for saturation, where its load values are three times those of 85 B.

Both location 97 B and 85 B have fair supporting strength if the moisture content is kept at values well below saturation. Therefore if provisions are made in the field to keep the soil from increasing its moisture content, these soils are, to some extent, capable of supporting a structure with light loads without excessive settlement. Since location 97 B gives the highest saturated load values of the three un-preconsolidated soils, the necessity of keeping its moisture content below saturation is not as important as for the other two, although it still is highly beneficial.

## D. Summary

By examining the e-log p curves for all five soils, a few trends become apparent and are worthy of discussion. One apparent fact is that practically all the samples tested showed an especially large drop in void ratio with the first increment of load. This immediate large decrease in volume was mentioned previously as an initial breakdown of the loess structure. Since as shown by the curves, the lower the moisture content the less the initial change, the amount of breakdown in structure is directly related to moisture content. However, for loess at low moisture contents the remaining breakdown in structure is thought to occur at higher loadings. The shear stress envelope for 49 B-2, tested at 14.6% moisture, had a break in the curve which may be explained by this theory.

Another moisture variable that could relate to the amount of consolidation is the effect of adding the moisture at different times in the loading curve. This effect was studied by comparing samples saturated throughout the test with samples wetted after the final increment of load. The additional final consolidation caused by the addition of water is shown as the vertical dash line below the final load increment. Test data clearly show that the consolidation is about the same as when the sample is continuously saturated. Sample I of 49 B-3 seems to disprove this, but it must be remembered that this sample had an initially lower density than sample II or III, and this was the cause of its higher consolidation.

Density also plays a significant part in the overall picture of loess. Although increased density failed to show any substantial effect on the e-log p curves for samples tested at low moisture contents, density did give a great increase in consolidation resistance for saturated samples. This additional resistance of saturated loess occurred regardless of whether the increase in density was due to preconsolidation or distance from the source area.

Due to the increase in strength of loess with preconsolidation, both in consolidation resistance and shear strength, it would seem that this would be an important method by which the structural properties of loess could be improved. Of particular importance is the fact that the full effect of preconsolidation is not taken in account unless the soil in the field has been saturated. If the loess will become saturated after construction is completed, as in a dam, consolidation could be greatly reduced by saturating the loess before construction began.

# VII. CONCLUSIONS

The following conclusions pertain only to the loess tested for this report: Wisconsin age loess from western Iowa. These conclusions may or may not apply to loess from other areas of this country.

1. The angle of shearing resistance for loess is a constant for normal loads below about 25 psi (4,100 lb./sq.ft) regardless of density, clay content, preconsolidation, or moisture content. The values are:

$$\phi = 24.2$$

$$\tan \phi = 0.45$$

2. Preconsolidation is the only variable that affects the value of cohesion of saturated loess. If there is little or no preconsolidation, cohesion is zero. Therefore for saturated loess that has had only insignificant preconsolidation, Coulomb's formula for shearing stress reduces to:

$$S = 0.45\ N$$

3. a. As the moisture content of loess decreases from saturation to about 20%, there is a substantial increase in cohesion. This increase in cohesion is somewhat higher for loess with higher clay contents.

   b. As the moisture content decreases from 20% to about 10%, cohesion remains relatively constant.

c. At a moisture content of about 10% there is a very sharp increase in cohesion that is probably due to ionic attractions between clay particles.

4. At low moisture contents (below 8%) loess generally has an average resistance to consolidation. In this moisture range there is no significant variation in resistance due to differences in density, degree of preconsolidation, or clay content. An exception is extremely low density loess (70 pcf and lower) which consolidates excessively regardless of moisture content.

5. There is a large decrease in the resistance of loess to consolidation as the moisture content increases. This is especially true for low density loess.

6. a. For loess samples with low clay contents there is no further reduction in resistance to consolidation above about 15% moisture.

   b. For loess samples with higher clay contents there is a continuous decrease in strength with increasing moisture contents until saturation is reached. Although this reduction in strength occurs over a larger range of moisture contents, the total reduction is probably about equal to or possibly less than that for loess with lower clay contents and the same density.

7. Although preconsolidation fails to significantly increase the resistance of loess to consolidation at low moisture contents, it substantially increases the resistance of saturated loess.
8. For saturated loess the higher the natural density the greater the resistance to consolidation. (This is thought to apply only when comparing loess from the same source. Therefore a loess with a density of 82 pcf from one source may have a greater resistance than one with a density of 85 pcf from another source.)
9. Most loess deposits in the field are not preconsolidated to the potential afforded by the weight of the above material, suggesting that they have never been completely saturated.

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