A CASE STUDY OF A SILO FOUNDATION FAILURE

by

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ABSTRACT

In May, 1978, Mr. Rene Menard had a 55' x 18' concrete stave tower silo built on his farm in St. Lazare. That fall, eight days after it was filled with corn silage, it tilted and assumed a very dangerous position. It would probably have fallen if he had not put supporting guy wires around the structure. The outside diameter of the footing is 18.2 feet, and rests on a fairly dense sand. However, just over 2.5 feet below the surface, is a saturated, sensitive clay-loam stratum over 100 feet thick. By performing accepted shear strength tests on this soil, and using the results in engineering equations, it was found that the footing was grossly undersized, and the silo was bound for failure the day it was built.

LIST OF FIGURES

TABLE OF CONTENTS

INTRODUCTION

The trend towards taller and larger tower silos on Canadian farms has increased significantly in the past several years. Often, in an effort to reduce the high cost of these structures, required soil investigations are not being made. This may lead to footings that are relatively cheap to build, but are designed with little or no sound engineering criteria. It is for this reason that on many weaker soils, the bearing capacity is approached or exceeded, often resulting in dangerous tilting and sometimes complete failure of the silo.

This is a case study of a silo foundation failure where the necessary size as well as the correct structural design was not implemented. In fact, from the results of this investigation, it is very evident that no accepted engineering design practices were used by the silo company in question when they constructed this footing.

In the future, unless specially trained agricultural or civil engineers perform the necessary soil strength tests prior to construction, we will be seeing increasing numbers of silo foundation failures. The extra initial cost for a well designed footing will be small when compared to the loss of life, property, and livestock that can occur with the complete failure of one of these massive structures.

Because most of the pertinent literature as well as present silo design criteria have not as yet switched to the metric system of units, British units will be used throughout the text of this project.

LITERATURE REVIEW

This section is a review of some of the important literature that was used in the completion of this project. Each pertinent topic will have its own heading, with the related work by the different authors summarized below each sub-title. Other references will be made throughout the text of this study where necessary.

General and Local Shear Failure with their Respective Bearing Capacity Equations:

Lambe states that the basic criterion governing the design of a foundation is that the settlement must not exceed some permissible value. In order to ensure that this basic criterion is met, an engineer must design the foundation so that the actual bearing stress is less than the bearing capacity of the soil, with an appropriate margin of safety to cover uncertainties in the estimate of both the bearing stress and bearing capacity. When this has been done, the longterm settlement should also be checked to see whether it will be below a permissible value. Generally, the bearing capacity is taken as the bearing stress causing local shear failure. The load that causes a general shear failure (the ultimate bearing capacity) is an upper limit for the bearing capacity. In this case, the full shear resistance is mobilized all along a failure surface which starts beneath the footing and extends to the surface of the soil beyond the footing. Local shear failure increases in importance as a soil becomes looser or softer.

Terzaghi proposed two different equations for determining the bearing capacity of a soil. One assumes that a general shear failure will occur, and should be used on a dense or stiff soil. The other assumes that local shear failure occurs, and should be used with loose or soft soils. General shear failure equation for a circular footing:

$$q_u = 1.2cN_c + \gamma dN_q + .6\gamma_b rN_\gamma \quad . . . . (1)$$

Local shear failure equation for a circular footing:

$$q_L = 1.2c'N_c' + \gamma dN_q' + .6\gamma_b rN_\gamma' \quad . . . . (2)$$

q_u = ultimate bearing capacity (psf)

q_L = bearing capacity assuming local shear failure (psf)

c = value of cohesion for particular soil (psf)

γ = total unit weight in contact with or adjacent to footing (pcf)

d = depth of footing (ft.)

γ_b = buoyant unit weight of soil in failure zone.(pcf)

r = radius of foundation (ft.)

c' = 2/3 c

N_c, N_q, N_γ, N_c', N_q', N_γ', are bearing capacity factors from fig.1 and are dependent on the internal angle of friction.

Bozozuk (1974) recommends that Skempton's ultimate bearing capacity equation with an adequate factor of safety should be used for the footings of tower silos on Canadian clays. Skempton's ultimate bearing capacity equation:

$$q_u = c\,N_c + \gamma d \quad . . . .(3)$$

q_u = ultimate bearing capacity (psf)

c = average shear strength of the soil to a depth below the footing equal to 2/3 of the outer diameter of the foundation (psf)

N_c = 6.6 = shape factor for a circular foundation

γ = total density of the soil in contact or adjacent to footing(pcf)

d = depth of footing (ft.)

An adequate factor of safety must be used to allow for strength anisotropy ,non-uniform pressure applied to the soil, overturning pressures due to high winds, large eccentric loads, etc. Therefore, Bozozuk suggests, in accordance with Terzaghi, Peck, and Skempton, that a minimum factor of safety of three should be used in bearing capacity determinations.

Skempton also proposed another equation for determining the ultimate bearing capacity of a clay with a shallow footing:

$$q_u = 6.2c(1 + .2\ d/D) \quad . . . . (4)$$

c = value of cohesion

d = depth of footing

D = diameter of foundation

According to Sowers, loose sands and highly sensitive clays fail by local shear when cracking of the soil develops around the foundation.

Undrained Shear:

Sowers states that if the internal angle of friction is zero, as for a saturated clay in undrained shear, the cohesion contributes most if not all of the bearing capacity.

According to Lambe, in clays, the permeability is generally so low that the foundation loading generates significant pore pressures. Thus the shear strength of an impermeable soil immediately following the placement of a foundation load is not the drained strength, but rather the undrained or partially drained strength. Except in heavily overconsolidated clays, the bearing capacity for undrained loading is less than that for drained loading, and thus controls the foundation design.

Terzaghi states that in connection with soils of such low permeabilities as those possessed by most clays and silts, there are many practical problems in which we can assume that the water content of the soil does not change for an appreciable time after the application of a stress. That is, undrained conditions prevail. Moreover, if a sample is extracted at the same water content and is tested without allowing change in water content, the strength of the soil with respect to total stresses will be approximately the value of c determined from these tests, with ϕ equalling zero.

Sensitivity of Soils:

Lambe states that the ratio of undisturbed to remoulded strength is defined as sensitivity. Sensitivity is related to the liquidity index, since the greatest loss of strength should occur in a highly flocculated soil whose water content is large compared to its liquid limit.

According to Terzaghi, if a natural sediment is thoroughly kneaded or remoulded, the flocs are largely disrupted and many of the clay particles become oriented in nearly parallel arrays. As a consequence, the shearing resistance may be substantially decreased. The clay is therefore said to be sensitive to disturbance. Certain marine clays in eastern Canada are characterized by high sensitivity. The remoulded strengths of some saturated clays are so low that an unconfined specimen cannot stand under its own weight. If the water content of a natural soil stratum is greater than the liquid limit, remoulding transforms the soil into a thick viscous slurry.

Sowers says that deposits of soil formed in the sea, which is a strong electrolyte, are frequently highly flocculent. If an undisturbed flocculent soil is thoroughly mixed without the addition of water, it becomes soft and sticky as though water had been added to it. In fact, water has been added, for the bond between the particles has been destroyed so that the free water trapped between them has been released to add to the adsorbed layers at the former points of contact. This softening upon remoulding is termed sensitivity. The water content of most highly sensitive clays exceeds their liquid limit which aids in their recognition.

Silo Foundation Design:

J.E. Turnbull and H.E. Bellman (1977) have presented a series of equations to aid in the design of reinforced extended ring foundations for tower silos. These are presented below:

$$F = \frac{Wh}{KD}\left[1 - \frac{h}{3KD}\right] \quad \ldots\ldots\ldots\ldots (5)$$

$$B^2 + B\,(D - 2) - \frac{D(F + S)}{P - 150d} = 0 \quad \ldots\ldots (6)$$

$$e = (2/3)B\left[\frac{3/2 + B/(D/2 - 1)}{2 + B/(D/2 - 1)}\right] - 1 \quad \ldots\ldots\ldots (7)$$

$$A_s = \frac{De\,(F + S)}{41{,}136(d-.33)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$(d - .33)^2 - \frac{De\,(F + S)}{53{,}312B} = 0 \quad \ldots\ldots\ldots\ldots\ldots (9)$$

$$A_L = \frac{d\,(D - 2)}{2(24{,}000)}\left[100 + 1.92h\,(D - 2)^{.55}\right] \ldots\ldots\ldots (10)$$

F = total sileage-to-wall friction load on a unit of wall circumference (lbs./ft. of circumference)

W = total weight of sileage per unit of wall circumference (lbs./ft. of circumference)

h = height of silo (ft.)

K - a constant = 4.72

D = diameter of the silo (ft.)

B = required footing width (ft.)

S = weight of silo, roof, and accessories per foot of circumference (lbs./ft. of circumference)

P = allowable bearing capacity (psf)

d = thickness of footing (ft.)

e = eccentricity of the soil reaction with respect to the wall loads

A_S = required spiral steel area (in.^{2})

A_L = required extra spiral steel area to resist lateral pressure (in.^{2})

With the above equations, we can design a suitable footing based on the wall loads. However, the size of this footing may not satisfy another fundamental requirement put forth by Turnbull, that the total soil area under the footing plus floor must be sufficient to support the total weight of the silo, foundation, and contents. With weaker soils and taller silos, this requirement tends to apply, and the important dimension is the outside footing diameter.

OBJECTIVES

Below is a list of the author's main objectives for this study.

1) To retrieve samples from the field which were as undisturbed as possible.

2) To use these samples to perform laboratory strength tests that would simulate closely the actual conditions that would occur in the field.

3) To use the results from these tests in well known equations to determine values for the bearing capacity of this soil.
4) To find the actual pressure exerted by the silo, and compare it to the bearing capacity values found experimentally.
5) To design a suitable foundation using accepted engineering procedures, and compare it to the existing foundation.

GENERAL INFORMATION

This section will give some of the pertinent information regarding the silo in question. Most of this information came from conversations with Mr. Rene Menard, the owner of the silo.

Location of Mr. Menard's farm and silo: Shown on the map in fig.2
Month and year silo was built: May, 1978
When did it reach its present position(fig.3)? At harvest time, 1978, eight days after it was filled with corn silage.
Moisture content of the silage: 70% to 75%
Company that built the silo: Superior Silo Quebec
Height and Diameter of silo: 55ft. x 18ft.
Dimensions of existing footing: Shown in fig.4
Thickness of concrete staves: Two inches
Was the silage evenly distributed? Yes, according to the owner.
Type of tests performed by the silo company on soil: Some type of penetrometer test to a depth of about 20 inches, therefore probably never reaching the weaker stratum below.
Why has the silo not fallen or the condition worsened? The silo is presently being supported by guy wires which are held by thirty foot piles embedded in the surrounding fields. Also, cement blocks were placed on the side opposite to the direction of failure so as to counterbalance the falling weight of the silo.
Was a bulge of the earth noticed near the silo, which would be an

indication that a general shear failure complete with a slip circle had occurred?

No bulge was noticed, but rather cracking of the soil occurred up to about eight feet from the silo foundation.

SOIL PROFILE AND SAMPLING METHODS

The soil profile that was found when digging during sampling is shown in fig.5. It consisted of about nine inches of organic matter at the surface, approximately 2.5 feet of sand below this, then a saturated clay-loam layer.* The water table was at about three feet. No mottling at all was noticed in the clay-loam layer, where as significant amounts of mottling was noticed in the upper sand layers. This gave evidence that the clay-loam layer was almost always saturated. As we got deeper the difficulty of digging increased, because of the water seeping into the hole and the extreme stickiness of the soil. Also, we could not step on the soil where samples were to be taken because of the large disturbance that would occur. Therefore, samples were taken at a depth of approximately 4.5 feet. Cores that were made of four inch outer diameter irrigation tubing cut into four inch lengths were used for sampling. The edges were smoothed, and petroleum jelly was applied to the inside of the cores to reduce the friction during penetration. Ten of these cores were pushed into the soil with the least amount of disturbance possible. They were then dug up with a shovel, the edges trimmed and were immediately placed into plastic bags to prevent moisture loss. The ten cores were then put into large clamps for transport. (fig.6)

* The owner of this silo states that this soft clay-loam layer is over 100 feet thick.

TYPE OF SHEAR TESTS USED FOR DETERMINATION OF c-ϕ VALUES AND REASONS FOR THE CHOICE:

Arpad Kezdi states that in order to obtain appropriate strength values experimentally using any testing method, we must produce a state of stress causing failure under test conditions that simulate as close as possible the actual conditions under which the soil is stressed.

P. L. Capper and W. F. Cassie say that for soils other than sands, the choice of test conditions depends on the purpose for which the shear strength is required. The guiding principle is the drainage condition of the test should conform as closely as possible to the conditions under which the soils will be stressed in practice.

In all probability no significant drainage or dissipation of pore pressure occurred in the soil under the silo after the total stress was applied or before failure occurred due to the following reasons:

1) The relatively quick time for the load to be applied (time required to harvest corn) along with the short time for failure to occur (eight days).
2) Because the clay-loam layer was saturated at the time of failure, and is probably always saturated very close to the bottom of the footing.
3) The high percentage of fine particles in the clay-loam layer (70% silt and clay) presents a relatively impermeable barrier to the flow of water.

Therefore, I decided, in consultation with the literature on this subject, that the test that would best simulate the actual soil conditions would be an unconsolidated-undrained shear test. This type of test should give ϕ values of approximately zero degrees, and the cohesion c should contribute most to the bearing capacity. In the next sections, the results of these tests will be presented.

UNCONSOLIDATED-UNDRAINED SHEAR TESTS

Direct Shear Tests

The direct shear box as shown in fig.7 was used to perform these tests. From each core we were able to get five relatively undisturbed samples. Five cores were used, thus giving us 25 samples. Tests were also made on completely remoulded samples with no change in water content so as to give us an idea of the sensitivity of the soil.

Procedure: The soil was gently pressed out of the core and a slab of suitable thickness was cut off using a fine jig-saw . Out of this slab a sample was cut using a mold that is supplied with the shear box. The specimen was placed in the shear box and normal loads of between zero and 35 lbs. were applied. The displacement was started quickly after the addition of the normal load to reduce the time for drainage. The rate of displacement was appoximately .06 inches/minute, a rate suitable for undrained tests according to Lambe. At 15 second intervals the displacement and the force reading from the proving ring were read simultaneously. The test was stopped when the readings from the ring stabilized. The average of all the maximum shear stress values (Tmax) at each normal pressure was found, andthe results were plotted against their specific normal pressure (P) . This was done for both undisturbed and remoulded samples, and the best straight line using linear regression was drawn through the points (fig.8). Sample calculations are given in Appendix A, and the results are shown on the next page.

Table 1: Direct Shear Test Results for Undisturbed Samples

CORE # ⟶	1	2	3	4	5	Mean
	Tmax (psi)	Tmax (psi)	Tmax (psi)	Tmax (psi)	Tmax (psi)	Tmax (psi)
P=0.0 psi	1.500	.641	1.169	.773	1.170	1.051
P-2.204 psi	1.830	1.566	2.160	2.094	2.624	2.055
P=4.408 psi	3.547	3.285	2.556	3.449	3.053	3.178
P=6.612 psi	4.010	3.880	4.934	4.539	4.342	4.341
P-8.813 psi	5.529	4.871	6.123	5.265	5.861	5.530

c = .982 psi

ϕ = 27.03 degrees

correlation = .9995

Table 2: Direct Shear Test Results for Remoulded Samples

CORE # ⟶	3	4	5	Random Sample	Random Sample	Mean
	Tmax (psi)	Tmax (psi)	Tmax (psi)	Tmax (psi)	Tmax (psi)	Tmax (psi)
P= 0.0 psi	.234	.376	.311	.278	.377	.315
P-2.204 psi	.840	.641	.575	.674	.707	.687
P=3.306 psi			.641	.773	1.170	.861
P-4.408 psi	.840	.972	.575	.839	1.269	.899
P=5.510 psi			1.037	1.038	1.140	1.072
P=6.612 psi				1.236	1.401	1.319

c = .334 psi

ϕ = 8.18 degrees

correlation = .987

Unconsolidated-Undrained Triaxial Shear Tests

The triaxial shear tests were made using the triaxial apparatus as shown in fig.9. From each core, three relatively undisturbed samples were obtained, except from core #6, where one sample was accidentally destroyed. The dimensions of the samples were approximately equal to three inches in height and 1.5 inches in diameter, the ratio between them (2:1) as recommended by Lambe.

<u>Procedure</u>: Samples were cut out of each core and were placed in an impermeable membrane. They were then put into the triaxial apparatus, and confining pressures of 10, 20, or 40 psi were introduced. At equal intervals of the displacement gage a force reading from the proving ring was taken. The rate of displacement was set at about .001 inches/second. No drainage was allowed during any tests to simulate as closely as possible the probable in situ conditions. Sample calculations are given in App. A, and the results will be shown below.

TABLE 3: Results of Triaxial Tests

CORE # →	6	7	8	9	10	Mean
$P_3=10$	$P_1=?$	$P_1=17.24$	$P_1=15.69$	$P_1=18.25$	$P_1=13.27$	$P_1=16.11$
$P_3=20$	$P_1=35.43$	$P_1=26.03$	$P_1=26.76$	$P_1=28.19$	$P_1=25.77$	$P_1=28.44$
$P_3=40$	$P_1=52.22$	$P_1=47.64$	$P_1=51.53$	$P_1=49.82$	$P_1=48.74$	$P_1=49.99$

(N.B.- All of the above values are given in pounds per square inch)

$c = 2.43$ psi $=$ 350 psf

$\phi = 3.873$ degrees

Discussion of Results of the Triaxial and Direct Shear Tests:

As stated earlier, unconsolidated-undrained tests were decided upon as the test that would best simulate our actual soil conditions. However, drainage and consolidation was very difficult to control in the direct shear test. Although the test results correlate very well, because of the small size of the samples significant increases in drainage and consolidation were noticed during testing as heavier normal loads were applied. Pore pressures were dissipated and effective stresses increased, thus increasing the apparent strength. It is for this reason that a friction angle of 27 degrees rather than the expected ϕ of zero degrees was found. The results from the direct shear tests would have been valid for the case where significant drainage and dissippation of pore pressures could have occurred while the load was being applied. This is not the case in our situation however, so bearing capacity values using ϕ = 27 degrees and c = .982 psi would be higher than what we could actually expect with our particular loading conditions. Therefore, these values will not be used in the determination of the bearing capacity of this soil.

On the other hand, drainage was very easy to control in the triaxial apparatus. Thus, the value of c determined from these tests should give us a fairly good indication of the shear strength that we could actually expect. The value of ϕ turned out to be 3.9 degrees which is close to 0 as expected. The reason it didn't turn out to be exactly zero is probably due to the slight moisture loss and subsequent air entrapment during storage. As the normal pressure increased, the air compressed, thus increasing the strength slightly. In subsequent bearing capacity calculations, however, we will assume that ϕ = 0 conditions prevail, and the value of c = 2.43 psi from the results of the triaxial tests will be used.

PARTICLE SIZE ANALYSIS

The procedure used for determining the percentage of sand, silt, and clay is as outlined in reference 9. It requires the use of a "Bouyoucos Soil Hydrometer". First, 50 grams of the soil was separated into its individual particles. These particles were then thoroughly mixed with 1000 cc's of water. Hydrometer readings were taken 40 seconds and 2 hours after the end of mixing, because all the sand sized particles will have settled out of solution after 40 seconds, and all the silt will have settled after 2 hours, leaving only the clay in solution. Using the calculations in App.A, we find that:

Sand = 30%

Silt = 39%

Clay = 31%

From the soil textural triangle, we find that this soil is classified as a "Clay-Loam".

LIQUID LIMIT DETERMINATION

The liquid limit is defined as the moisture content at which the soil has such a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. The procedure used is outlined by Lambe in reference 7. The liquid limit device and grooving tool used were the standard tools used by most engineers in determining this value. Samples of soil at different moisture contents were tested, and the number of blows required to close a specified length of groove was recorded. The moisture content of the soil sample whose groove closed at 25 blows is the liquid limit. From the calculations in App.A, we find that:

Liquid limit = 25.67%

Terzaghi states that the liquid limit of a sensitive clay is about 26%. Our value, therefore, correlates very well with that found by Terzaghi.

MOISTURE CONTENTS AND TOTAL DENSITIES

The test results of moisture content and total density determinations will be presented in this section.

Table 4: Moisture Content of Cores 1 Through 5

CORE #	1	2	3	4	5
MOISTURE CONTENT %	30.56	32.29	31.87	33.24	32.42
MEAN = 32.08 %					

Table 5: Moisture Content of Cores 6 Through 10

CORE #	6	7	8	9	10
MOISTURE CONTENT %	27.63	27.89	27.33	28.70	28.78
MEAN = 28.07 %					

The triaxial shear tests, which used cores 6 to 10, were performed at a later time than the direct shear tests. From the results above, we notice that there was a small moisture loss during storage in these cores.

The liquidity index equals the in situ moisture content divided by the liquid limit.

Maximum liquidity index = 32.08/25.67 = 1.25

Minimum liquidity index = 28.07/25.67 = 1.09

Table 6: Total Densities

CORE #	2	3	7	9	10
TOTAL DENSITY pcf	119.78	122.00	123.12	121.41	118.95
MEAN = 121.05 pcf					

FACTORS INDICATING THAT THIS SOIL IS FAIRLY SENSITIVE

Some of the criteria that often point to the fact that a soil is sensitive were explained earlier. This section will state some of the factors indicating sensitivity that were evident in this particular soil.

1) In situ moisture content is greater than its liquid limit.
2) Liquid limit value correlates very closely to Terzaghi's value for a sensitive clay.
3) Soil was deposited under salt-water conditions by the Champlain Sea, and most probably has a flocculated structure in its undisturbed state.
4) The soil was never heavily overconsolidated.
5) Upon remoulding, the soil becomes very soft and sticky, and it seems that water has been added.
6) From the results of the direct shear tests , a large reduction in the value of c(.98psi to .33psi) and in the value of ϕ(27 degrees to 8 degrees) occurred upon remolding.
7) An unconfined sample will not stand under its own weight for testing.

From the above information, we can safely conclude that our soil is at least moderately sensitive. Sensitive soils often present serious problems when designing and building foundations. Usually the bearing capacity will be based on the undisturbed strength. However, if careless construction or severe vibrations changes the soil into its remoulded state, the design might not be suitable at all due to the great loss in strength. Also, local shear failure increases in importance as soils become softer. Therefore, care must be taken when building large, heavy structures on this type of soil.

DETERMINATION OF ACTUAL PRESSURE EXERTED BY THE SILO ON THE SOIL

From fig.10 we find that the average capacity for a 55 ft. x 18 ft. tower silo is 355 tons. This capacity applies to corn or grass silage in a range of 68% to 72% moisture content. The silage in this particular silo was at 70-75% moisture content. However, 355 tons will be used as a close approximation of the silo's capacity.

355 tons $\doteq$ 710,000 lbs.

Assume that the unit weight of the concrete staves plus the steel reinforcement is 150 pcf. Therefore, the total weight of concrete

$$\doteq (18)\ (3.14)\ (55)\ (2/12)\ (150) \doteq 77{,}754 \text{ lbs.}$$

(N.B.- The thickness of each stave is 2 inches $\doteq$ 2/12 feet.)

Assume that the weight of the roof and the required silo accessories is 3000 lbs.

The footing is a concrete ring (fig.4) with a total weight equal to

$$(3.14/4)\ (18.17^2 - 7.17^2)\ (2)\ (150) = 65{,}676 \text{ lbs.}$$

The total of silage, silo, roof, accessories, and footing equals

710,000 + 77,754 + 3000 + 65,676 = <u>856,430 lbs</u>.

Distribution of Weight:

From Equ.5, we can determine the approximate total silage-to-wall friction load by replacing W by the total silage weight rather than the weight per unit of wall circumference. Therefore, the total friction load on the walls equals

$$\frac{(710{,}000)\ (55)}{(4.72)\ (18)} \left[1- \frac{55}{(3)\ (4.72)\ (18)} \right]$$

$$= 360{,}446 \text{ lbs.}$$

However, because the ring is 5.5 ft. wide, much of the remaining silage weight rests on the footing. The area under the silo that is not part of the footing $\doteq$ (7.17^2) $(3.14/4)$ $\doteq$ 40.4 ft.^{2}. To approximate the weight acting on this area, we will assume that all the silage above this portion of the foundation acts on this 40.4 $ft.^2$. From fig.11, we find that the average density of silage at a height of 55 feet is 55 pcf. Therefore, the weight of silage above the center area equals

$$(40.4)\ (55)\ (55) = 122{,}210 \text{ lbs.}$$

Weight of silage acting on footing equals

$$710{,}000 - 122{,}210 = 587{,}790 \text{ lbs.}$$

Total weight acting on footing equals

$$856{,}430 - 122{,}210 = 734{,}220 \text{ lbs.}$$

Bearing pressure exerted by footing on soil equals

$$\frac{734220}{(3.14/4)\ (18.17^2 - 7.17^2)} = 3353.8 \text{ psf} = 23.3 \text{ psi.}$$

Bearing pressure exerted by total weight over the total area equals

$$\frac{856{,}430}{(3.14/4)\ (18.17^2)} = 3302.9 \text{ psf} = 22.9\text{psi.}$$

These pressures are the ones that are acting right below the foundation. However, since the footing is two feet thick, and the sand layer is 2.5 feet thick, we will have about .5 feet of sand between the footing and clay-loam layer. Therefore, the load of the silo will be distributed over a slightly larger clay-loam area. According to Sowers, for a vertical distance of .5 feet, we can increase our effective outer diameter by .5 feet. The weight of the .5 feet of sand, however, is also acting on the clay-loam. Therefore, the total weight acting over the total effective clay-loam area equals

$$\frac{856{,}430}{(3.14/4)\ (18.67^2)} + (.5)\ (110) = 3183.3 \text{ psf} = 22.1 \text{ psi.}$$

The average pressure is slightly lower at the sand-clay-loam interface. In the next section, we will use the experimental results from our triaxial tests in equations 1 through 4, and compare the results with the values found above.

BEARING CAPACITY VALUES FROM EXPERIMENTAL RESULTS

The bearing capacity values from equations 1 through 4 will be compared not only to the actual bearing pressure, but also to typical values from the literature for this type of soil. Then, a suitable allowable bearing capacity will be determined for design. It will be assumed throughout that $\phi = 0$ conditions prevail.

Equation 1:

Equation 1 is Terzaghi's ultimate bearing capacity equation for a circular footing. Since the dry unit weight of the top layer of sand was not found experimentally, a typical value will be used. Lambe states that the minimum and maximum dry unit weights for a fine to coarse sand are 85 pcf and 138 pcf respectively. Therefore, an average figure would lie somewhere around 110 pcf. From fig.1, we find that for $\phi = 0$,

$$N_c = 5.14,\ N_q = 1,\ \text{and}\ N_\gamma = 0$$

For c = 2.43 = 350 psf

$$q_u = (1.2)(350)(5.14) + (110)(2)(1)$$
$$= 2378.8 \text{ psf} = 16.5 \text{ psi}$$

When using the ultimate bearing capacity equation, it is assumed that a general shear failure will occur, and that the soil is fairly dense or stiff. However, no bulging of the earth was noticed, and we have already determined that our soil is at least moderately sensitive. Also, cracking of the soil occurred around the foundation which is often a sign of local shear failure. Therefore, it is very possible that local shear rather than general shear was the cause for failure.

Equation 2:

Equation 2 is Terzaghi's local shear failure equation.

For $c' = 2/3c = 1.62$ psi ≈ 233.3 psf,

$$q_L = (1.2)(233.3)(5.14) + (110)(2)(1)$$
$$= 1658.9 \text{ psf} = 11.5 \text{ psi}$$

Equation 3:

Equation 3 is Skempton's equation for the ultimate bearing capacity of a clay. However, c is defined as the average shear strength of the soil to a depth of 2/3 times the diameter of the foundation. We do not have this value of c, but will assume that the value of c from our tests will be a good estimate of this average strength. Therefore,

$$q_u = (350)(6.6) + (110)(2)$$
$$= 2530 \text{ psf} \approx 17.6 \text{ psi}$$

Equation 4:

Equation 4 is another equation proposed by Skempton to determine the ultimate bearing capacity of a clay with a shallow footing. D is the diameter of the footing and will be taken as 18.2 feet.

$$q_u = (6.2)(350)(1 + (.2)(2)/(18.2))$$
$$= 2217.7 \text{ psf} = 15.4 \text{ psi}$$

From the results above, we see that the maximum ultimate bearing capacity value is 17.6 psi. The actual bearing pressure is approximately 22 psi. Therefore, it is no wonder that the foundation of this silo failed.

Comparison with Values from the Literature:

Terzaghi states that the unconfined compressive strength of undisturbed clays with a liquidity index near unity, as it is in our case, ranges between 4.3 and 14.2 psi. However, $c \approx 1/2$ times the unconfined compressive strength. Therefore, c ranges from 2.15

to 7.1 psi. Using this value in equations 1 through 4, we get a range of bearing capacities, with a minimum of 10.37 psi to a maximum of 48.4 psi. Since our liquidity index is actually greater than one, it is probable that the ultimate bearing capacity of our soil would lie somewhere in the lower region of the above range. The values that we found from our experimental results do lie in this lower part of the range.

From fig.12, taken from the "Canadian Farm Building Code", we find that for a soft cohesive soil, the approximate undrained shear strength ranges from 250 to 500 psf (1.74 to 3.47 psi). Using these values as c in equations 1 to 4, our range of bearing capacities goes from 7.49 to 24.43 psi. Our values fall within this range. It therefore seems that the values of bearing capacity found experimentally are very reasonable, because they correlate well with typical values for this type of soil which were found in the literature.

Choice of a Suitable Allowable Bearing Capacity for Design:

M. Bozozuk and J.E. Turnbull have both worked specifically with the design of tower silos on the sensitive marine deposited soils of Canada. They both recommend that Skempton's ultimate bearing capacity equation(equ.3) with a factor of safety of three should be used to determine an allowable bearing capacity. It seems that they have decided that this safety factor would take into account the risk of local shear failure and still give us a safe design criteria. Therefore, in the next section, the footing that should have been built for this silo will be designed with Skempton's equation as the design criteria.

DESIGN OF AN ADEQUATE FOOTING:

The "Canadian Farm Building Code" recommends that any footing that is not resting on bedrock should be placed at a depth below the frost zone. This was not done for the original footing, but the one that will be designed here will have a bottom depth of four feet. However, as you increase the depth of the footing the bearing capacity changes due to the increased surcharge. Assume that the density of the soil surrounding and above the footing equals the average value of the sand and clay-loam.

$$= (121.05 + 110)/2 = 115.5 \text{ pcf}$$

Therefore, using Skempton's equation (equ.3) with a factor of safety of three, we find that the allowable bearing capacity for design will be

$$q_a = \frac{(6.6)(350) + (115.5)(4)}{3}$$

$$= 924\text{psf} = 6.42\text{psi}$$

Under the same surcharge conditions, using equations 1 and 2, we find that

$$q_u = 18.20 \text{ psi}$$
$$q_L = 13.20 \text{ psi}$$

An allowable bearing capacity of 6.42 psi will give us a safety factor of 2.83 according to Terzaghi's ultimate bearing capacity equation, and a safety factor of 2.06 with reference to his local shear failure equation. Therefore, designing with an allowable bearing capacity of 6.42 psi should give us a safe, adequate footing size.

Equations 6 through 10 will be used for design purposes. From a previous section, we know that the total silage-to-wall friction load equals 360,446 lbs. The friction load per foot of circumference, F,

$$= 360{,}446/(3.14)(18)$$
$$= 6374.08 \text{ lbs.}$$

S equals the weight of the silo, roof, and accessories per foot of circumference

$$= (77{,}754 + 3000)/(3.14)(18)$$
$$= 1429 \text{ lbs.}$$
$$(F + S) = 7803.08 \text{ lbs.}$$

The weight of the footing itself has been taken into account in equation 6. However, it does not account for the pressure that the soil is exerting on the footing. This pressure is dependent on the thickness of the footing, d. Also, because one foot of the footing width, B, is inside the silo(Turnbull), the soil is not resting on the total area of the footing. Therefore, this soil pressure, Z,

$$= \frac{(3.14/4)\left[(2B + 16)^2 - 18^2\right](115.5)(4-d)}{(3.14/4)\left[(2B + 16)^2 - 16^2\right]} \quad . . . . . (11)$$

The numerator is the area of the footing resting outside the silo times the density of the soil times the depth of soil above the footing. The denominator is the total area of the footing. Therefore, by iteration, we can assume a depth d, solve for B in equ.6, use these results to find e in equ.7, and check our value of d in equ.9. After several iterations, with P = 924 - Z, we find that

$$d = 1.5 \text{ ft.}, \quad B = 11.86 \text{ ft.}$$

However, we must now check whether the requirement that the total area must be able to support the total weight is satisfied.

$$D_o = \text{outside ring diameter} = (11.86)(2) + 16 = 39.72 \text{ ft.}$$
$$\text{Total area} = 1239.11 \text{ ft.}^2$$

$$\text{Total weight} = (710{,}000 + 77754 + 3000) + (3.14/4)(39.72^2 - 16^2)(150)(1.5) + (3.14/4)(39.72^2 - 18^2)(115.5)(2.5) =$$

1,308,627.7 lbs.

Bearing pressure = 1,308,627.7/1239.11

= 1056 psf = 7.33 psi

Since this value is greater than our allowable bearing of 6.42 psi, we must base our design on the outer diameter, D_o. The equation that must now be satisfied is

$$q_a = \frac{790{,}754 + (3.14/4)(D_o^2-16^2)(150)(d) + (3.14/4)(D_o^2-18^2)(115.5)(4-d)}{(3.14/4)(D_o^2)}$$

$$\therefore 924 \text{ psf} = \frac{1{,}006{,}819.3 + (150)(D_o^2-16^2)(d) + (115.5)(D_o^2-18^2)(4-d)}{D_o^2} \quad . . . (12)$$

By iteration, using d = 1.5 ft. as the first approximation in equ.12, we can find a value for D_o, B, then e from equ.7, and check if our value of d is correct by equ.9. After several iterations, we find that

D_o = 45.9 ft., B = 14.95 ft., d = 1.6 ft., e = 7.68

From equ.8, we can find the required spiral steel area, A_s.

$$A_s = \frac{(18)(7.68)(7803.08)}{41{,}136(1.6 - .33)}$$

$= 20.64$ sq. in.

From equ.10, the extra required spiral steel area to resist lateral pressure, A_L,

$$= \frac{(1.6)(16)}{48{,}000} \quad 100 + (1.92)(55)(16)^{.55}$$

$= .312$ sq. in.

If we use no.6 rebars (section area = .44 sq. in.), the number of turns required to satisfy the A_s equals 20.64/.44 = 47 turns. One extra turn will be required to satisfy A_L. In fig.13, the placement of these bars and the footing size required, in accordance with Turnbull, will be shown. Radial no.4 rebars should be placed just below the spiral bars, at a spacing of four feet around the interior circumference of the footing. Therefore, 13 radial rebars are required.

CONCLUSIONS

The results of this study show that the actual bearing pressure exerted by this 55' x 18' silo exceeded the ultimate bearing capacity of the sensitive clay-loam stratum by a significant amount. Since no bedrock was present to depths greater than 100 feet, failure was imminent.

The outside diameter of the actual footing is 18.2 feet. From the results and calculations presented in this report, we have found that a footing with an outside diameter of 45.9 feet would have been required to safely support the silo load. This huge footing, however, would have costed a very large sum of money. It is most probablethen that Mr. Menard would never have had this type and size of silo built if the silo company had designed the footing properly. Unfortunately, Mr. Menard had the structure built, and it failed the first time it was filled. Now he must decide what action to take. He cannot leave the silo in the precarious position it is in,because of the danger of it falling. Also, he cannot just straighten it up, because the soil under its present position has failed, and is probably in its remoulded state, which is an exceedingly weak one. If he moves the silo, he must build a footing that will reduce the bearing pressure below a permissible value or failure may again occur. He may want to look into altrnate methods of support as outlined in the "Requirements for further study" on the next page. Also, to reduce the size of footing he may want to take a chance and design it using a safety factor of 2 instead of 3. With a safety factor of 2, and using the calculations presented earlier, D_o would be approximately equal to 31.5 feet, with the footing width equal to 7.75 feet.

Finally, in my opinion, I think that some steps should be taken by the government to force the silo companies to perform the required soil tests so that the adequate footing size will be used. This would greatly reduce the number of silo foundation failures on these weaker soils.

REQUIREMENT FOR FURTHER STUDY

The design of the required footing was based only the bearing capacity of the soil. However, the effect of settlement must also be checked. New samples must be taken, and tests be performed to see whether the settlement will be below a permissible value. If it is not, the footing must be redesigned with settlement as the criteria.

Because of the huge size of the footing needed to distribute the weight of the silo safely, the cost will be excessive. Therefore some study should be made to see if an alternative method could be designed to support the silo at a lower cost. One method could be through the use of piles below the foundation to reduce the size of footing needed. Also, the water in the soil below the footing could be pumped out, thus increasing the bearing capacity of the soil.

APPENDIX A

Sample Calculations

SAMPLE CALCULATIONS

Direct Shear Tests:

For core # 1, the maximum proving ring readings for each test at different normal loads were as follows:

NORMAL LOAD (lbs):	0	8.82	17.64	26.46	35.27
Max. PROVING RING READING (.001 in.)	30	35	61	68	91

Before performing any tests with soil samples in the shear box, several trials were made without any soil to determine the force that was necessary to move the top brass plate over the bottom brass plate. It was found that an average of 7.3 thousandths of an inch on the force gage was required. Therefore, this amount was subtracted from the readings that were obtained with soil present.

From the calibration of the proving ring, we find that each .001" of deflection represents a force of .2696 lbs. The size of each sample was approximately 2.02" x 2.02" x .8", giving us a shear area of 4.08 in.^{2}. Lambe states that due to the type of shear box that we have,(broad-lipped)and because of the small displacements involved,a constant area could be assumed for all calculations. Therefore, for a normal load of 0 lbs. and a maximum force reading of 30, we get a normal stress of 0/4.08 and a maximum shear stress of

$$(30-7.3)(.2696)/4.08$$
$$= 1.50 \text{ psi}$$

The maximum shear stress for the remaining tests were found in the same manner as shown above.

Triaxial Tests:

From the calibration of the proving ring, we find that a deflection of .0001" represents a force of .843 lbs. Lambe suggests that the area to be used for calculations = $A = A_o/(1-\epsilon)$, where A_o is the initial area and ϵ equals the strain(in./in.). Therefore, P_1 at any time $= P_3 + (.843 \times \text{force reading})/A$.

The results are plotted on a stress-strain curve (fig.14) and the maximum P_1 is then chosen. These maximum P_1's from the different tests at the same normal pressure (10,20, or 40 psi) are then averaged and are used along with their respective P_3 values to draw up three Mohr's stress circles. (Refer to fig.15). By geometry, for any Mohr's circle, we know that

$$P_1 = P_3 \tan^2(45 + \phi/2) \quad 2c \tan(45 + \phi/2) \quad . . . (13)$$

By subtraction, we can find the tangent line to any two circles by using

$$P_{1b} - P_{1a} = (P_{3b} - P_{3a}) \tan^2(45 + \phi/2) \quad . . . . (14)$$

Circles 1 and 2:

$P_1 = 16.11$ psi	$P_1 = 28.44$ psi
$P_3 = 10$ psi	$P_3 = 20$ psi

$$(28.44 - 16.11) = (20 - 10) \tan^2 (45 + \phi/2)$$

$$45 + \phi/2 = \tan^{-1}(12.33/10)^{\frac{1}{2}} = 47.99^{\circ}$$

Therefore, $\phi = 5.99^{\circ}$, and $c = 1.70$ psi. In the same manner, for circles 1 and 3, c and ϕ for the tangent line are 2.27 psi. and 3.48°. For circles 2 and 3, $c = 3.32$ psi and $\phi = 2.14^{\circ}$. With these c-ϕ values for each line, we can find,by linear regression, the line that will come closest to being tangent to all three circles. From this line, we get $c = 2.43$ psi and $\phi = 3.873^{\circ}$.

Particle Size Analysis:

Table 7: Hydrometer Readings

TRIAL #	1	2	3	4
40 sec. reading	1.0210	1.0215	1.0240	1.0220
2 hr. reading	1.0100	1.0095	1.0105	1.0093

From Lambe, we find that

$$N = (G/(G-1))(V/W_s)(\gamma_c)(r-r_w) \times 100\% \;........(13)$$

N = % finer

G = specific gravity of solids (range: 2.60 to 2.80, use 2.70)

W_s =weight of dry soil = 50g

γc = .9982

r = hydrometer reading in suspension

r_w = hydrometer reading in water at the same temperature as suspension (essentially = 1)

V = volume of suspension = 1000 cc's

Therefore, for trial #1, 40 second reading,

$$N = (2.70/1.70)(1000/50)(.9982)(1.0210-1) \times 100\%$$
$$= 66.57\%$$

Percent sand = 100 - 66.57 = 33.41%

Continuing in the same fashion we get the results shown below:

Table 8: Particle Size Results

TRIAL #	1	2	3	4	Mean
% sand	33.41%	31.83%	23.90%	30.24%	29.85%
% silt	34.89%	38.05%	42.81%	40.26%	39.00%
% clay	31.70%	30.12%	33.29%	29.50%	31.15%

From the textural triangle, we find that we have a "Clay-Loam".

Liquid Limit Test:

Table 9: Results of Liquid Limit Tests

TRIAL #	1	2	3	4	5	6
# of Blows	43	36	24	22	15	9
Moisture Content %	23.27%	24.36%	24.99%	25.97%	30.19%	29.39%
TRIAL #	7	8	9	10	11	
# of Blows	46	24	21	15	6	
Moisture Content %	21.48%	24.81%	28.24%	28.83%	30.50%	

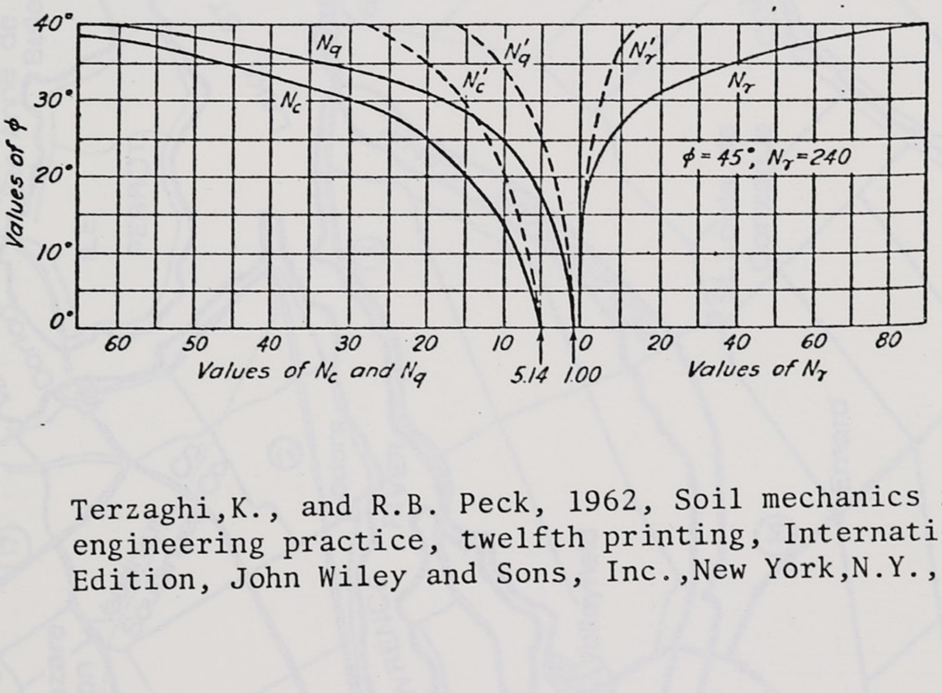
In fig.16 is the plot of moisture content versus the log of the number of blows. The moisture content at 25 blows is the liquid limit. Therefore,

Liquid Limit = 25.67%

APPENDIX B

Figures

fig. 1 : BEARING CAPACITY FACTORS



Terzaghi,K., and R.B. Peck, 1962, Soil mechanics in engineering practice, twelfth printing, International Edition, John Wiley and Sons, Inc.,New York,N.Y., USA

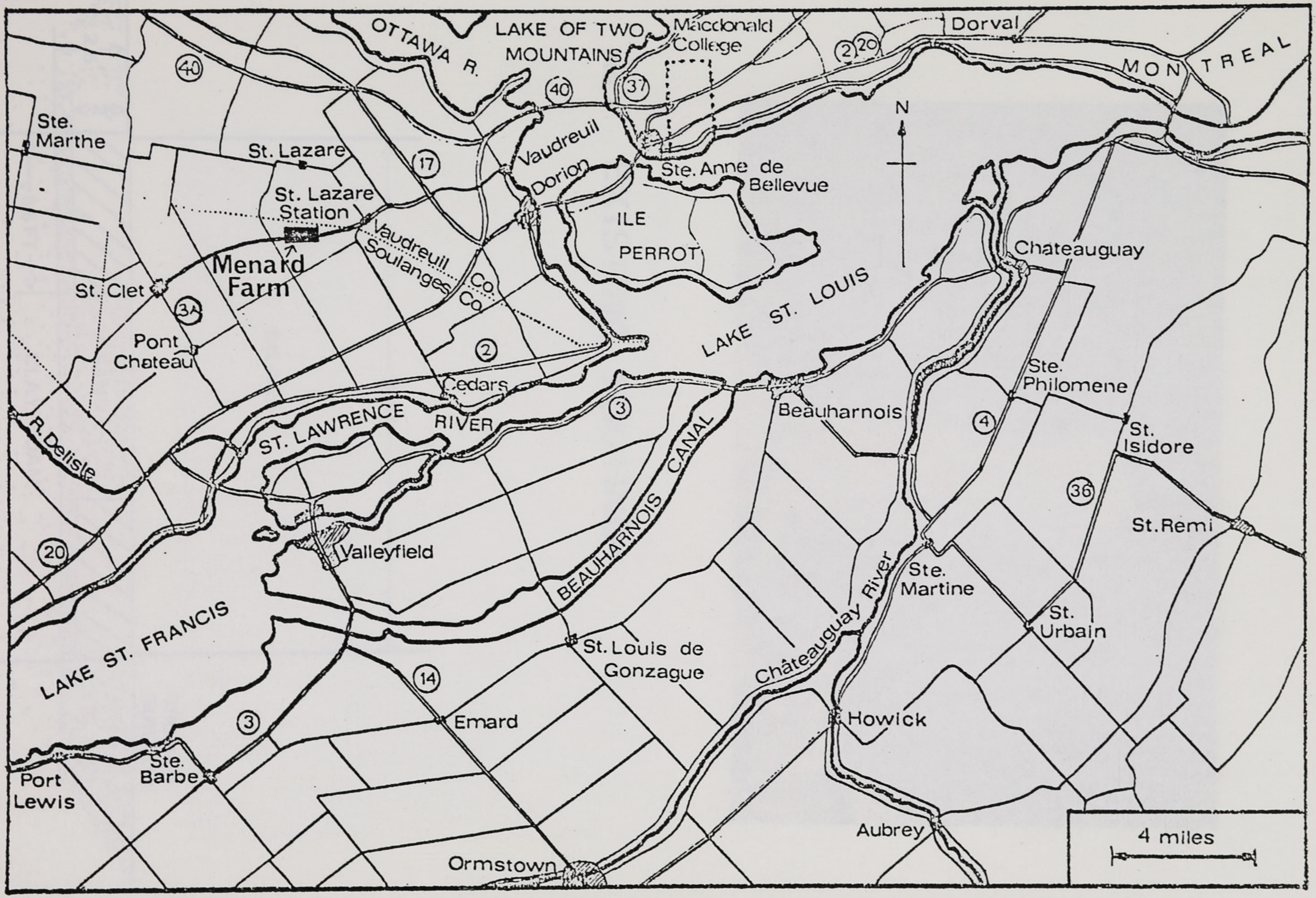


Figure 2 Map Showing Part of South-western Quebec and Location of MENARD FARM

FIG. 3 THE SILO

FIG. 4 EXISTING FOOTING

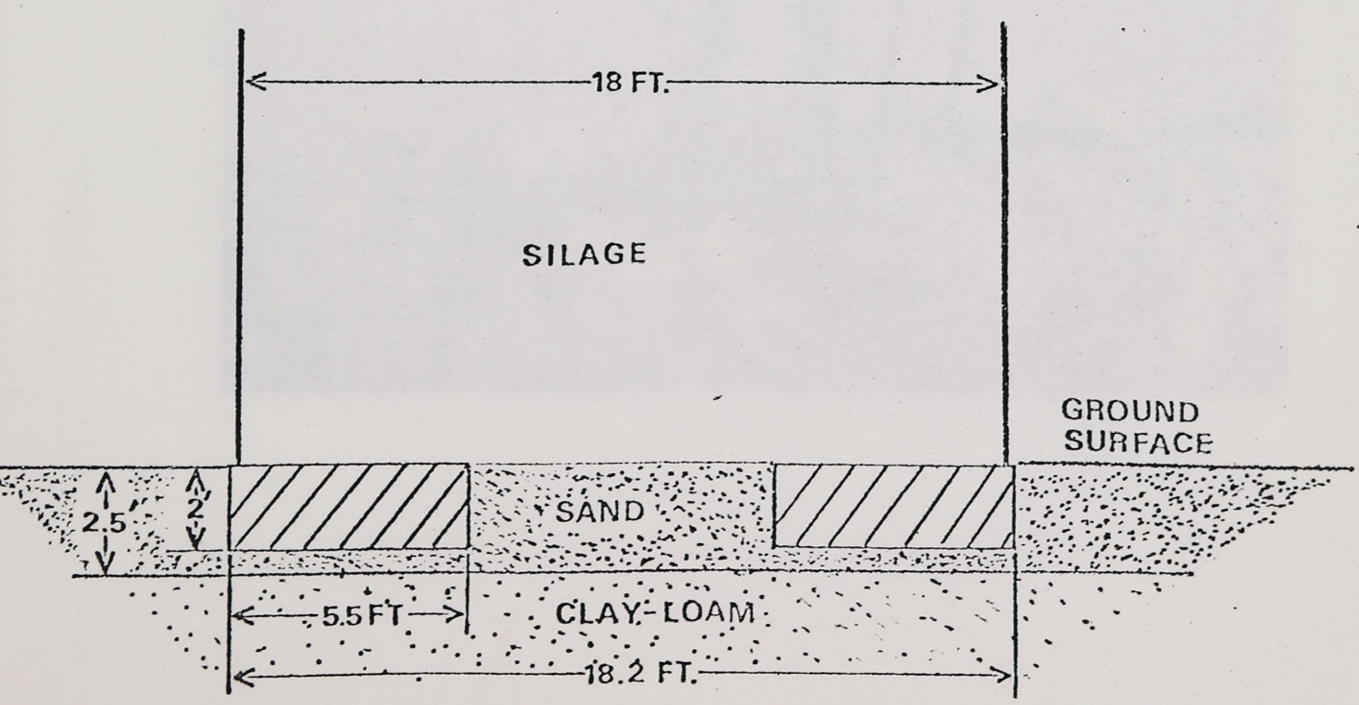


FIG. 5 **SOIL PROFILE**

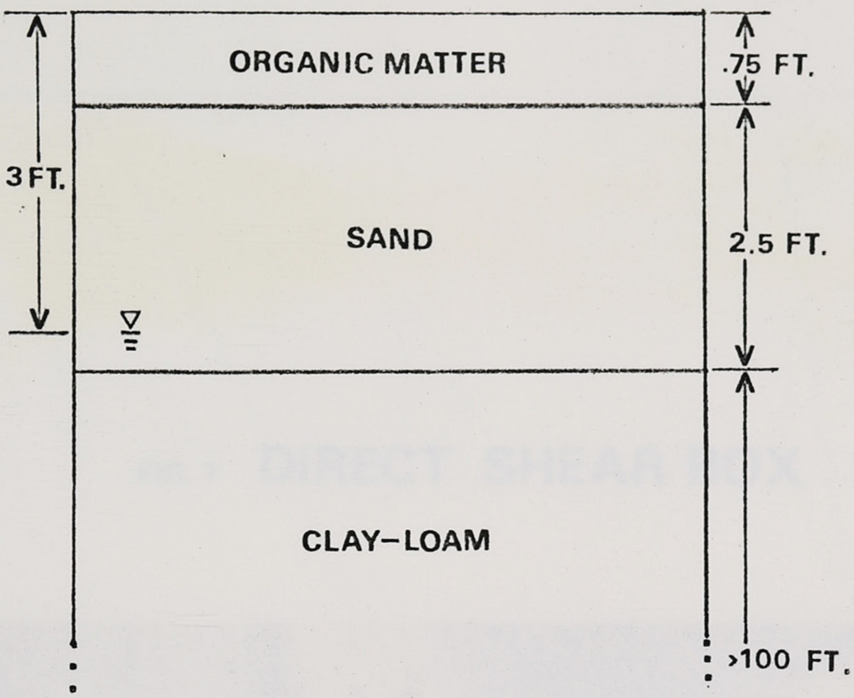


FIG. 6 **THE SAMPLES**

FIG. 7 **DIRECT SHEAR BOX**

FIG. 8 DIRECT SHEAR TEST RESULTS

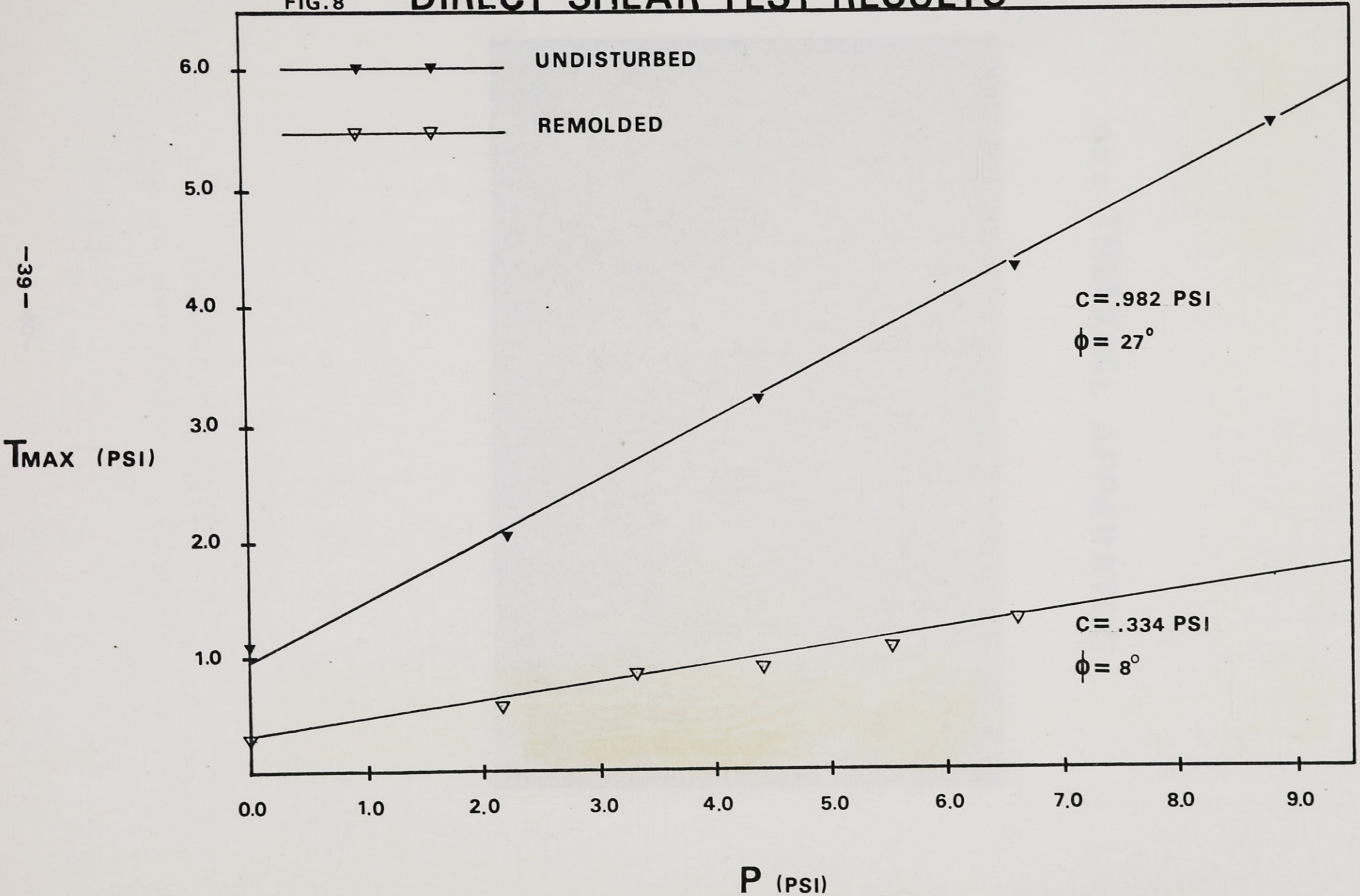
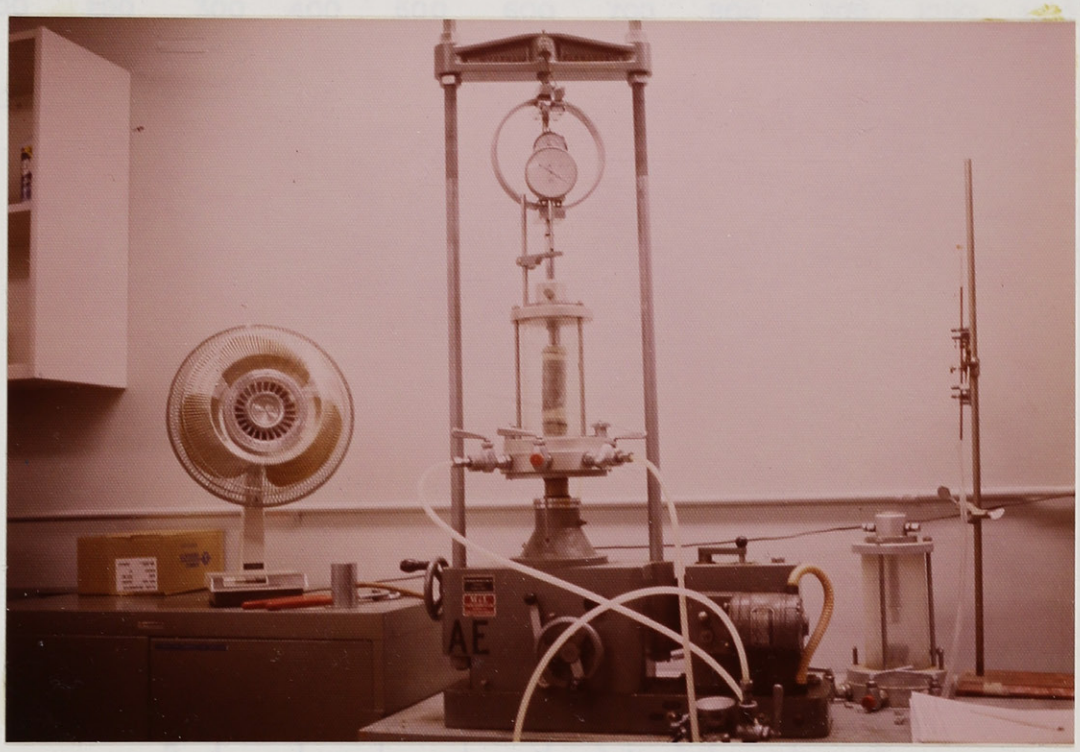


FIG. 9 **TRIAXIAL APPARATUS**



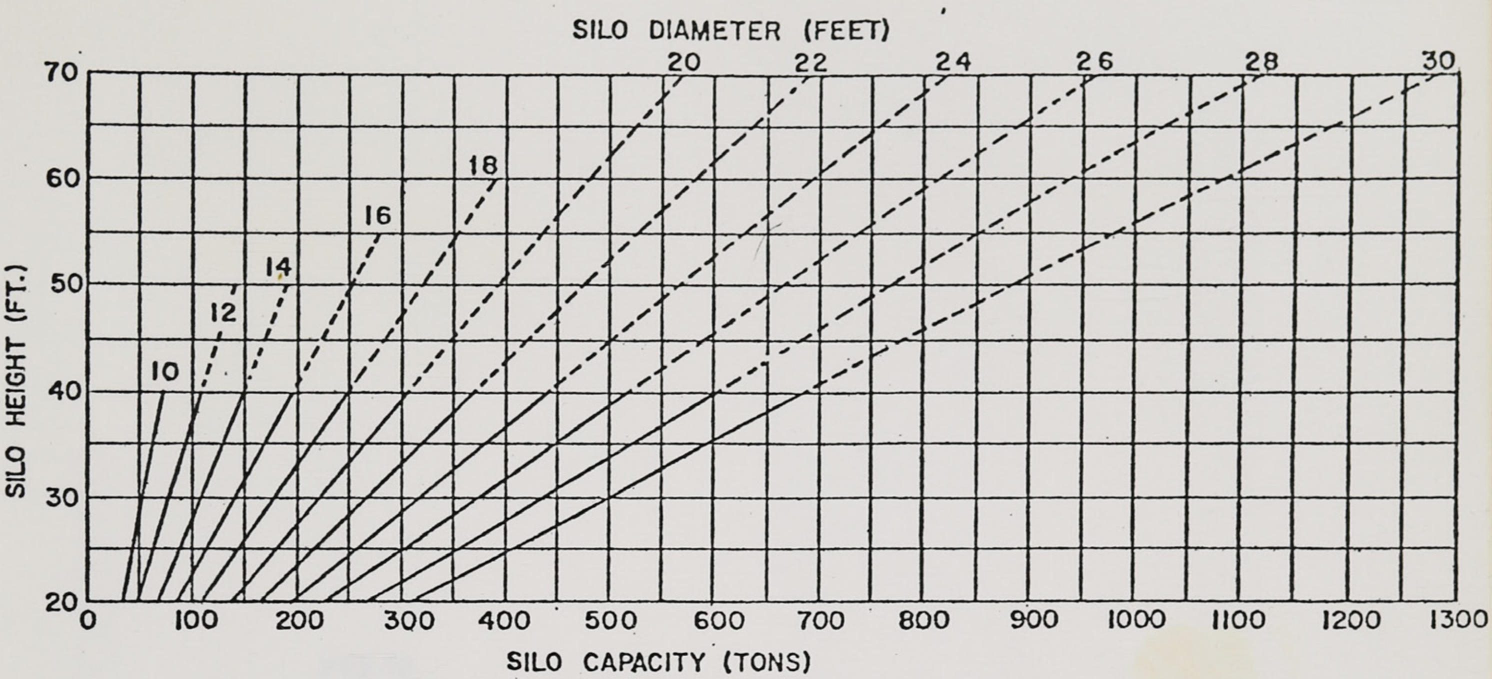


FIG.10 AVERAGE CAPACITIES OF TOWER SILOS

Figures 10 and 11 taken from ASAE DATA: ASAE D252, Tower silos: Unit weight of silage and silo capacities, 1976, Agricultural Engineer's Yearbook

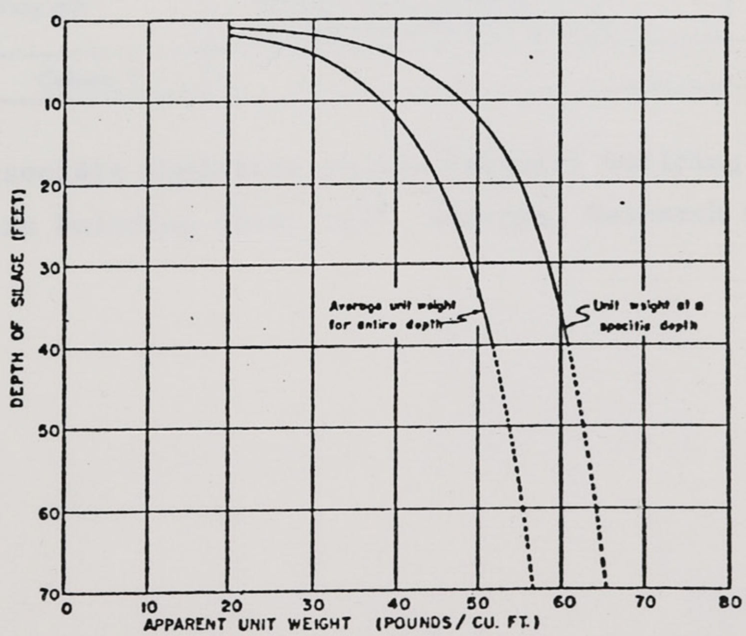


FIG.11 APPARENT UNIT WEIGHT OF SILAGE IN TOWER SILOS

FIG.12

IDENTIFICATION OF COHESIVE SOILS		
Consistency	Description	Approximate Undrained Shear Strength, psf
Very stiff	Of a type impossible to indent with the thumb but readily indented with the thumbnail	Over 2,000
Stiff	Of a type difficult to indent with the thumb; with difficulty it can be remoulded by hand	1,000 to 2,000
Firm	Of a type that can be indented by moderate thumb pressure	500 to 1,000
Soft	Of a type that can be penetrated several inches with the thumb	250 to 500
Very soft	Of a type that can easily be penetrated several inches by the fist	less than 250
Column 1	2	3

Associate Committee on the National Building Code, Canadian farm building code, 1977, National Research Council of Canada.

FIG. 13 DESIGN OF ADEQUATE FOOTING

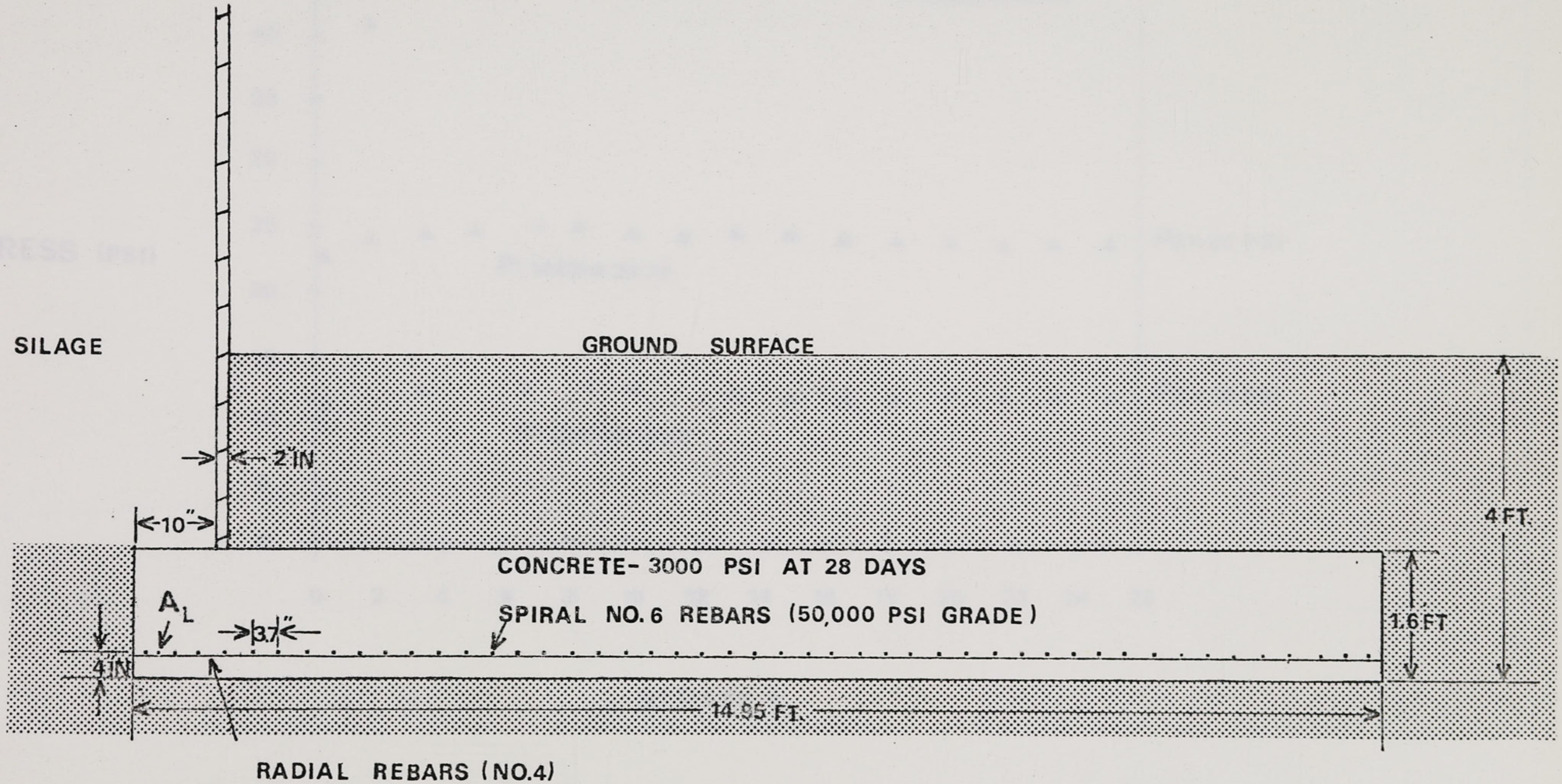


FIG.14 STRESS - STRAIN CURVE (CORE NO. 10)

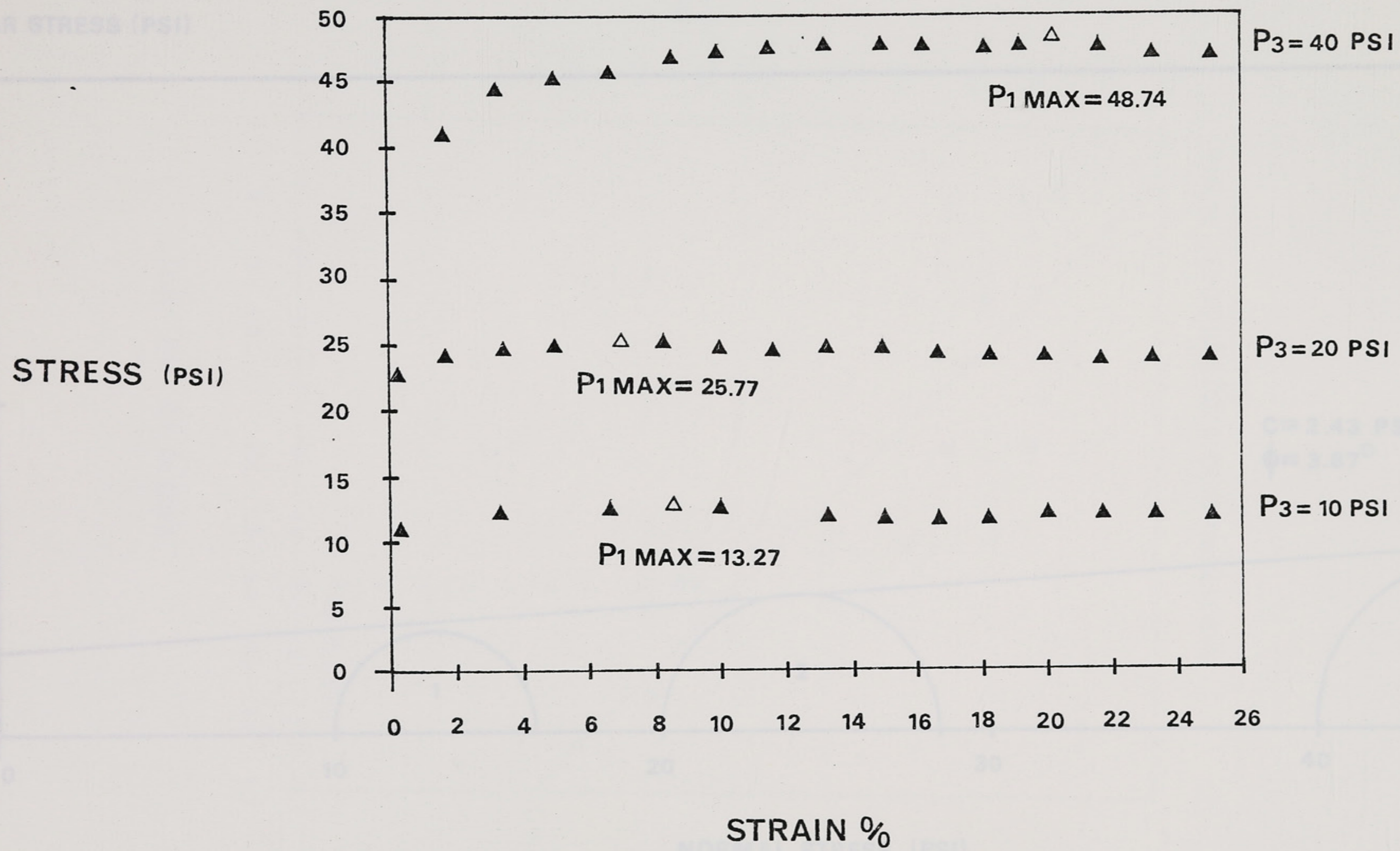


FIG. 15 MOHR'S STRESS CIRCLES FOR TRIAXIAL RESULTS

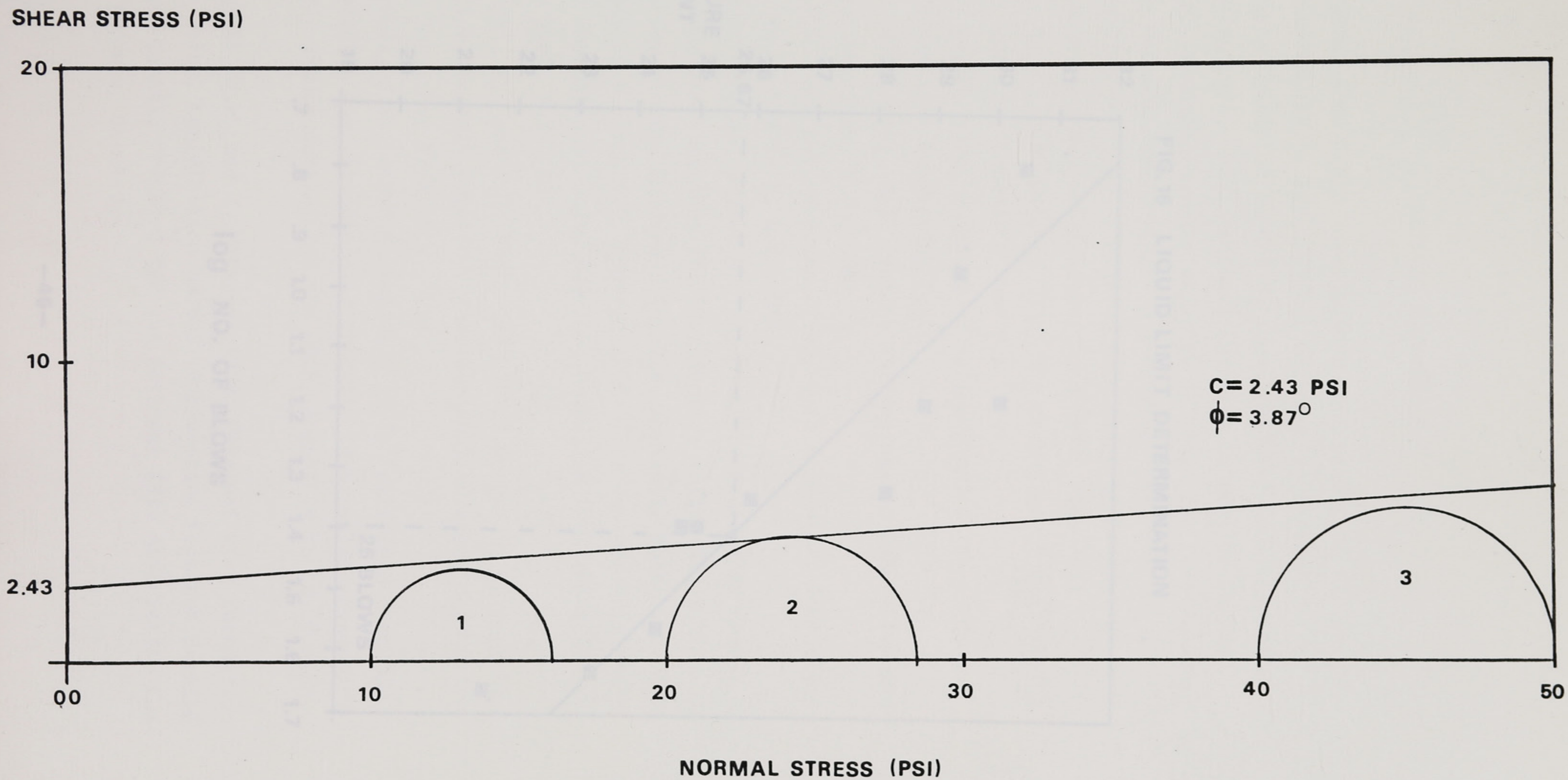
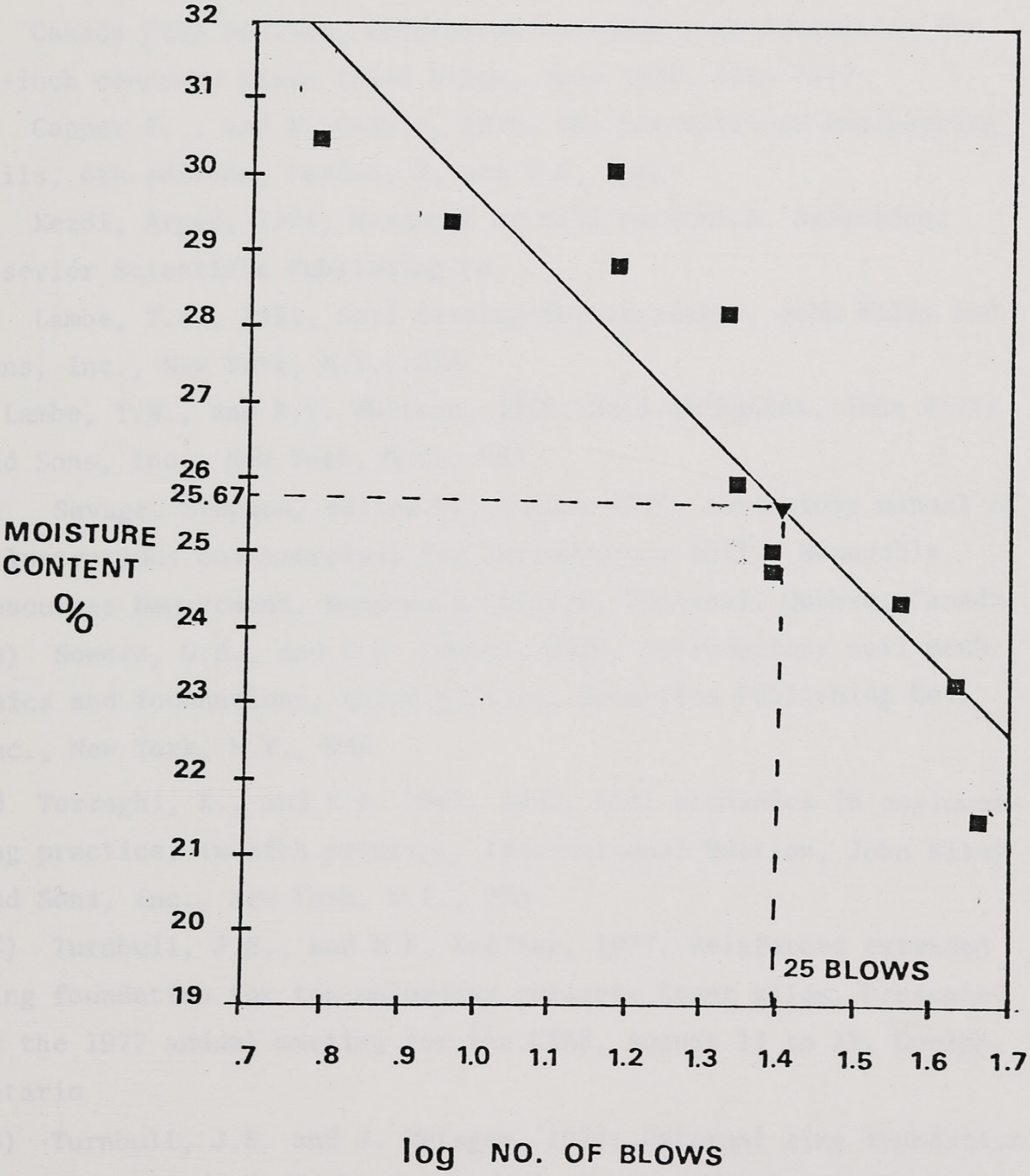


FIG. 16 LIQUID LIMIT DETERMINATION



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