FRICTION VALUES FOR CAST-IN-PLACE CONCRETE PILES IN A TYPICAL WINNIPEG CLAY

A Thesis

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Master of Science in Civil Engineering

by

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TABLE OF CONTENTS

CHAPTER | | PAGE

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LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

c	Cohesion
c_a	Adhesion
ϕ	Angle of shearing resistance
δ	Angle of skin friction
s	Shearing strength
s_a	Skin friction
s_n	Natural shearing strength of the clay
s_s	Softened shearing strength of the clay
α	Coefficient of friction
C_s	Coefficient of softening
p	Normal pressure
Q_u	Ultimate bearing capacity
Q_p	End bearing capacity
Q_s	Skin friction capacity
W	Weight of pile
N_c	Bearing capacity factor
N_{cr}	Bearing capacity factor of rectangular pile
N_{cs}	Bearing capacity factor of long strip pile
D_f	Depth of pile below ground surface
γ	Average density of the soil
A_s	Circumferential or skin area of the pile shaft embedded in clay
A_p	Area of pile point

W/C	Water cement ratio
T	Distance of shear plane from mortar
w	Moisture content at the shear plane
Δw_1	Change in moisture content at the interface of clay-mortar
Δw_2	Change in moisture content at 3/8 in. from the mortar
Δw_3	Change in moisture content at 3/4 in. from the mortar
K_A	Coefficient of active earth pressure
K_o	Coefficient of earth pressure at-rest
K_s	Coefficient of earth pressure of the clay
μ	Poisson's ratio

SUMMARY

Friction values for cast-in-place piles in a typical Winnipeg clay were studied. Shear conditions in the soil near a pile were simulated in the laboratory, using a direct shear apparatus and undisturbed samples of the clay were placed in contact with wet mortar.

Softening of the clay due to soaking under controlled pressures was investigated. In addition, the shear strength of the clay at various distances from the clay-mortar interface was investigated to determine effects of the water/cement ratio and the time of curing of the mortar.

It was found that the consolidated undrained shear parameter c, decreased with increasing moisture content whereas ϕ remained unchanged.

The friction value between the clay and cast-in-place mortar was increased due to the effect of cementing action of the mortar. At the contact surface, the friction was about 2.2 times the shear strength of the clay. The strength of the clay-mortar decreased with increasing distance from the mortar. At a distance of about 0.25 inches from the mortar, its value was approximately equal to the natural shearing strength of the clay for the same moisture content.

The weakest plane was found to be a short distance from the clay-mortar interface, it was at the distance where

the shear strength of the clay-mortar was about the same as the clay strength. The clay was softened by water from the cast-in-place mortar. The coefficient of softening (C_s) of the clay depended on the W/C ratio of the mortar.

The test results indicated that the friction value of the clay for cast-in-place concrete piles was actually the shearing strength of the clay, reduced for water-softening.

The clay used for the test was a clay of medium strength. A safe friction value equal to 200 psf may be used for design practices.

CHAPTER I

STATEMENT OF PROBLEM, SCOPE AND METHOD OF INVESTIGATION

1.1 THE PROBLEM

The friction pile, where a bored hole is made by an auger and filled with concrete to form a pile, is one of the most widely used types of pile foundations in cohesive soils. The advantages of a cast-in-place concrete pile are that the required length is known exactly, there is no wasted length to be cut off and no need of handling and driving. Disadvantages of the cast-in-place concrete pile include the uncertainity of the final condition of the concrete. The concrete may be non-homogeneous because the soil in the bored hole may become mixed with the concrete; the proportion of the mix may not be uniformly distributed due to the segregation of the aggregate during the pouring of the concrete.

The use of the friction value for calculation of pile bearing capacity is very complicated and is still based to a large extent on the empirical correlationships obtained from field loading tests. The variation of the friction value is not well understood due to lack of data. The boundary conditions at the considerable depth of a pile below the ground surface are generally unknown. Friction between a pile and soils around it depends on many factors such as types of soil, types of pile, depth, moisture content and time. In design practice, the friction value is assumed to be a fraction of the shear strength of the soil so as to give an adequate safety factor, usually of about 2 to 3. Settlements must be also considered, but are not part of the present study.

The shear strength of clay will decrease with increasing moisture content in the clay. Friction between the clay and pile is directly proportional to the shear strength of the clay(3), so that it will also decrease when moisture content increases. This may be seen from :

$$c_a = \alpha c \qquad \text{....................(1)}$$

c_a = Unit skin friction, or adhesion value, between the soil and the pile.

α = Coefficient of friction value.

c = Shear strength of the soil.

From previous studies, they indicated that value of α is not constant and frequently falls between 0.5-1.0

The change of moisture content in the clay around the pile is still one of the main problems and only a few field investigations have been reported(14). Clay will be softened by water from fresh concrete or from the boring equipment, and its strength and friction value will decrease to some extent.

The load carrying capacity of a friction pile in a cohesive soil seems to be well established after a period of time, when the soil strength lost by the effect of disturbance during boring, placing or driving the pile is regained. Such action has been attributed to thixotropy, or when the excess water diffuses away into the surrounding area, the soil strength will increase automatically. From the observations of pile pulling tests it has been shown that a thin layer of cohesive soil adheres to the pile. This indicates that the true skin friction at the pile shaft is greater than the soil strength at the failure surface. In general, the failure surface of a friction pile in clay occurs in the clay mass at some distance beyond the pile shaft,and it is thus quoted by many engineers that the friction value of a cast-in-place concrete pile should be the same as the soil strength around the pile (2,9)

1.2 SCOPE AND METHOD OF THE INVESTIGATION

The object of this thesis was to study the friction value of clay for cast-in-place concrete piles. An attempt was made through laboratory investigations, using direct shear tests, to represent the actual movement of the pile. Mortar was used instead of concrete to carry out the test. More details are given in Chapter III.

The testing program included :

1. Investigating the shearing strength of the clay at different moisture contents and different normal pressures.

2. Investigating the friction values between the clay and cast-in-place mortar blocks. The observations were made under the variation of the following factors :

- water/cement ratio of mortar,
- normal pressure,
- distance of shear plane from mortar face,
- aging of mortar in contact with soil.

(More details are also given in Chapter III).

3. Investigating the change in moisture content of the clay specimen due to cast-in-place mortar for different W/C ratios and different periods of aging.

The author has tried to interpret the test results using forms of graphical representation to establish, where possible, correlations between the strength and friction value of clay on cast-in-place mortar under different conditions so that the results may be used as a guide for further study or for practical purposes.

CHAPTER II

THEORIES OF FRICTION PILE CAPACITY AND PREVIOUS INVESTIGATIONS

2.1 DEFINITION OF STRENGTH PARAMETERS

In general, the skin friction is closely related to the soil properties and the characteristics of the pile. Thus, the shearing strength of soils can be expressed by :

$$s = c + p.\tan\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in terms of total stresses, where :

c = cohesion,

p = normal pressure on shear plane,

ϕ = angle of shearing resistance.

Similarly, the skin friction between a soil and a pile can be expressed by :

$$s_a = c_a + p.\tan\delta \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in terms of total skin friction, where :

c_a = adhesion,

p = normal pressure on shear plane,

δ = angle of skin friction.

2.2 FRICTION PILE FORMULAS

Pile formulas, based on static conditions for friction pile, are semi-empirical. These formulas use the relationship :

Ultimate bearing capacity = Skin friction capacity + End bearing capacity

The simplified form of the static formula, based partly on the average skin friction and shear strength of soil, and partly on the observation of field loading tests, has been proposed by Terzaghi & Peck(1) and is known as Terzaghi's Semi-Empirical formula. This formula has been put forward by Skempton and has become one of the most widely used formulas for friction piles.

Skempton's solution for clays

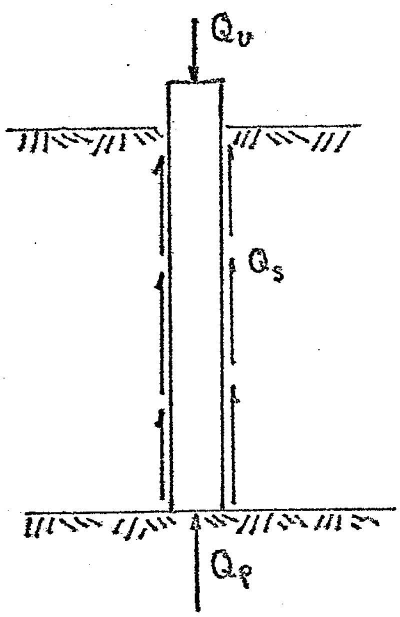


Figure 1.Friction Piles

The simplest formula, partly theoretical and partly empirical, has been proposed by Skempton for piles embedded in saturated clay, where ϕ can be assumed to be zero :

$$Q_u = Q_p + Q_s \qquad \ldots\ldots(4)$$

Q_u = ultimate bearing capacity,

Q_p = end bearing capacity,

Q_s = shaft bearing capacity or skin friction capacity.

End bearing capacity can be calculated from :

$$Q_p + W = A_p(c.N_c + \gamma .D_f) \qquad \ldots\ldots\ldots\ldots\ldots(5)$$

W = weight of pile,

A_p = area of pile point,

c = average shear strength of the soil,

N_c = bearing capacity factor,

γ = average density of the soil within a depth of D_f below the ground surface.

The c value is equal to one-half of the unconfined compressive strength of the soil. Skempton(3) suggested that it would be more accurate if the shear strength of the soil at a depth of about two-thirds of the pile's diameter below its base be used.

In most case, it is a sufficiently close approximation to assume that the weight of the soil, which is replaced by the pile, is equal to the weight of the pile :

$$W = A_p . \gamma . D_f \quad \text{.........................(6)}$$

Thus, the equation (5) becomes :

$$Q_p = A_p . c . N_c \quad \text{.......................(7)}$$

The N_c value has been studied and observed by many investigators who have come to the conclusion that for circular areaa loaded at a considerable depth within a saturated clay, the value is equal to about 9. This value is accurate enough for practical purposes and has been generally accepted.

Figure 2 shows the N_c value, which is based partly on laboratory tests, on theory and on observation of full scale loading tests by A.W. Skempton(13). For a long pile the ratio of D_f/B is much more than 5, so that the N_c value of 9 is usually used in the case of a square or a circular pile.

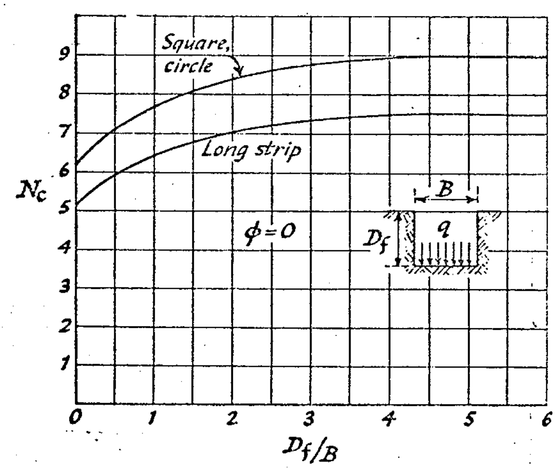


Figure 2. Bearing Capacity Factors
(after A.W.Skempton)

For the long strip (sheet pile) or square pile, N_c values may be obtained directly from the curves. For intermediate shapes, a rectangular strip of length L and width B, the N_c can be calculated from the equation :

$N_{cr} = (1 + 0.2\ B/L)N_{cs}$ ....(7.1)

N_{cr} = bearing capacity factor of rectangular pile,

N_{cs} = bearing capacity factor of long strip from Figure 2.

Shaft bearing capacity or friction capacity can be calculated from :

$Q_s = A_s . c_a$ ....(8)

A_s = area of the shaft of the pile embedded in clay,

c_a = average unit skin friction or adhesion value on the pile in clay = α c.

2.3 UNIT SKIN FRICTION OR ADHESION VALUE(c_a) FROM PREVIOUS INVESTIGATIONS

The investigation of the friction value of clay on concrete piles has been made by many investigators. Most of them made the observations on field loading tests, and only a few laboratory tests were carried out. Equation (1) $c_a = \alpha c$, indicates that the adhesion is directly related to the cohesion of the clay. Golder and Leonard(3) suggested that the α value should be 0.7 for piles more than 30 feet long.

M.J.Tomlinson(4) assumed that the adhesion value between the pile and soil(c_a) is equal to the remolded cohesion (c_r), when the piles are loaded soon after driving, or full cohesion(c) when the piles are not loaded until the soil regains its strength by thixotropy.

Meyerhof and Murdock (1953) proposed that the adhesion should be equal to the soil strength of the clay after it has been allowed to be fully softened under zero pressure. A series of laboratory tests(12) were carried out by Meyerhof using direct shear tests. Precast mortar blocks were placed in the lower part and clay specimens in the upper part of the shear box. The tests were performed under undrained conditions. He found that the amount of deformation to mobilize the full skin friction was about one times the shear strength of the clay ($\alpha = 1.0$). The same test had been performed at Ecole Polytechnique, Montreal in 1954 by J.E.Hurtubise and Jean Granger(12), the test results were similar to those obtained by Dr.Meyerhof.

Tomlinson(4) presented the results of his investigations on 56 pile loading tests. The approximate values of adhesion of clay on the piles can be reliably used for calculation of pile bearing capacity. The reduction of the ratio, adhesion/shear strength, with increasing shear strength, as shown below, may have been due to lack of contact between clay and the pile shaft.

ADHESION BETWEEN CLAYS AND PILES
(After Tomlinson)

Pile type	Soil type	Shear strength psf.	Adhesion psf.
Concrete	Soft clay	0 - 750	0 -700
or	Firm clay	750 - 1500	700 -900
Timber	Stiff clay	1500 - 3000	900 -1300

The allowable adhesion value for Winnipeg clay as shown in the Metropolitan Winnipeg Building Code(7-b) is 300 psf for firm clay and 150 psf for soft clay.

2.4 FAILURE PLANES OF FRICTION PILES

There is general agreement that, from the observations of pile pulling tests, a thin layer of cohesive soil adheres to the pile shaft and appears as a coating of soil around the pile. This indicates that the true skin friction at the pile surface is much greater than the soil strength. The failure surface of friction pile is not exactly at the surface of the pile but occurs at some distances beyond the pile shaft in the soil mass, as shown in Figure 3.

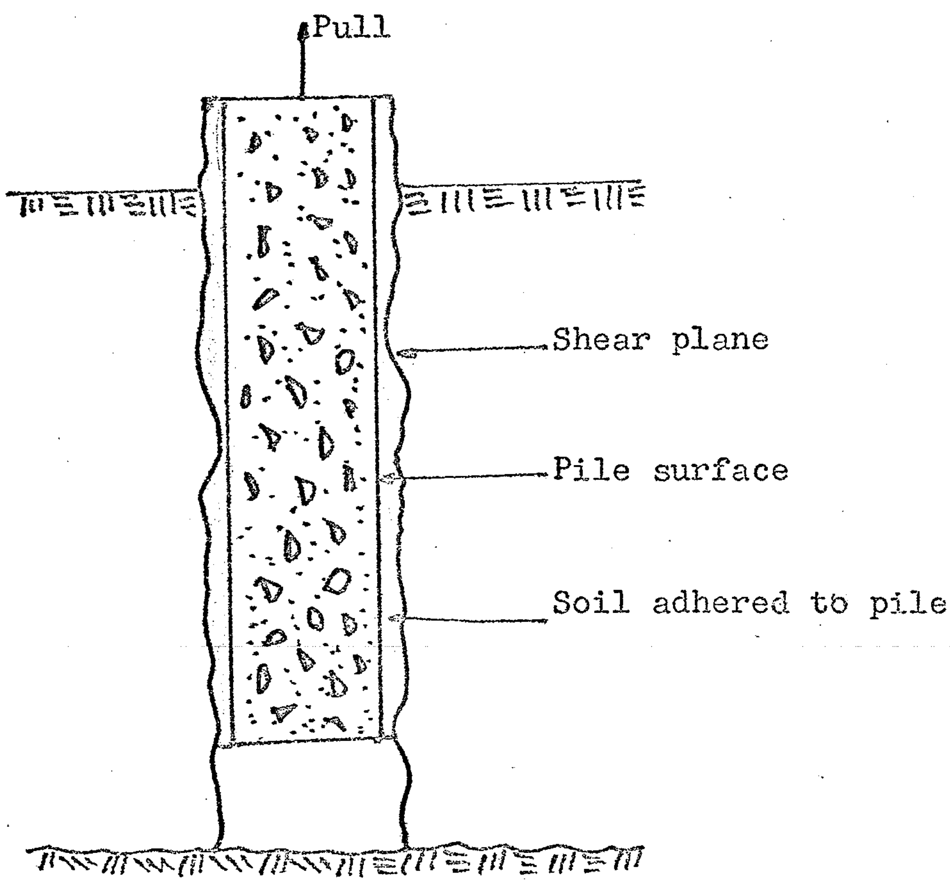


Figure 3.The failure plane of friction piles in cohesive soils (Pull-out tests)

From this point of view, many engineers have quoted that the adhesion value of a cohesive soil, for calculation of pile bearing capacity, should be the same as the soil strength.

2.5 FACTORS AFFECTING FRICTION VALUES

As mentioned before, the friction value between a pile and a soil depends on soil types, pile types, depth of pile, moisture content of the soil, and time after casting of the pile. Friction will increase with increasing grain size, good grading, and with the degree of compaction of a cohesive soil. For fine grained soil, friction will decrease with increasing moisture content in the soil.

Friction and strength of soils may also increase with depth of the pile because of the increase in lateral pressure. A concrete pile with a rough texture gives a greater friction value than the pile with a smooth surface texture as in the case of a steel pile.

In the case of a driven pile, the friction may also be less than the shear strength of the clay because of the lack of contact between the pile and the clay due to lateral whipping of the pile during driving(4). For a cast-in-place concrete pile, the small particles of the concrete mix, such as sand and cement, may be forced into the voids of the soil while the concrete is still fresh. The soil around the pile may become harder than in its natural state, because of possible cementing. During the same time, the moisture content in the soil may be increased by the water from the fresh concrete. The initial softening of the clay, is quite well established, and it seems possible that as the excess water diffuses away into the surroundings, there will be an improvement in the shear strength of the clay in contact with the pile. The shear strength of the clay around the pile is more markedly increased by the effect of hardening of the concrete.

An observation was made in London by W.H.Ward(8-b) on a bored pile which was cast in clay. The clay contained calcium sulphate and the bored pile was made with calcium resisting cement. The pile was exactly one year old when a pit was excavated along its side and the clay carefully removed in a small area. The clay was joined to the pile by a thin film of whitish translucent gel which dried to a white powder on exposure to the atmosphere. The natural rich brown colour of the clay had become faded within ½ inch from the pile, and the clay in this zone was of much higher strength than in its natural state.

This result confirms the statement, as mentioned before (Figure 3), that the failure plane of a friction pile in clay will not occur exactly at the pile shaft, where the strength of the clay is higher than that in the other regions around the pile.

2.6 THE INCREASE IN MOISTURE CONTENT OF THE CLAY DUE TO CAST-IN-PLACE CONCRETE PILES

The increase in moisture content of the clay due to cast-in-place concrete pile may be caused by any or all of the four following causes(14) :

1. Water flows through clay during the process of boring, more markedly in the more fissured clay.

2. Migration of water from the body of the clay towards the less-stressed zones around the bored hole.

3. Water from fresh concrete which usually must be placed at a fairly high W/C ratio.

4. Water from boring equipment.

For cases 1, 2 and 4, the increase of moisture content in the clay may be reduced by technical experience and good workmanship. For case 3, concrete should be placed with relatively dry mixes.

An observation was made in India on bored piles in an expansive clay by Mohan and Chandra(14). They found that the moisture content of the soil adjacent to the pile increased by about 2-3 % above the natural moisture content. It was also noticed that only a thin layer of clay adjacent to the pile was affected and that at a distance of about $1\frac{1}{2}$ inches from the pile shaft, the moisture content was in its lower natural state.

Another observation was made by Meyerhof and Murdock (1953). They found that the moisture content of the clay adjacent to the shaft of a bored pile increased by about 4 % at the contact surfaces and also that at about 3 inches from the pile shaft, the moisture content was not altered.

Time is another factor affecting the friction value of the clay, especially for piles in sensitive clays whose strength decreases to a very large degree by the disturbance during boring and by the pouring of the concrete. After a period of time, when the soil reverts from the remolded state to its natural state, its strength and friction will increase automatically and may be greater than the original strength.

CHAPTER III

LABORATORY INVESTIGATIONS

3.1 TESTING EQUIPMENT

The Direct Shear Box Apparatus using constant rate of strain was employed for the test. This machine is based on the design of Dr.A.W.Bishop, Imperial College of Science and Technology, London. Loads were applied to the specimen by a load hanger, and a shear force was applied by a power driven screw jack at the rate of 0.024 in./min. All samples were fitted in the bronze boxes of size 6x6 cm. and about 1 inch thick. Shear loads were measured with a proving ring suitably calibrated. Figure 4 shows the testing machine.

3.2 MATERIALS :

Materials used for this investigation were :

a. Soil

A sample of soil in the form of an undisturbed block was taken from a depth of 14 feet at the campus of the University of Manitoba.

Some properties of the soil are as follow :-

Average natural moisture content = 56 %

Specific gravity = 2.76

Liquid limit = 73.2 %

Plastic limit = 29.3 %

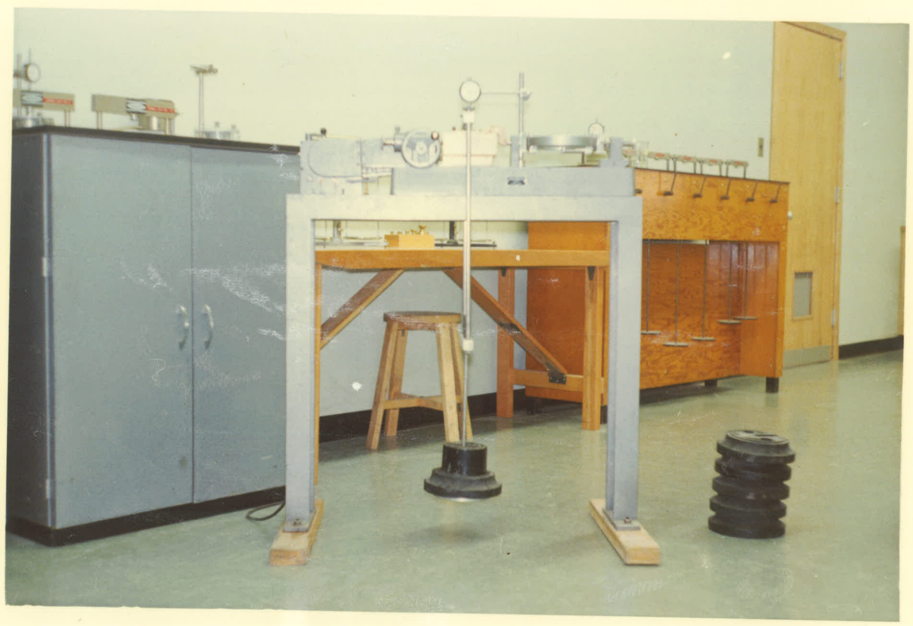


FIGURE 4 THE DIRECT SHEAR TESTING MACHINE

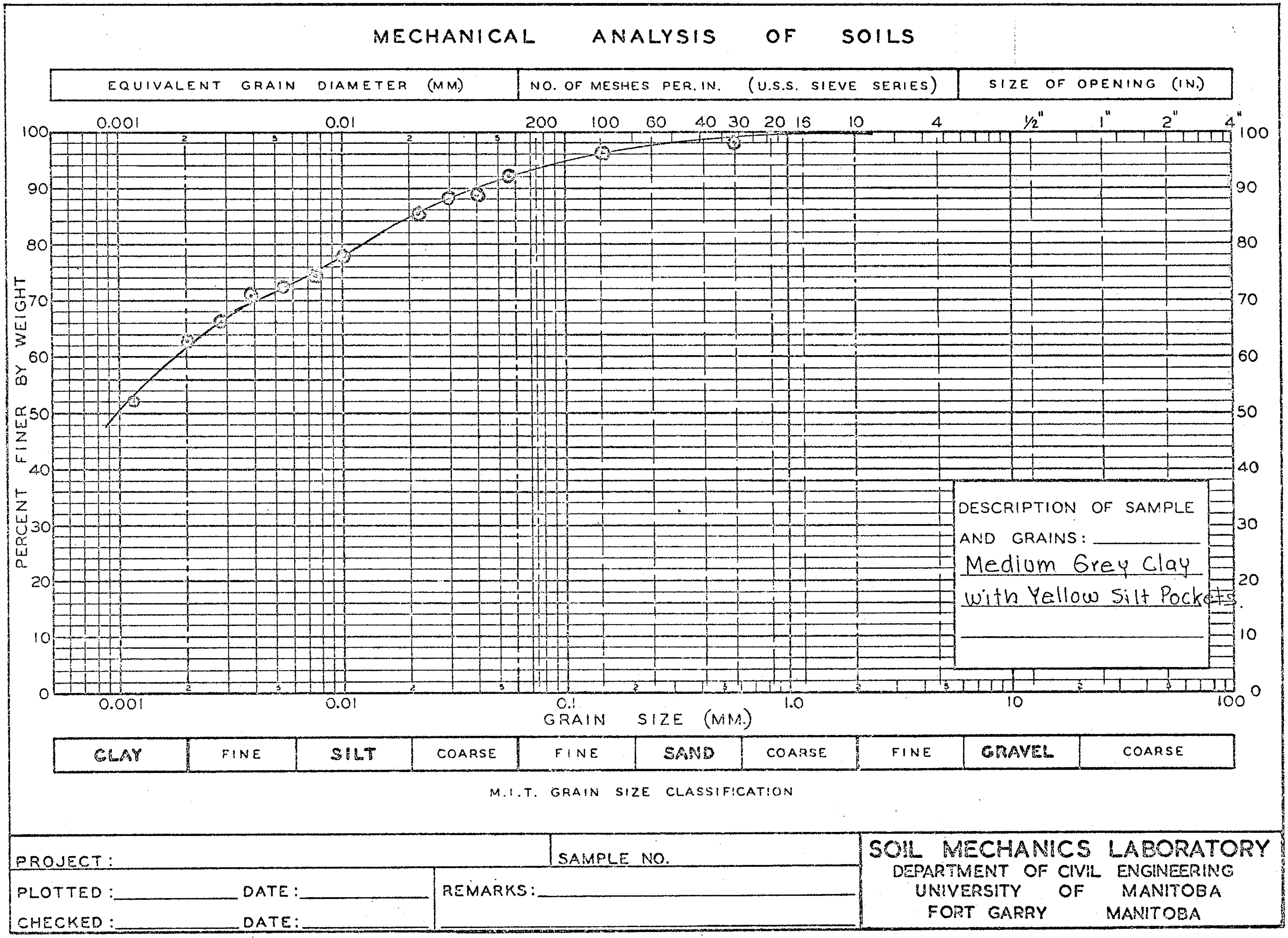

MECHANICAL ANALYSIS OF SOILS
EQUIVALENT GRAIN DIAMETER (MM)
NO. OF MESHES PER. IN. (U.S.S. SIEVE SERIES)
SIZE OF OPENING (IN.)
0.001
0.01
200 100 60 40 30 20 16 10 4
½" 1" 2" 4"
100 90 80 70 60 50 40 30 20 10 0
PERCENT FINER BY WEIGHT
0.001 0.01 0.1 1.0 10 100
GRAIN SIZE (MM.)
DESCRIPTION OF SAMPLE AND GRAINS:
Medium Grey Clay with Yellow Silt Pockets.
CLAY
FINE
SILT
COARSE
FINE
SAND
COARSE
FINE
GRAVEL
COARSE
M.I.T. GRAIN SIZE CLASSIFICATION
PROJECT:
SAMPLE NO.
PLOTTED:
DATE:
CHECKED:
DATE:
REMARKS:
SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

Figure 5

The soil is a highly plastic,medium grey clay with some yellow silt pockets. It is composed of 8 % sand, 30 % silt and 62 % clay. Figure 5 shows the grain size distribution of the soil.

b. Mortar

Mortar used for this investigation was composed of cement and clean sand, 1:3 by weight. W/C ratios of the mixes were 0.45, 0.55 and 0.65

Cement - ordinary Portland cement

Sand - sand with maximum size of 4.76 mm.

3.3 TESTING PROGRAM

The test was separated into three parts :-

1. To determine the shearing strength of the clay at different moisture contents. Normal loads of $\frac{1}{2}$, 1 and $1\frac{1}{2}$ ton/sq.ft. were used. More details are given in the test procedure.

2. To determine the friction value between the clay and cast-in-place mortar block at different W/C ratios, different ages of mortar and different distances of shear planes by applying normal loads of $\frac{1}{2}$, 1 and $1\frac{1}{2}$ ton/sq.ft. on the specimens. More details are also given in the test procedure.

3. To investigate the change of moisture content in the clay specimen due to cast-in-place mortar at different

W/C ratios and different ages of mortar.

3.4 SAMPLE PREPARATION

The soil was cut into small samples, which were big enough for trimming to 6x6 cm. and 1 inch thick. All samples were cut in such a way that they could be sheared along the vertical planes of the sample block, so as to correspond to the pile's movement in the ground. These samples were wrapped in aluminium foil, and covered with wax. They were placed in a storage room where the temperature and humidity were kept constant.

3.5 METHODS OF TESTING

3.5.1 Shearing strength of clay

In order to determine the shearing strength of the clay for different moisture contents, a series of consolidated undrained tests were performed on the soil samples by the direct shear testing machine(shown in Section 3.1). The prepared specimens were trimmed and placed in the shear boxes. Extra shear boxes were made to permit aging of several samples simultaneously. Porous stones were placed at the bottom and on the top of each specimen. The changes of moisture content in the specimens were effected by allowing the specimens to consolidate under different loads (ranges from 0 to 8 psi), for a period of about 24 hours or until the consolidations had been completed as observed on the vertical gauge. They were then sheared under $\frac{1}{2}$ ton/sq.ft. normal load. The rate of shear was 0.024 in./min. The shear load was recorded from the proving ring at every $\frac{1}{2}$ minute interval, until it began to decrease. The test was finished within 10 minutes, so that the moisture content in the clay specimen could not change due to a rapid increase in the normal load. The moisture content after shearing of each specimen, was obtained.

The same method was used for normal loads of 1 and $1\frac{1}{2}$ ton/sq.ft. Seven specimens with seven different consolidation loads were used for each normal load. The test results are given in Table I.

3.5.2 Friction between clay and mortar

This investigation was made while varying the following factors :

W/C ratio = 0.45, 0.55 and 0.65,

T = distance of shear plane from mortar,

= 0, 1/10 and 1/4 inch,

p = normal loads = $\frac{1}{2}$, 1 and $1\frac{1}{2}$ ton/sq.ft.,

Age of mortar = 7 and 14 days.

Note. Samples no.1,2 and 3 (T = 0,1/10 and 1/4 inch), W/C ratio = 0.45, p = $\frac{1}{2}$ ton/sq.ft. and age of mortar = 7 days were used.

The test procedure included the following steps:

1. Sand, cement and water were weighed in sufficient quantity for 3 samples of each mix.

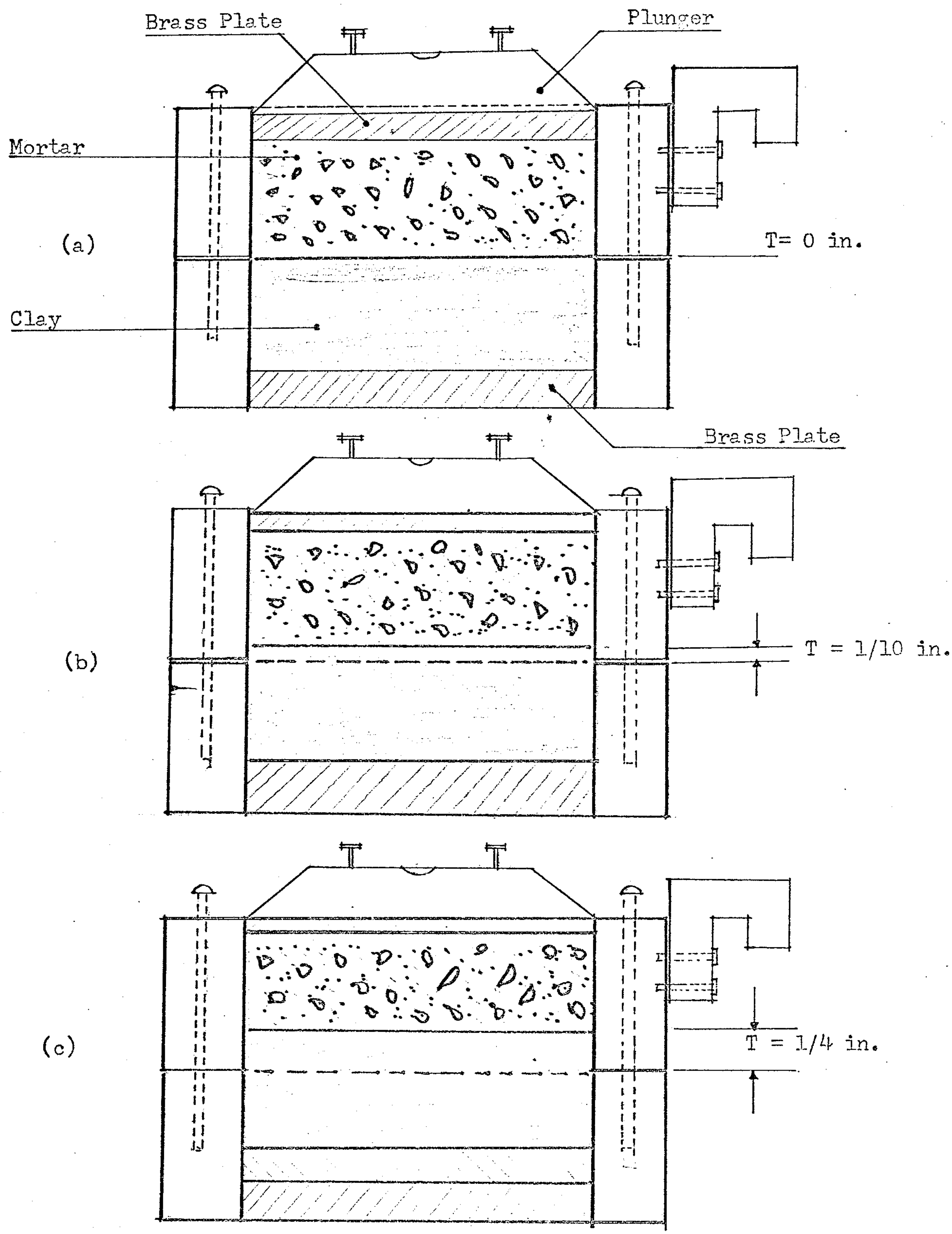
2. Three sets of shear boxes were used for each W/C ratio and each normal load. All samples were tested in the undrained condition. The brass plate, 6x6 cm., with different thickness, were placed on the lower part of the shear box in order to give the required distance of the shear plane from mortar. Sample no.1(T = 0), no.2(T = 1/10 inch) and no.3(T = 1/4 inch), are shown in Figure 6. The edges of the brass plates were coated with grease to prevent the moisture from coming out or going into the sample.

3. The inside of the upper part of each shear box was also coated with grease to facilitate its free movement in the vertical direction when the mortar had hardened.

4. Three prepared specimens were trimmed to 6x6 cm., and 3/4 inch thick. The specimens were put in the lower parts of the shear boxes on a brass plate providing the required distance of the shear plane (as shown in Figure 6). The original moisture content of the clay specimen before placing of the mortar was obtained from a representative piece of each specimen.

5. Mortar was mixed and placed on the top of the clay samples with a thickness of about 3/4 inch. It was tamped in place with a small steel rod.

Figure 6. Positions of Shear Planes



6. The brass plates, coated with grease, and the plungers of the shear boxes were both placed on the top of the mortar. Small edges between the plungers and the shear boxes were sealed with grease or wax. The shear boxes were kept in the moist room for a period of 7 days.

7. After 7 days, these shear boxes were taken out of the room, excess grease or wax removed and the samples were sheared under $\frac{1}{2}$ ton/sq.ft. normal load by the same method as in part 3.5.1 .

8. The moisture content at the shear plane, at the top, at the middle and at the bottom of each specimen was obtained after shearing.

9. The same procedure as above was performed by using W/C ratio = 0.55 and 0.65

The friction value between the clay and mortar under 1 and $1\frac{1}{2}$ ton/sq.ft.normal pressure was also obtained by the same procedure. Nine samples were used for each normal pressure.

The second set of tests were performed by the same method when the age of mortar was 14 days. Fifty-four specimens were used for this investigation and the test results are shown in Table II.

3.5.3 The change of moisture content in the clay specimens

The changes of moisture content in the clay specimens due to cast-in-place mortar were obtained from the second part of this test (Section 3.5.2, no. 4 and no. 8). The difference in moisture content before placing of mortar and after shear was the change in moisture content of the clay specimen. The results of the investigation are given in Table III.

3.6 TEST RESULTS

3.6.1 Shearing strength of clay

The test results for which the maximum shearing strength of the clay and its moisture content after shear were obtained, are shown in Table I (Page 27).

3.6.2 Friction between clay and mortar

Table II (Page 28) shows the results of the friction test between clay specimens and cast-in-place mortar blocks. The maximum friction values and the maximum shear strengths of the clay-mortar at different distances of shear planes from mortar and for different moisture contents at these planes are shown.

Note. T = distance of shear plane from mortar (see Fig.6),
w = moisture content (%) at the shear plane,
p = normal pressure,
s_a = maximum friction value,
s = maximum shearing strength of the clay-mortar.

3.6.3 The change in moisture content of the clay specimen

The change in moisture content of the clay specimens at different distances from the contact surfaces were obtained from the second part of the test. The increase and decrease of moisture content in clays at different ages of mortar and different W/C ratios are shown in Table III, where :

Change in moisture content = The difference in moisture content of the clay specimen before placing of mortar and after shear.

Minus sign (-)= Decrease in moisture content.

Plus sign (+)= Increase in moisture content.

Δw_1= Change in moisture content at the top of the clay.(0 in. from mortar).

Δw_2= Change in moisture content at the middle of the clay (3/8 in. from mortar).

Δw_3= Change in moisture content at the bottom of the clay (3/4 in. from mortar).

Note. The sample numbers are the same as in Table II.

TABLE I

MAXIMUM SHEARING STRENGTH OF THE CLAY

Sample no.	Normal pressure ton/sq.ft.	Moisture content % dry weight	Max.shearing strength psf.
C-1	½	51.2	707
C-2	"	55.0	630
C-3	"	55.5	578
C-4	"	59.1	532
C-5	"	62.0	460
C-6	"	63.5	485
C-7	"	65.9	398
C-8	1	50.0	865
C-9	"	52.0	850
C-10	"	53.0	800
C-11	"	54.0	700
C-12	"	57.0	725
C-13	"	59.5	635
C-14	"	62.0	595
C-15	1½	52.4	960
C-16	"	54.4	890
C-17	"	56.7	919
C-18	"	57.0	830
C-19	"	59.5	825
C-20	"	59.5	818
C-21	"	64.2	700

TABLE II

FRICTION BETWEEN CLAY AND MORTAR

Sample no.	W/C ratio	P tsf.	T in.	w %	s_a, s psf.	Age of mortar days
CM-1	0.45	½	0	51.0	1215	7
CM-2	"	"	1/10	50.4	811	"
CM-3	"	"	1/4	50.6	648	"
CM-4	0.55	"	0	57.0	1120	"
CM-5	"	"	1/10	60.0	569	"
CM-6	"	"	1/4	60.0	490	"
CM-7	0.65	"	0	57.0	1140	"
CM-8	"	"	1/10	56.1	695	"
CM-9	"	"	1/4	55.4	620	"
CM-10	0.45	1	0	50.4	1290	"
CM-11	"	"	1/10	50.1	950	"
CM-12	"	"	1/4	49.6	831	"
CM-13	0.55	"	0	52.0	1280	"
CM-14	"	"	1/10	52.0	915	"
CM-15	"	"	1/4	52.0	775	"
CM-16	0.65	"	0	57.0	1200	"
CM-17	"	"	1/10	57.7	820	"
CM-18	"	"	1/4	58.0	700	"
CM-19	0.45	1½	0	53.0	1440	"
CM-20	"	"	1/10	54.5	984	"
CM-21	"	"	1/4	55.1	830	"
CM-22	0.55	"	0	52.3	1450	"
CM-23	"	"	1/10	52.0	1038	"
CM-24	"	"	1/4	52.8	905	"
CM-25	0.65	"	0	61.6	1340	"
CM-26	"	"	1/10	58.0	944	"
CM-27	"	"	1/4	61.2	789	"

TABLE II

FRICTION BETWEEN CLAY AND MORTAR (Continued)

Sample no.	W/C ratio	P tsf.	T in.	w %	s_a, s psf.	Age of mortar days
CM-28	0.45	½	0	60.3	1060	14
CM-29	"	"	1/10	59.3	660	"
CM-30	"	"	1/4	61.0	513	"
CM-31	0.55	"	0	62.1	1005	"
CM-32	"	"	1/10	58.1	695	"
CM-33	"	"	1/4	59.1	530	"
CM-34	0.65	"	0	63.3	1005	"
CM-35	"	"	1/10	64.6	594	"
CM-36	"	"	1/4	65.6	440	"
CM-37	0.45	1	0	58.3	1175	"
CM-38	"	"	1/10	56.8	863	"
CM-39	"	"	1/4	57.0	685	"
CM-40	0.55	"	0	60.0	1180	"
CM-41	"	"	1/10	59.1	805	"
CM-42	"	"	1/4	60.8	640	"
CM-43	0.65	"	0	62.5	1160	"
CM-44	"	"	1/10	62.0	780	"
CM-45	"	"	1/4	62.4	660	"
CM-46	0.45	1½	0	57.3	1380	"
CM-47	"	"	1/10	57.6	980	"
CM-48	"	"	1/4	58.0	762	"
CM-49	0.55	"	0	60.3	1340	"
CM-50	"	"	1/10	59.7	930	"
CM-51	"	"	1/4	60.0	780	"
CM-52	0.65	"	0	60.0	1395	"
CM-53	"	"	1/10	60.0	941	"
CM-54	"	"	1/4	60.2	775	"

TABLE III

CHANGE IN MOISTURE CONTENT OF CLAY IN CONTACT WITH MORTAR

Sample no.	W/C ratio	Age of mortar days	Change in moisture content		
			$\Delta w_1\%$	$\Delta w_2\%$	$\Delta w_3\%$
CM-1	0.45	7	-1.00	-0.40	-0.20
CM-2	"	"	-1.10	-0.40	-0.40
CM-3	"	"	-1.40	-0.40	-
CM-10	"	"	-1.10	-0.70	-0.50
CM-11	"	"	-0.90	-0.80	-0.50
CM-12	"	"	-0.80	-0.40	-
CM-19	"	"	-1.00	-0.50	-1.00
CM-20	"	"	-1.00	-0.60	-0.40
CM-21	"	"	-1.20	-0.90	-
	average		-1.06	-0.61	-0.33
CM-4	0.55	"	+1.80	+1.40	+2.00
CM-5	"	"	+0.70	+1.60	+1.70
CM-6	"	"	+1.80	+1.80	+2.60
CM-13	"	"	+1.70	+2.50	+2.80
CM-14	"	"	+1.40	+1.80	+2.10
CM-15	"	"	+1.10	+2.00	+1.90
CM-22	"	"	+1.40	+1.90	+2.10
CM-23	"	"	+0.80	+1.40	+2.80
CM-24	"	"	+1.80	+2.80	+2.90
	average		+1.39	+1.91	+2.32
CM-7	0.65	"	+3.80	+4.30	+4.90
CM-8	"	"	+2.60	+3.50	+3.50
CM-9	"	"	+3.40	+3.80	+4.30
CM-16	"	"	+2.80	+4.50	+4.80
CM-17	"	"	+3.70	+4.40	+5.00
CM-18	"	"	+3.50	+4.00	+4.70
CM-25	"	"	+3.60	+4.20	+4.10
CM-26	"	"	+3.60	+5.00	+5.50
CM-27	"	"	+2.90	+4.10	+4.30
	average		+3.70	+4.20	+4.57

TABLE III

(Continued)

Sample no.	W/C ratio	Age of mortar days	Change in moisture content $\Delta w_1\%$	$\Delta w_2\%$	$\Delta w_3\%$
CM-28	0.45	14	-0.70	-0.50	-
CM-29	"	"	-1.20	-0.60	-0.50
CM-30	"	"	-0.70	-0.50	+0.30
CM-37	"	"	-1.20	-0.80	+0.20
CM-38	"	"	-1.00	-0.90	-0.90
CM-39	"	"	-0.90	-0.80	-0.40
CM-46	"	"	-0.70	-0.50	-0.10
CM-47	"	"	-1.10	-1.00	-0.50
CM-48	"	"	-1.00	-0.90	-0.80
	average		-0.94	-0.72	-0.24
CM-31	0.55	"	+1.10	+1.50	+2.00
CM-32	"	"	+1.00	+1.60	+1.70
CM-33	"	"	+1.40	+1.40	+3.10
CM-40	"	"	+2.00	+2.00	+2.00
CM-41	"	"	+1.10	+1.10	+1.50
CM-42	"	"	+1.40	+2.20	+2.70
CM-49	"	"	+2.40	+2.80	+2.80
CM-50	"	"	+2.40	+2.30	+2.30
CM-51	"	"	+1.30	+1.60	+1.90
	average		+1.46	+1.83	+2.20
CM-34	0.65	"	+3.30	+3.80	+4.30
CM-35	"	"	+2.80	+2.90	+3.60
CM-36	"	"	+3.40	+4.60	+4.80
CM-43	"	"	+4.10	+4.90	+5.20
CM-44	"	"	+3.00	+4.00	+4.10
CM-45	"	"	+4.20	+4.50	+4.70
CM-52	"	"	+3.10	+3.90	+4.10
CM-53	"	"	+3.20	+4.20	+5.70
CM-54	"	"	+2.30	+3.00	+4.00
	average		+3.27	+3.98	+4.50

CHAPTER IV

ANALYSIS OF RESULTS

4.1 SHEARING STRENGTH OF CLAY

The test results from the consolidated undrained direct shear test performed on clay specimens are given in Table I. All samples were allowed to consolidate under a pressure ranging from 0 to 8 psi.

Drainage of water from clay specimens in the direct shear testing machine could not be absolutely prevented,but the clay had a very low permeability and the tests were performed most rapidly after a normal pressure had been applied. These tests were finished within 10 minutes. The drainage of water which might have occurred in the clay due to the instantly increased normal pressure, was ignored.

Strength of clay can be expressed by Coulomb's theory of failure :

$$s = c + p.\tan\phi$$

It is more convenient approach to indicate the properties of any soil by using shear strength parameters, (cohesion c and angle of shearing resistance ϕ), regardless of the magnitude of the normal stress on the shear plane. From c and ϕ value, the shear strength of a soil at any state of normal stress(p), can be easily determined.

The major function influencing the strength parameters of saturated clay is moisture content (15,16). The two main discoveries were :

1. that the cohesion depends only on the moisture content.

2. that the angle of shearing resistance is a soil characteristic which influences the soil strength.

Many previous studies have indicated that cohesion(c), changed noticeably with moisture content, but that the angle of shearing resistance(ϕ) did not.

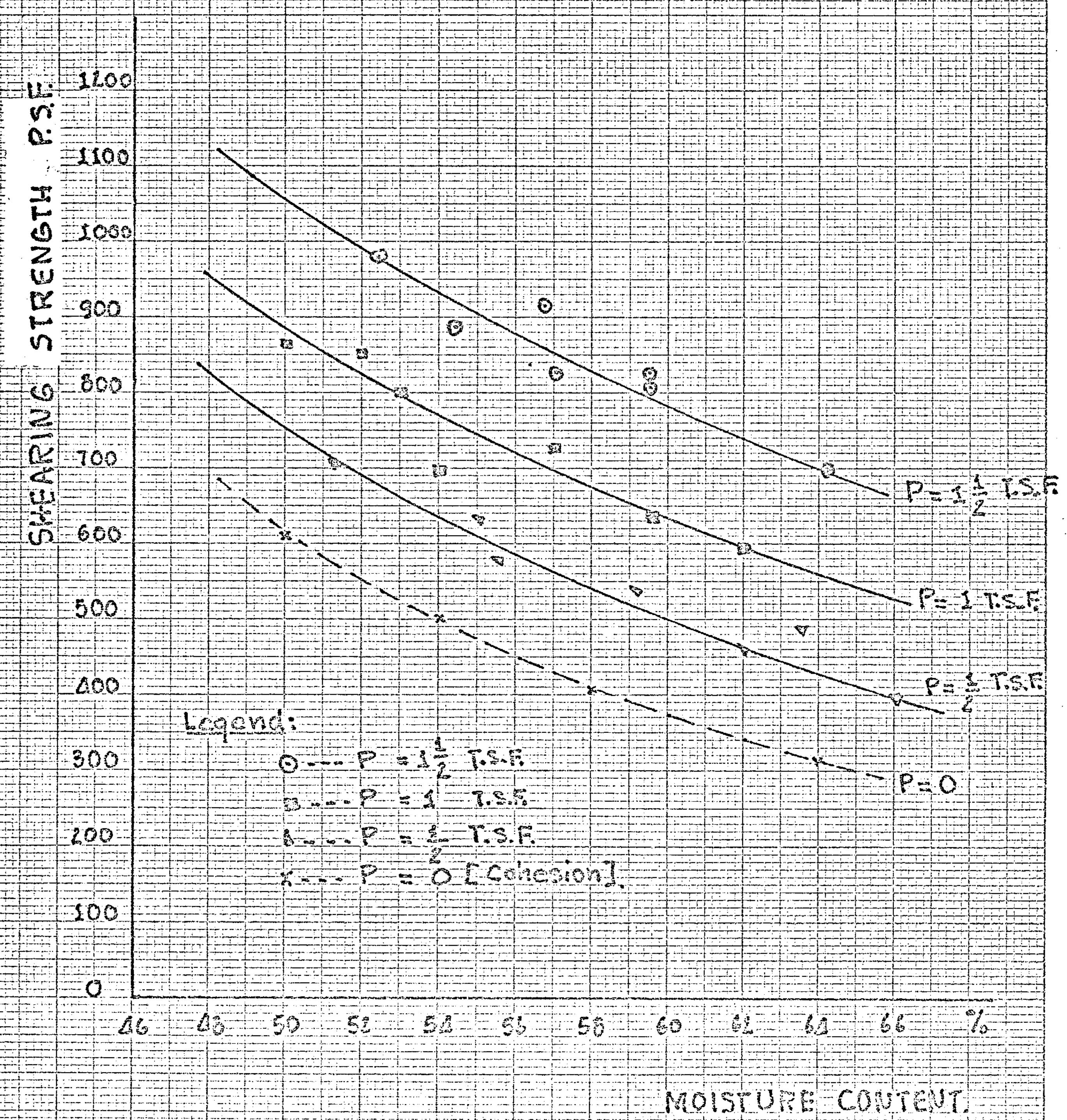
In order to determine the strength parameters (c,ϕ), the Mohr envelope must be constructed. As mentioned before, the shear strength of a cohesive soil will decrease with increasing moisture content in the soil. Strength parameters must be read from the failure envelope for the same moisture content. To do this, the shearing strength of the clay, under different pressures at the same moisture content, must be known. The relationship between moisture content and max-mum shearing strength (obtained from Table I) were plotted on a graph.

Three series of tests, under $\frac{1}{2}$, 1 and $1\frac{1}{2}$ ton/sq.ft. normal pressure, are shown in Figure 7. Three slightly curved lines were drawn to represent the shearing strength of the clay at different moisture contents.

The decrease of the shearing value of the clay with the increase in moisture content was represented as a slightly

FIGURE 7

SHEARING STRENGTH Vs. MOISTURE CONTENT.



curved line. The rate of this decrease was about 24 psf for each percent increasing in moisture content. The shearing strengths, under $\frac{1}{2}$, 1 and $1\frac{1}{2}$ ton/sq.ft. normal pressure for the same moisture content, were obtained from curves in Figure 7 and were plotted in Figure 8.

Figure 8 shows the failure envelopes at 50, 54, 58 and 64 % moisture content.

Shearing strength parameters (c, ϕ), were obtained from these failure envelopes, are given in Table IV.

TABLE IV

VALUE OF COHESION(c) AND ANGLE OF SHEARING RESISTANCE(ϕ)

Moisture content %	Cohesion c-psf.	Angle of shearing resistance ϕ-degree.
50	610	7.5
54	500	7.5
58	400	7.5
64	315	7.5

The decreasing in cohesion with increasing moisture content was very similar to the relationship between shearing strength and moisture content of the clay. Figure 9 shows the cohesion values of the clay at different moisture contents. The angle of shearing resistance(ϕ) did not change with the change in moisture content of the clay.

FIGURE 8.

SHEAR STRENGTH VS. NORMAL PRESSURE.

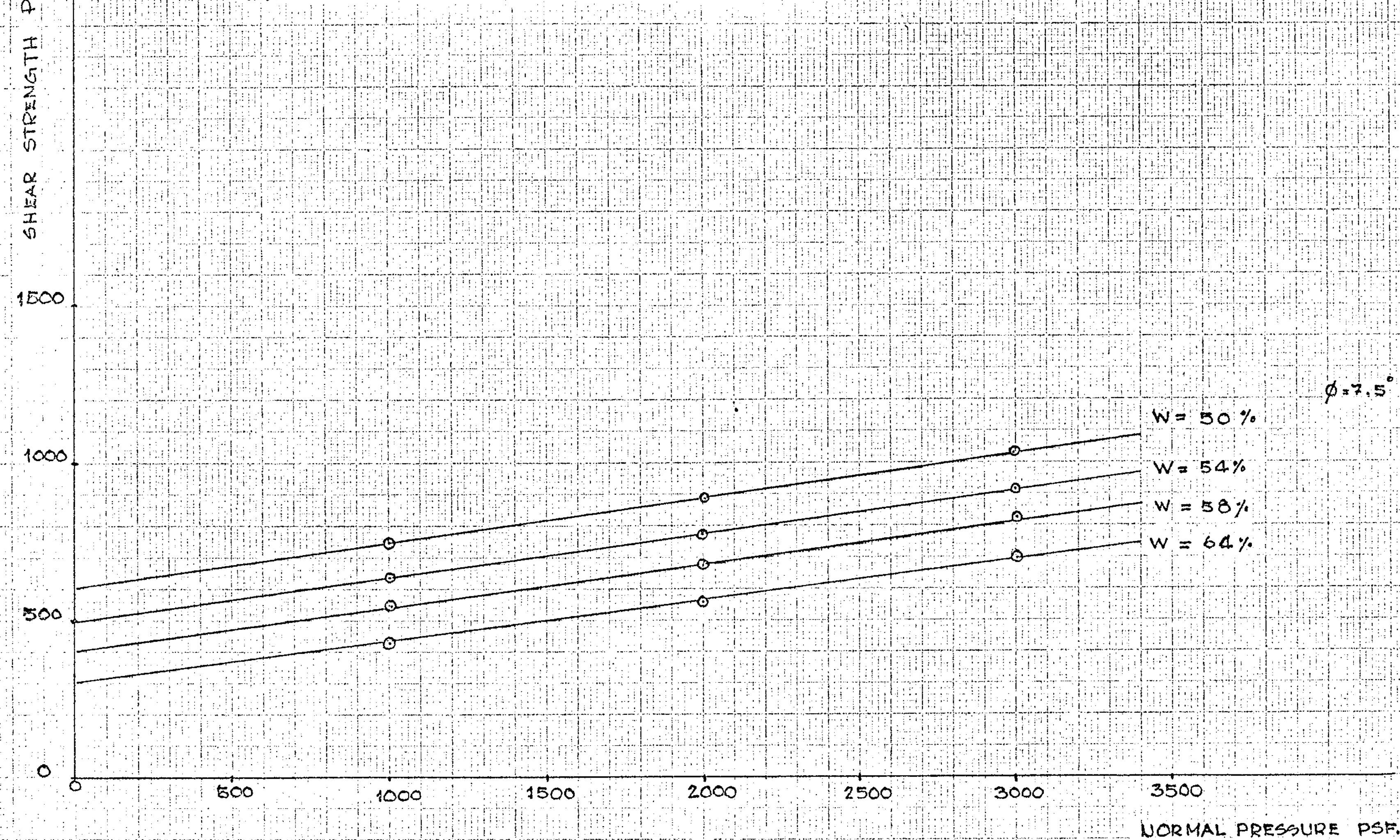
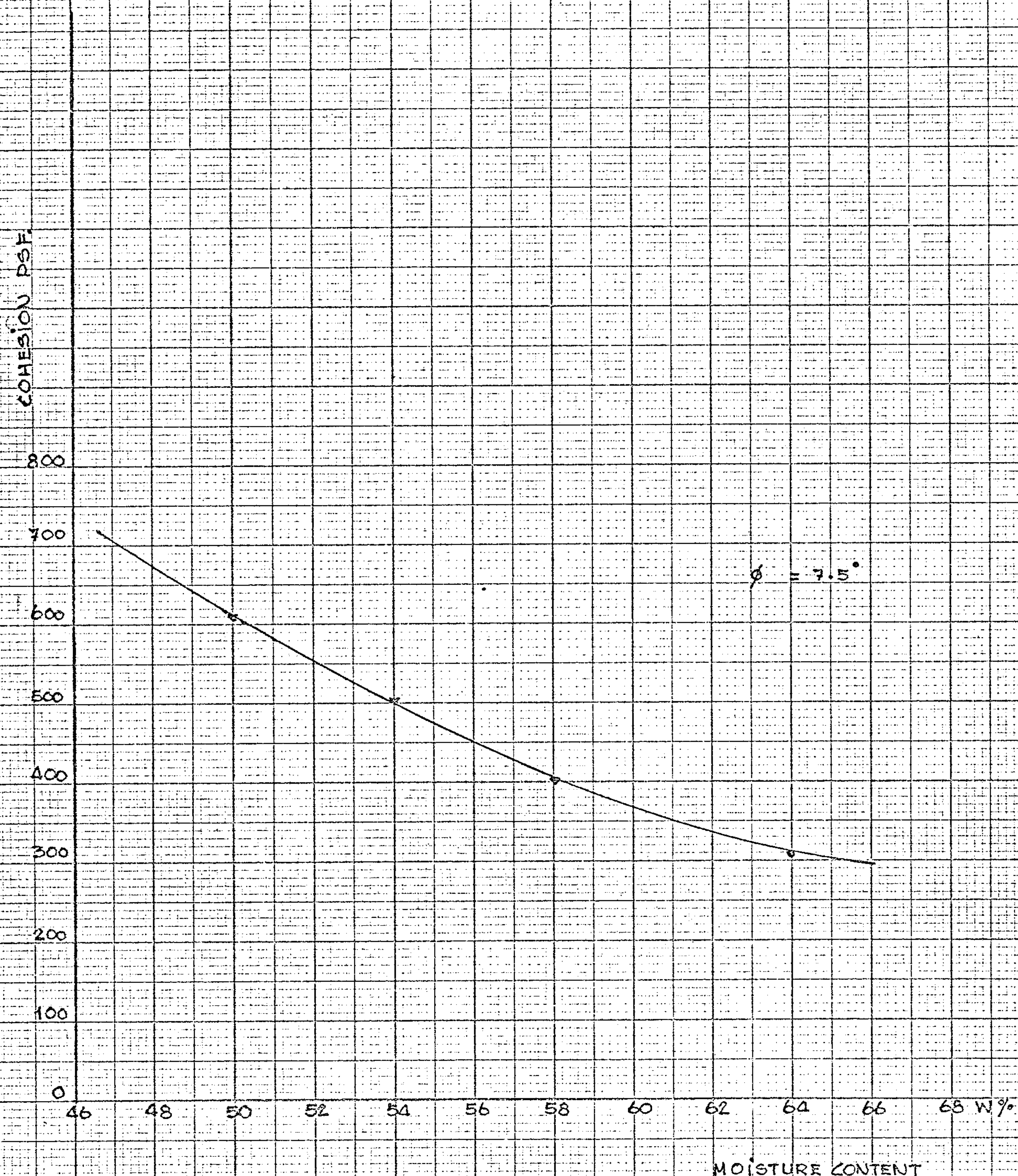


FIGURE 9.

COHESION OF CLAY VS. MOISTURE CONTENT.



4.2 FRICTION BETWEEN CLAY AND MORTAR

In order to investigate the skin friction value and the shearing strength of the clay-mortar at T = 1/10 and 1/4 inch from the contact surface,the relationship between the skin friction values, the shearing strengths and moisture contents at the shear planes, from Table II, were plotted. The skin frictions(at T = 0), and shearing strengths at T =1/10 and 1/4 inch under $\frac{1}{2}$, 1 and $1\frac{1}{2}$ tsf normal pressure are shown in Figures 10, 11 and 12 respectively. Shearing strengths of the clay under the same normal pressures obtained from tests on the clay alone(Table I) were also plotted for comparison.

It will be noted that the strength values for the clay-mortar specimen were usually the same for the 7 and 14 day curing periods.

Results of the test indicated that the clay strength was increased by the effect of cementing of the mortar. Skin friction at contact surfaces(T = 0) was very much higher than the shear strength of the clay-mortar at 1/10 and 1/4 inch. The shearing strength of the clay-mortar decreased with increasing distance from mortar. The skin friction (T = 0) was about 1.7 times the shear strength at T = 1/10 inch and about 2.2 times the shear strength at T = 1/4 inch respectively. The test results indicated that the shear strength of the clay-mortar specimen at T = 1/4 inch from mortar was approximately the same as the natural strength of the clay for the

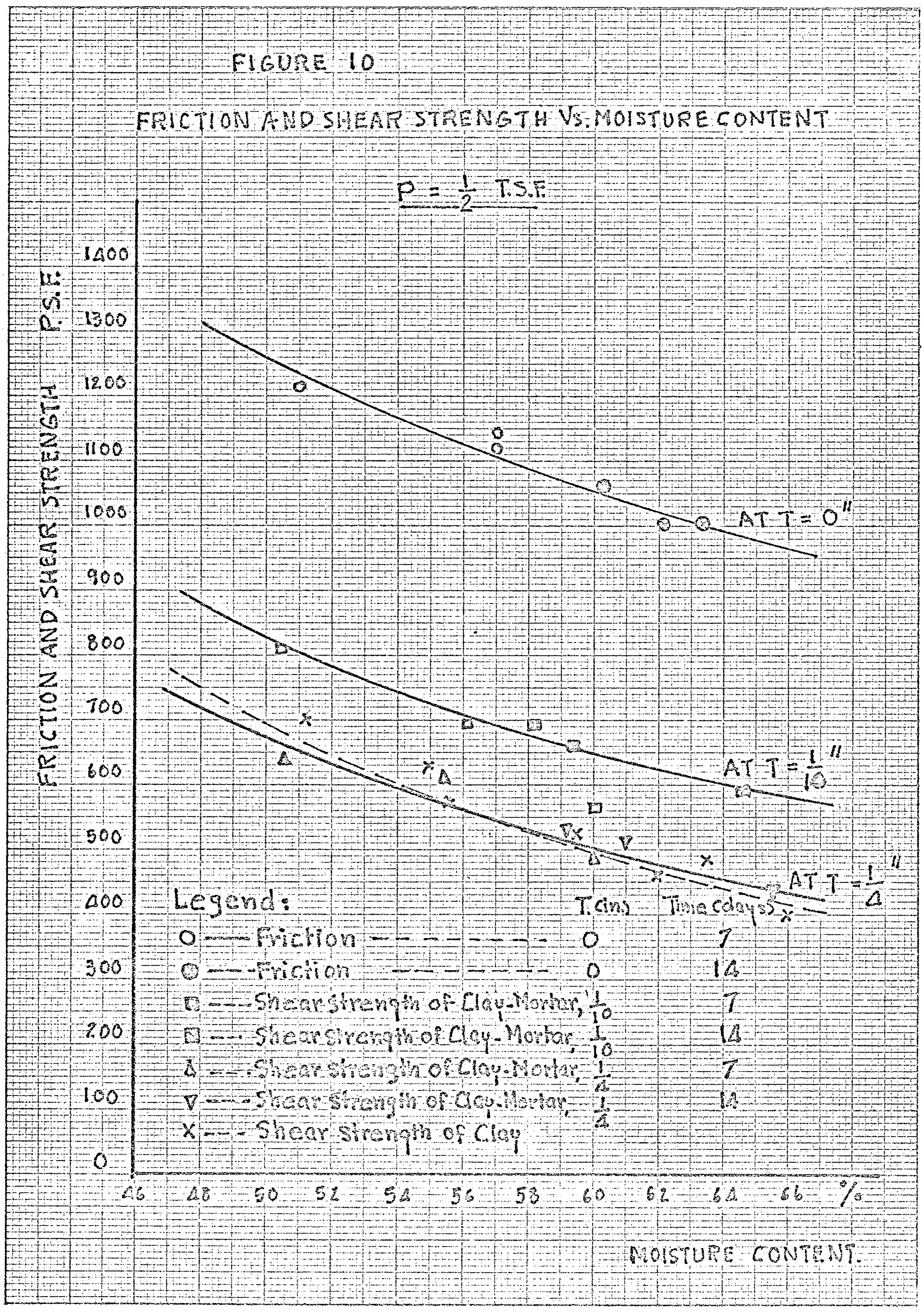


FIGURE 10
FRICTION AND SHEAR STRENGTH Vs. MOISTURE CONTENT
P = 1/2 T.S.F.
FRICTION AND SHEAR STRENGTH P.S.F.
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
46
48
50
52
54
56
58
60
62
64
66
%
MOISTURE CONTENT
AT T = 0"
AT T = 1/10"
AT T = 1/4"
Legend:
T. (ins)
Time (days)
O —— Friction 0 7
◎ —— Friction 0 14
□ ---- Shear Strength of Clay-Mortar, 1/10 7
⊡ ---- Shear Strength of Clay-Mortar, 1/10 14
Δ ---- Shear Strength of Clay-Mortar, 1/4 7
∇ ---- Shear Strength of Clay-Mortar, 1/4 14
X ---- Shear Strength of Clay

FIGURE II

FRICTION AND SHEAR STRENGTH Vs. MOISTURE CONTENT.

P = 1 T.S.F.

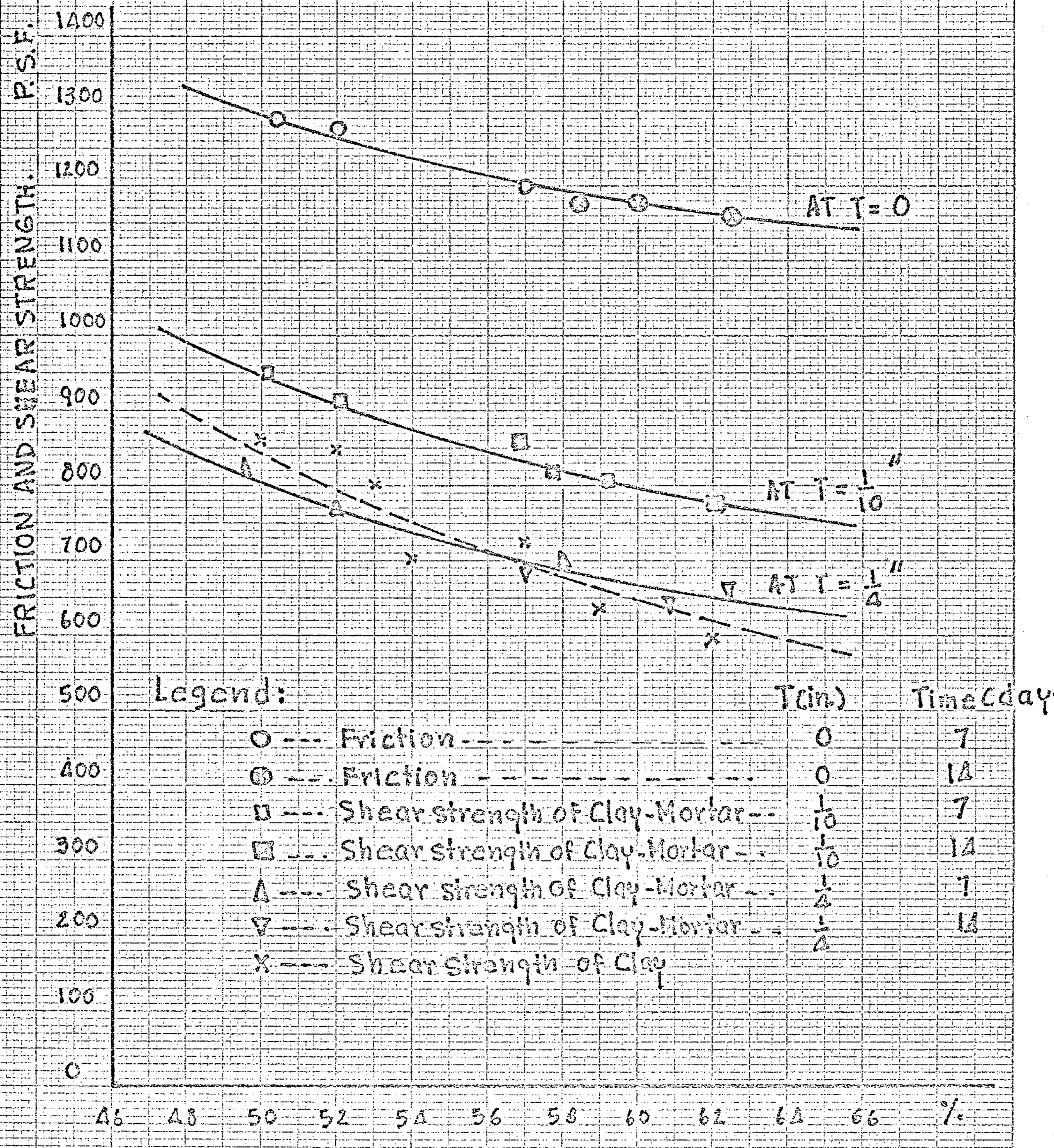
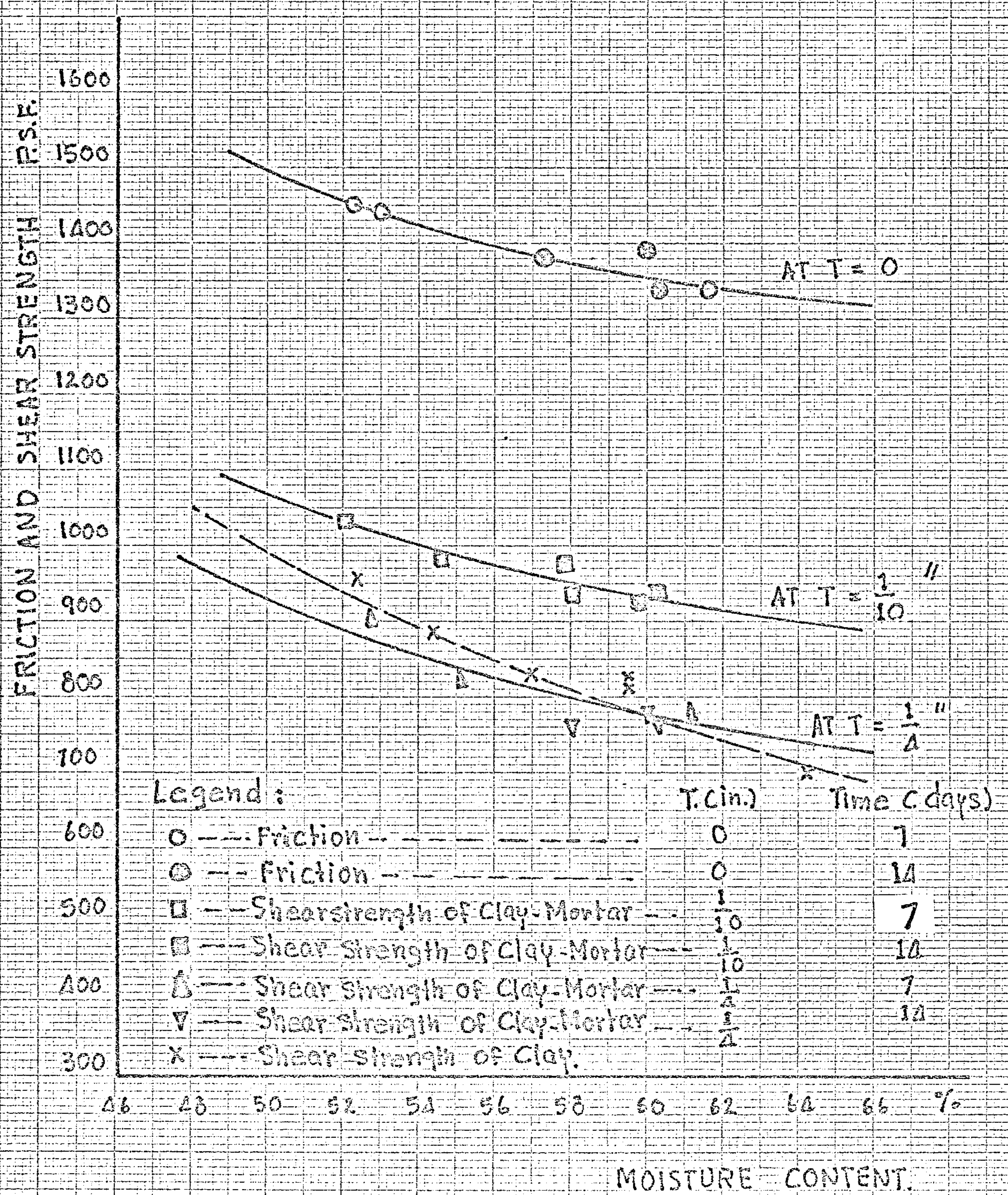


FIGURE 12

FRICTION AND SHEAR STRENGTH Vs. MOISTURE CONTENT.

$P = 1\frac{1}{2}$ T.S.F.



same moisture content.

In order to determine the friction parameters (adhesion c_a and angle of skin friction δ) between the clay and cast-in-place mortar, the failure envelope at the same moisture content must be constructed by the same method of Section 4.1. Frictions and shear strengths (from Table II) for $\frac{1}{2}$, 1 and $1\frac{1}{2}$ tsf normal pressure, and at distances T = 1/4, 1/10 and 0 inch from mortar, were plotted in Figures 13, 14 and 15 respectively. Failure envelopes between friction or shear strength and normal pressure, which were obtained from Figures 13, 14 and 15 at 50, 54, 58 and 64 % moisture content, were plotted in Figures 16, 17, 18 and 19.

All failure envelopes were found to be almost parallel to each other. Therefore the angle of skin friction (δ) did not change with the change in moisture content.

The values of adhesion c_a, and cohesion c for T = 1/10 and 1/4 inch from mortar, and angle δ or ϕ were obtained directly from Figure 16 to Figure 19 and are given in Table V.

TABLE V

VALUES OF c_a, c, δ and ϕ FOR THE CLAY-MORTAR

Distance T-inch.	Moisture content %	c_a or c psf.	Angle δ or ϕ degree
0	50	1080	8.0
"	54	1000	8.0
"	58	940	8.0
"	64	840	8.5
1/10	50	690	7.5
"	54	600	8.0
"	58	520	8.0
"	64	460	8.5
1/4	50	560	7.0
"	54	490	7.0
"	58	415	7.5
"	64	315	7.8

FIGURE 13

SHEAR STRENGTH Vs. MOISTURE CONTENT.

AT $T = \frac{1}{4}$ in.

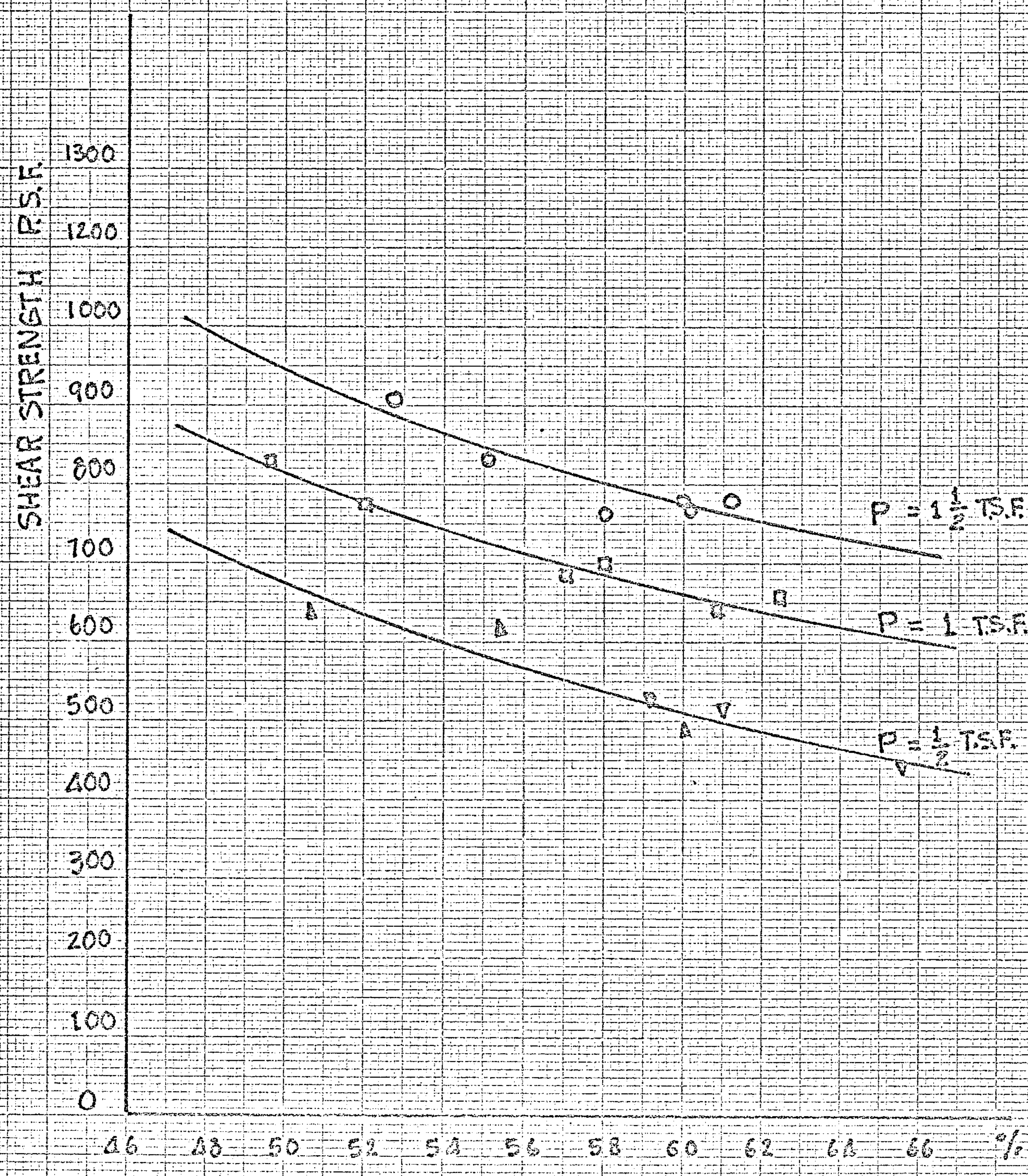
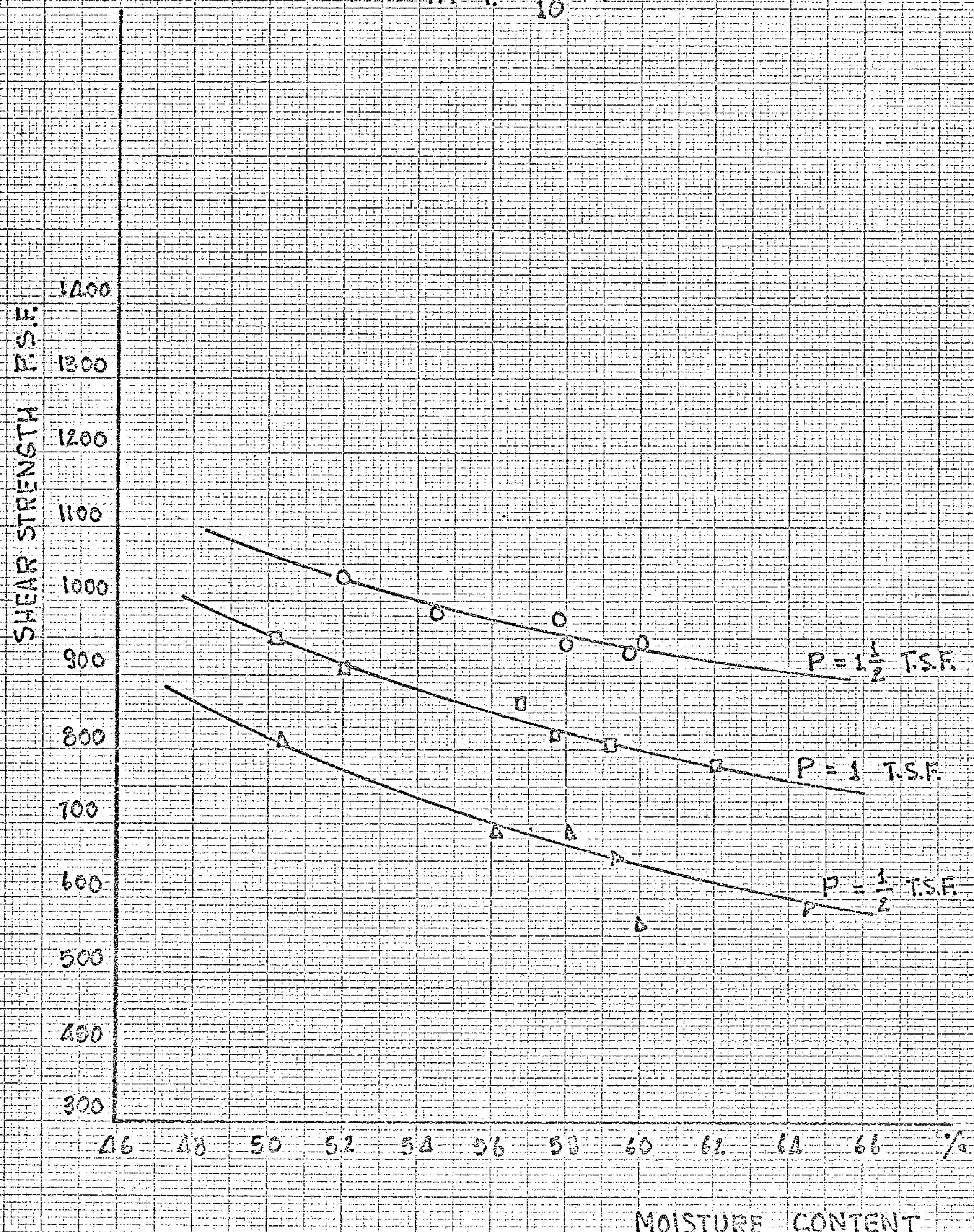


FIGURE 14

SHEAR STRENGTH Vs. MOISTURE CONTENT.

AT T. = $\frac{1}{10}$ in.



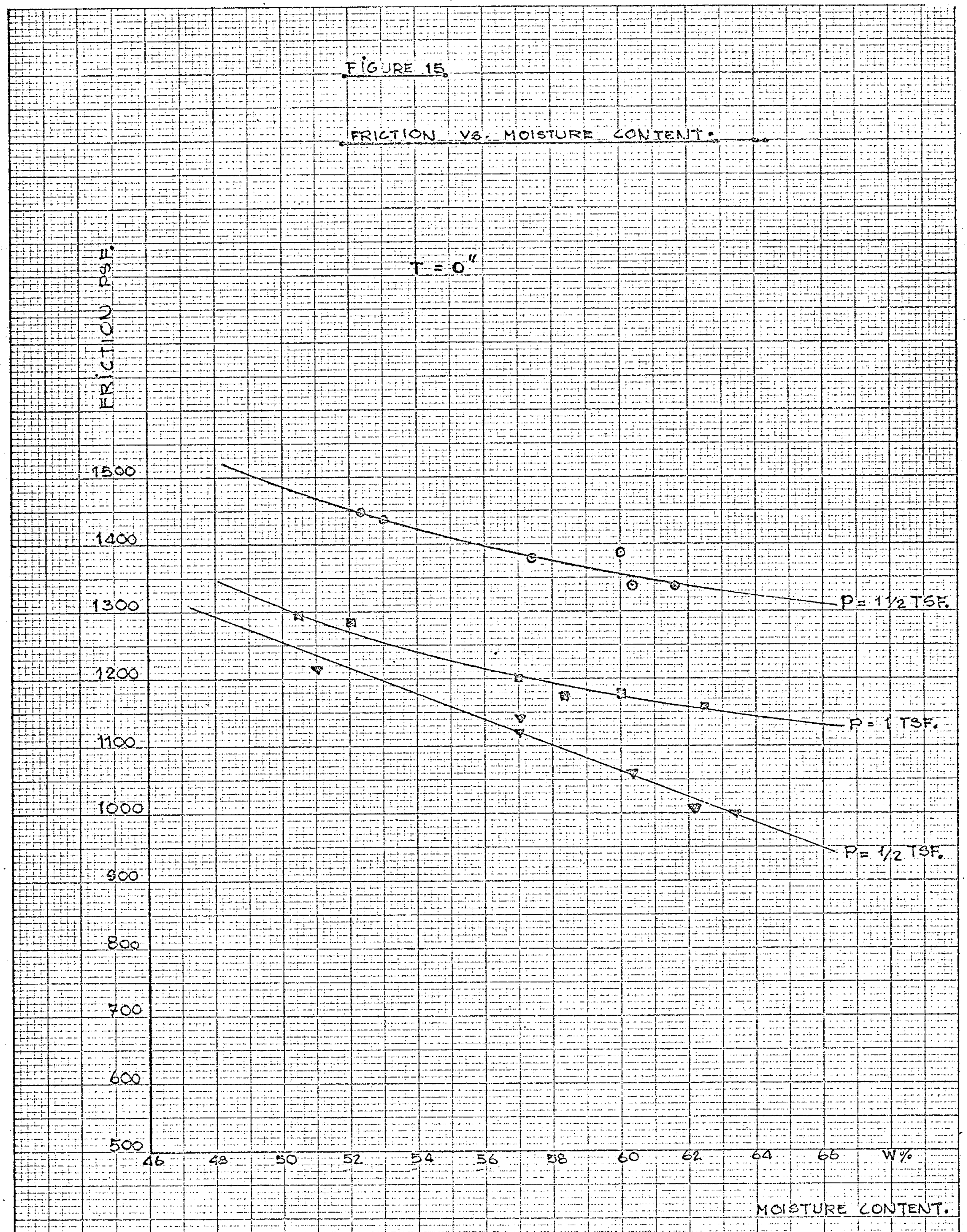


FIGURE 15
FRICTION VS. MOISTURE CONTENT.
T = 0"
FRICTION PSF.
1500
1400
1300
1200
1100
1000
900
800
700
600
500
46
48
50
52
54
56
58
60
62
64
66
W%
P = 1 1/2 TSF.
P = 1 TSF.
P = 1/2 TSF.
MOISTURE CONTENT.

FIGURE 16

SHEAR STRENGTH AND FRICTION Vs. NORMAL PRESSURE

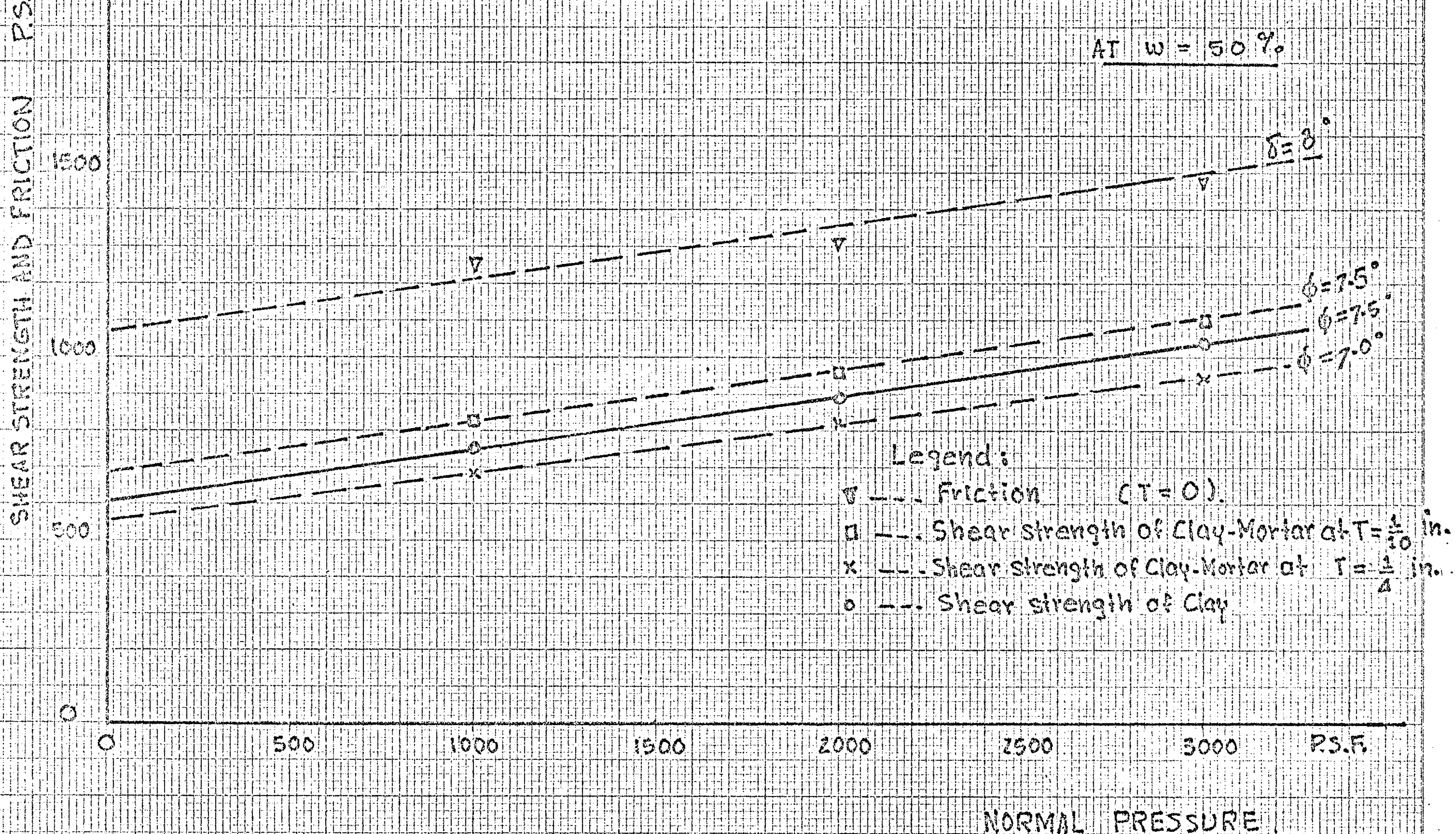


FIGURE 17

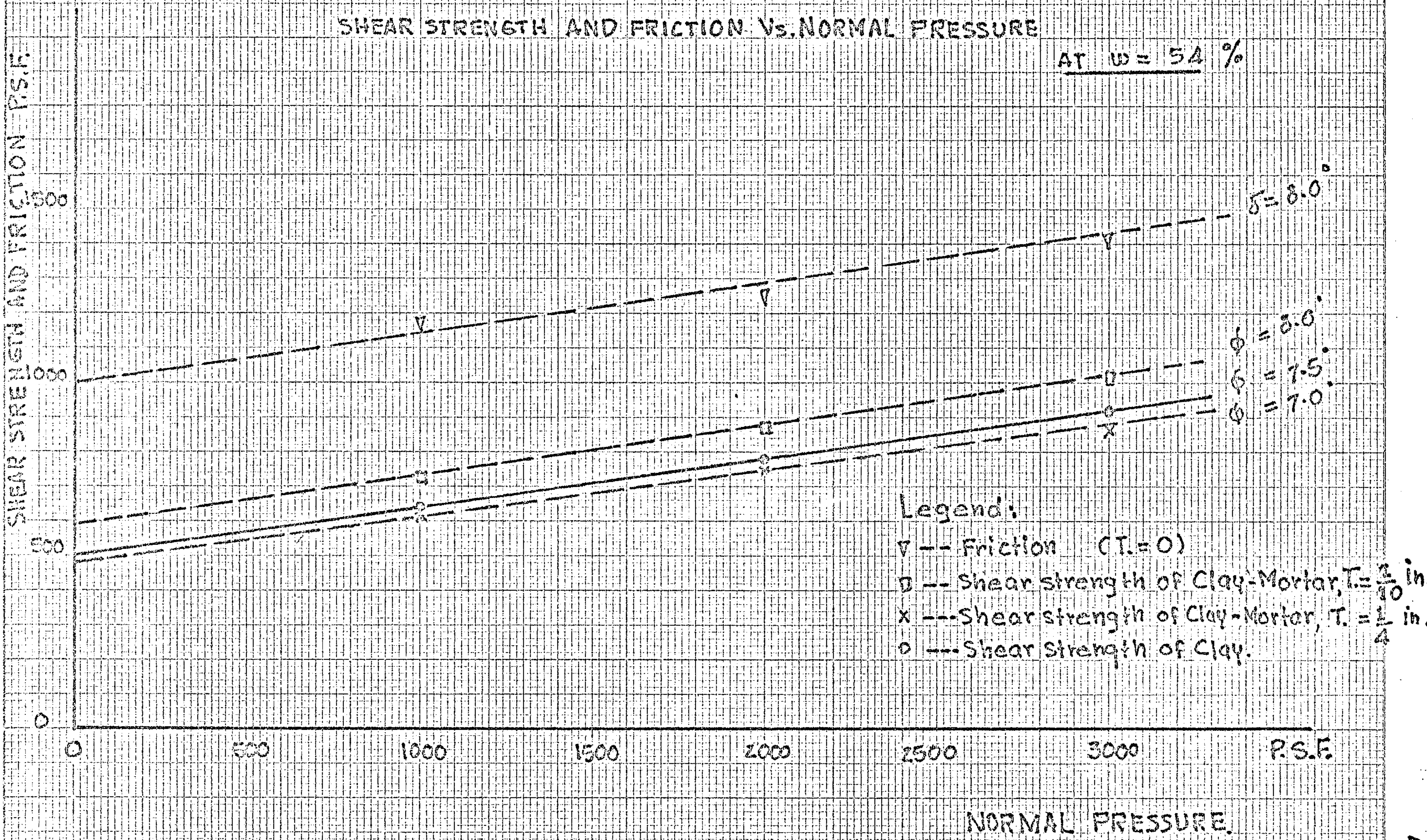
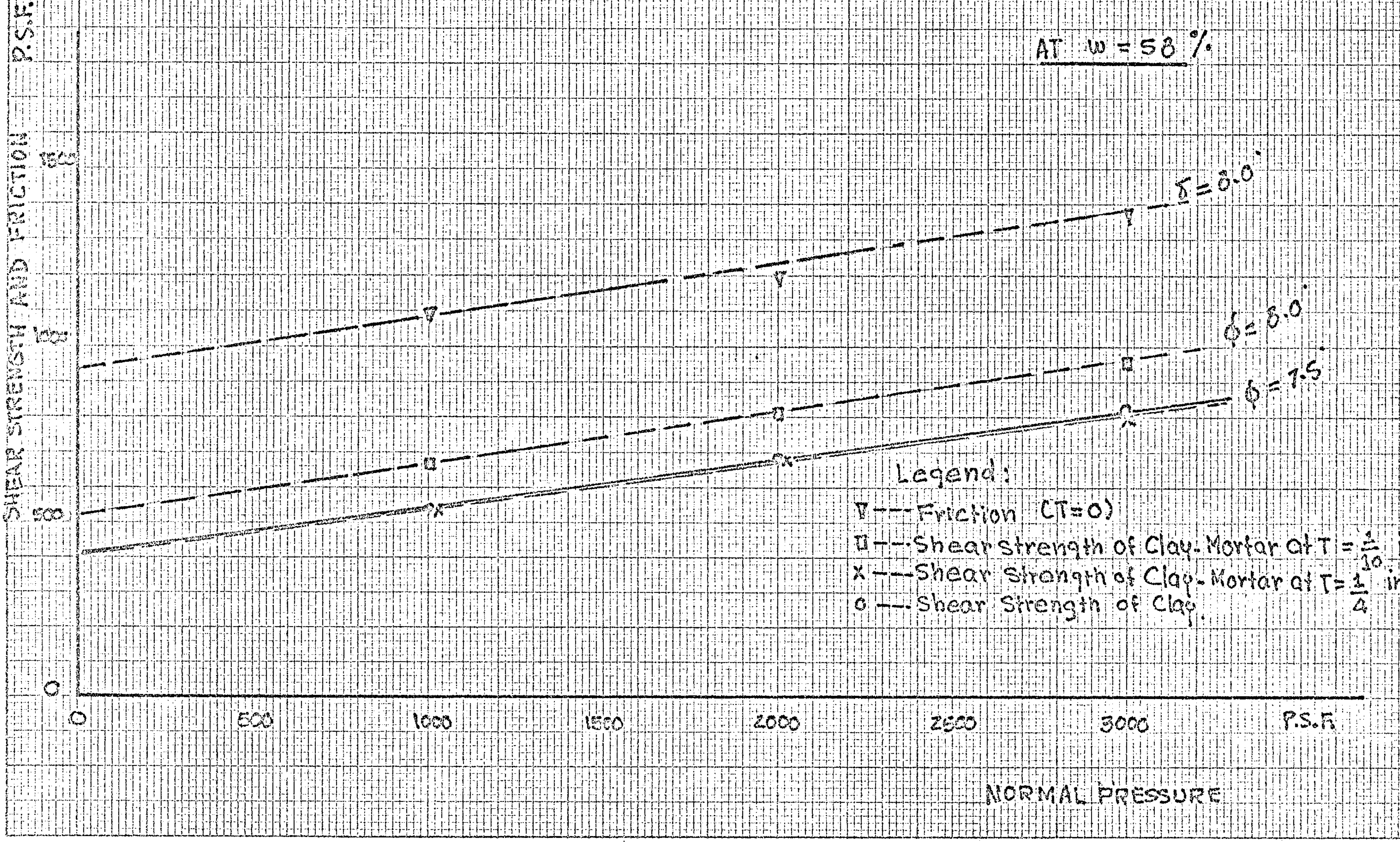

SHEAR STRENGTH AND FRICTION Vs. NORMAL PRESSURE
AT w = 54 %
SHEAR STRENGTH AND FRICTION P.S.F.
1500
1000
500
0
0
500
1000
1500
2000
2500
3000
P.S.F.
NORMAL PRESSURE.
δ = 8.0°
φ = 8.0°
φ = 7.5°
φ = 7.0°
Legend:
▽ -- Friction (T. = 0)
□ -- Shear strength of Clay-Mortar, T. = 1/10 in.
x ---Shear strength of Clay-Mortar, T. = 1/4 in.
o ---Shear strength of Clay.

FIGURE 18

SHEAR STRENGTH AND FRICTION Vs. NORMAL PRESSURE



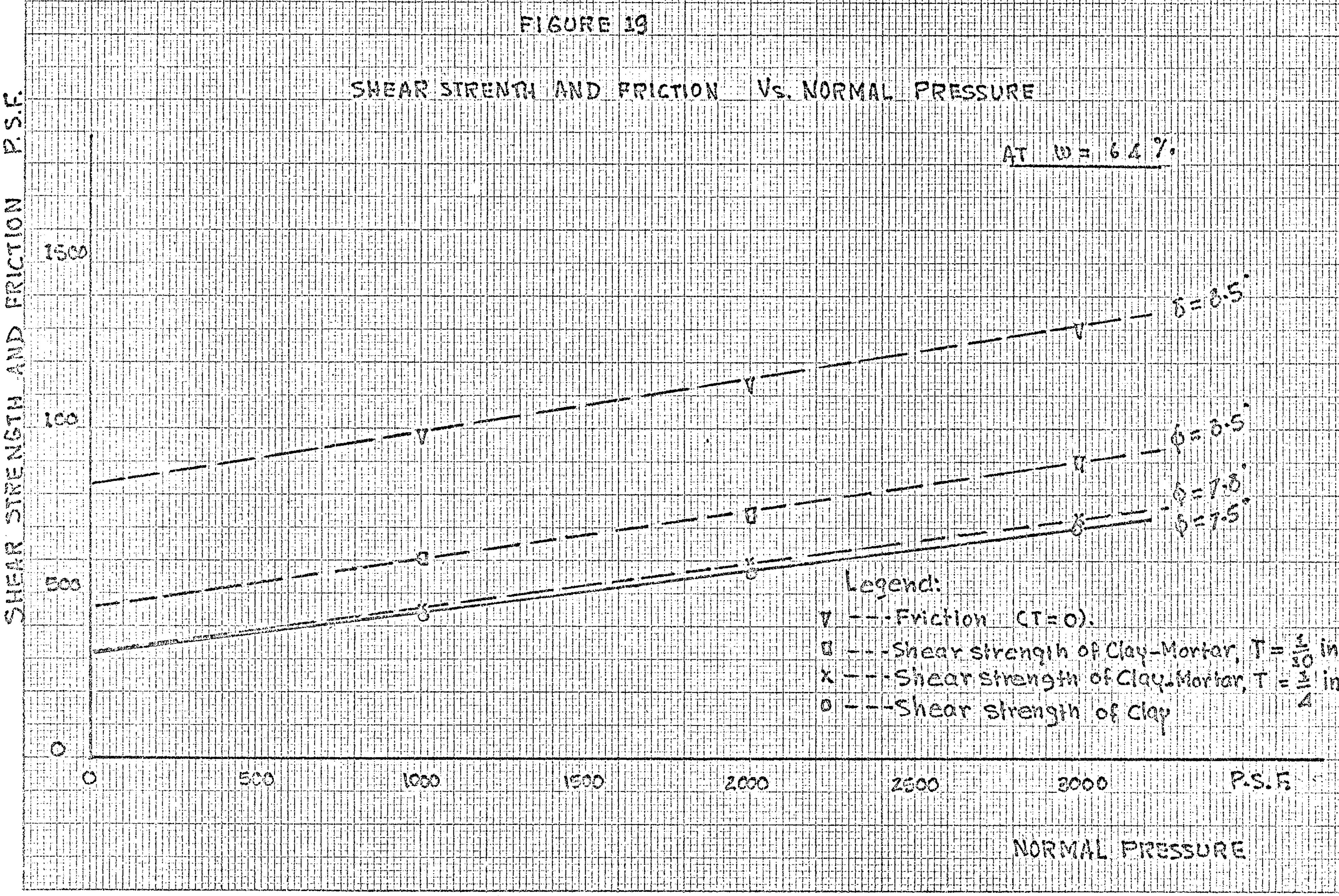


FIGURE 19
SHEAR STRENTH AND FRICTION Vs. NORMAL PRESSURE
AT w = 64 %.
SHEAR STRENGTH AND FRICTION P.S.F.
1500
1000
500
0
0
500
1000
1500
2000
2500
3000
P.S.F.
NORMAL PRESSURE
δ = 8.5°
ϕ = 8.5°
ϕ = 7.8°
ϕ = 7.5°
Legend:
∇ ---Friction (T = 0).
□ ---Shear strength of Clay-Mortar, T = 1/10 in.
x ---Shear strength of Clay-Mortar, T = 1/4 in.
o ---Shear strength of Clay

4.3 COMPARISON OF ADHESION(c_a) AND COHESION(c)

Adhesion values(c_a), cohesion values(c), and moisture contents from Table V were plotted in Figure 20. Cohesion values obtained from tests on clay alone (Table IV), were also plotted for comparison. The changes of the adhesion with the changes in moisture content were similar to that for the cohesion of the clay. All curved lines (Figure 20) were found to be almost parallel to each other. Angles of skin friction were approximately the same ranging from 7.0 to 8.5 degrees.

Let :-

c = cohesion of the clay (tested on clay alone),

$c_{0.25}$ = cohesion of the clay-mortar at T = 0.25 in.,

$c_{0.10}$ = cohesion of the clay-mortar at T = 0.10 in.,

c_0 = adhesion (T = 0).

Then the relationships between c, $c_{0.25}$, $c_{0.10}$ and c_0 which were extracted from Figure 20 can be written as follows :

$$c_{0.25} = c \qquad \text{.....................(9),}$$

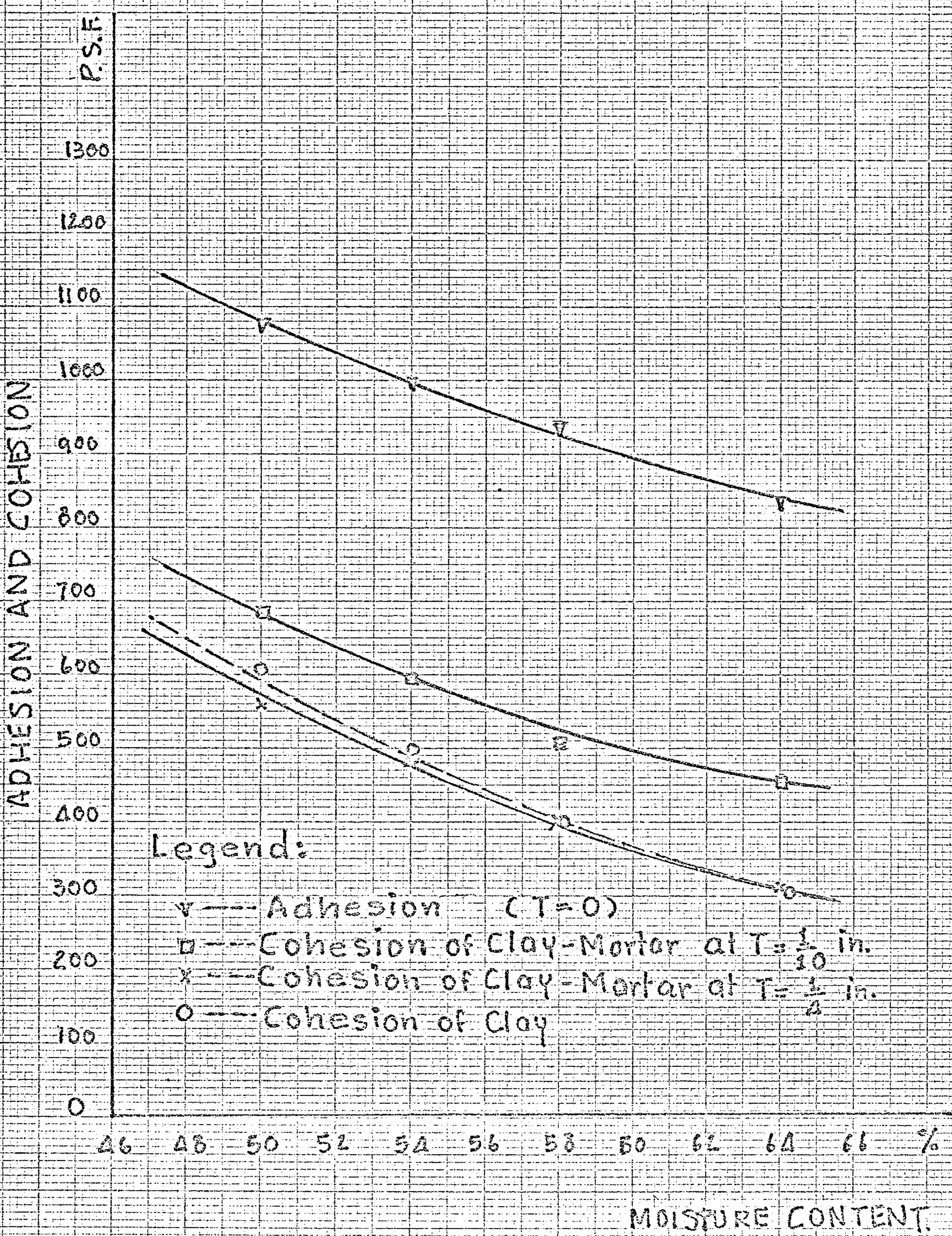
$$c_{0.10} = 1.3c_{0.25} = 1.3c \qquad \text{..........(10),}$$

$$c_0 = 1.7c_{0.10} = 2.2c_{0.25} \qquad \text{.....(11).}$$

It indicated that the coefficient α at the distance of 0.25, 0.10 and 0 inches from mortar were 1.0, 1.3 and 2.2 respectively.

FIGURE 20

ADHESION AND COHESION



4.4 DISTANCE OF FAILURE PLANE (from the test)

R. D. Chellis(2) and R.H. Karol(9), and many other engineers found from the observation of pile pulling tests that, there was a thin layer of cohesive soil adhering to the pile shaft and forming a hard coating of up to about 1 inch thickness as mentioned before in Section 2.4 and as shown in Figure 3. The strength of the soil around the pile was found to be much greater than in its original state due to the effect of the hardening of the concrete.

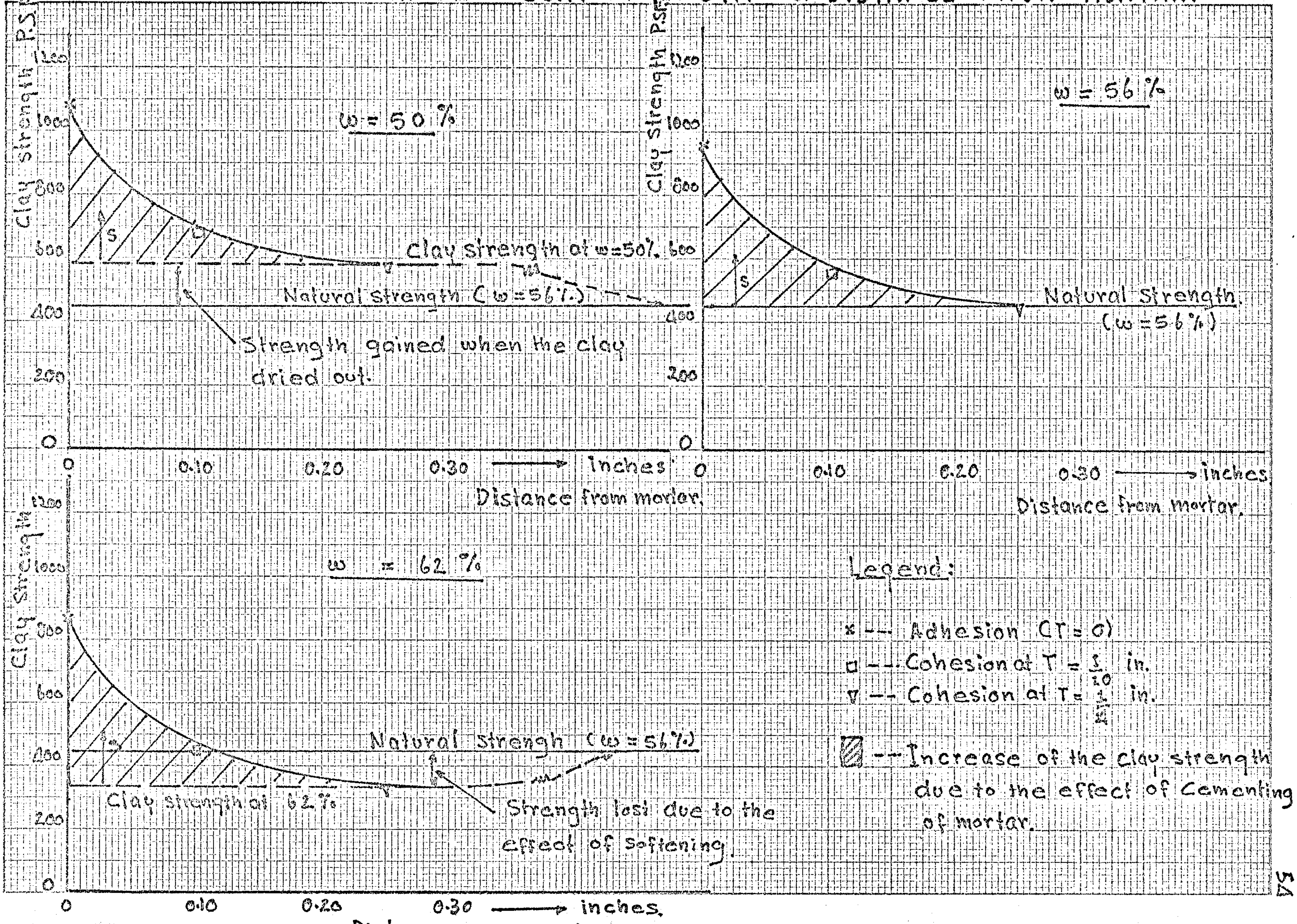
From the laboratory investigation, the aothor found that the clay adjacent to the mortar was a bit brittle and harder than the clay further away from the mortar, confirming the field observations.

Figure 21 shows the effect of cementing of the mortar to the clay. The strength of the clay was greatest at the contact surfaces and it decreased with increasing distance from the mortar. The failure plane will occur where the clay strength is lowest, where the shear strength of the clay-mortar is approximately the same as the natural shear strength of the clay for the same moisture content. This is at a distance approximately 0.23 to 0.28 inches from mortar.

4.5 THE CHANGE IN MOISTURE CONTENT OF THE CLAY DUE TO CAST-IN-PLACE MORTAR

The results of the change in moisture content of the clay are given in Table III. When a dry mix was used, (W/C

FIGURE 21 CLAY STRENGTH Vs. DISTANCE FROM MORTAR.



ratio = 0.45), after a period of 7 days the clay specimens were observed and it was found that the moisture content of the clay decreased by about 1.0 % at the contact surfaces and about 0.3 % at 3/4 inch from mortar. At 0.45 W/C ratio, the mix was very dry. The mortar needed more water for its hydration process, and it absorbed water from the clay. The closer the clay was to the mortar, the more the water from the clay was absorbed.

At the higher W/C ratio (0.55 and 0.65), the mortar was considerably wetter. The amount of water from the mix was sufficient for the hydration process, and the excess water from the hydration penetrated into the clay below the mortar. The clay was softened by the water. Because of the positions of the mortar and the clay in the shear box (see Figure 6), the water penetrated downward to the bottom of the clay specimen. As the results in Table III show the moisture content at the distance of 3/4 inch from the mortar was higher than that the distance of 3/8 inch and at the interface of the clay-mortar. The water could not penetrate any further through the lower part of the shear box, so it caused a bit higher moisture content at the bottom of the clay specimen. The change in moisture content of the clay depended on W/C ratio of the mix.

The relationship between the change in moisture content of the clay and the W/C ratio from Table III were plotted in Figure 22. Figure 22-a shows the change in moisture content of the clay at the age of 7 days after casting of the mortar, and Figure 22-b shows the results when the age of mortar was 14 days. The results at these ages of mortar did not show much differences. The average values of the 7 and 14 days ages of mortar were plotted in Figure 23, and it can be used to estimate the change in moisture content of the clay due to cast-in-place mortar.

The increase in moisture content of the clay around a cast-in-place concrete pile may be due to the water from other sources such as boring equipments and the surrounding area. Construction procedures should be to avoid such water because of the loss of strength caused.

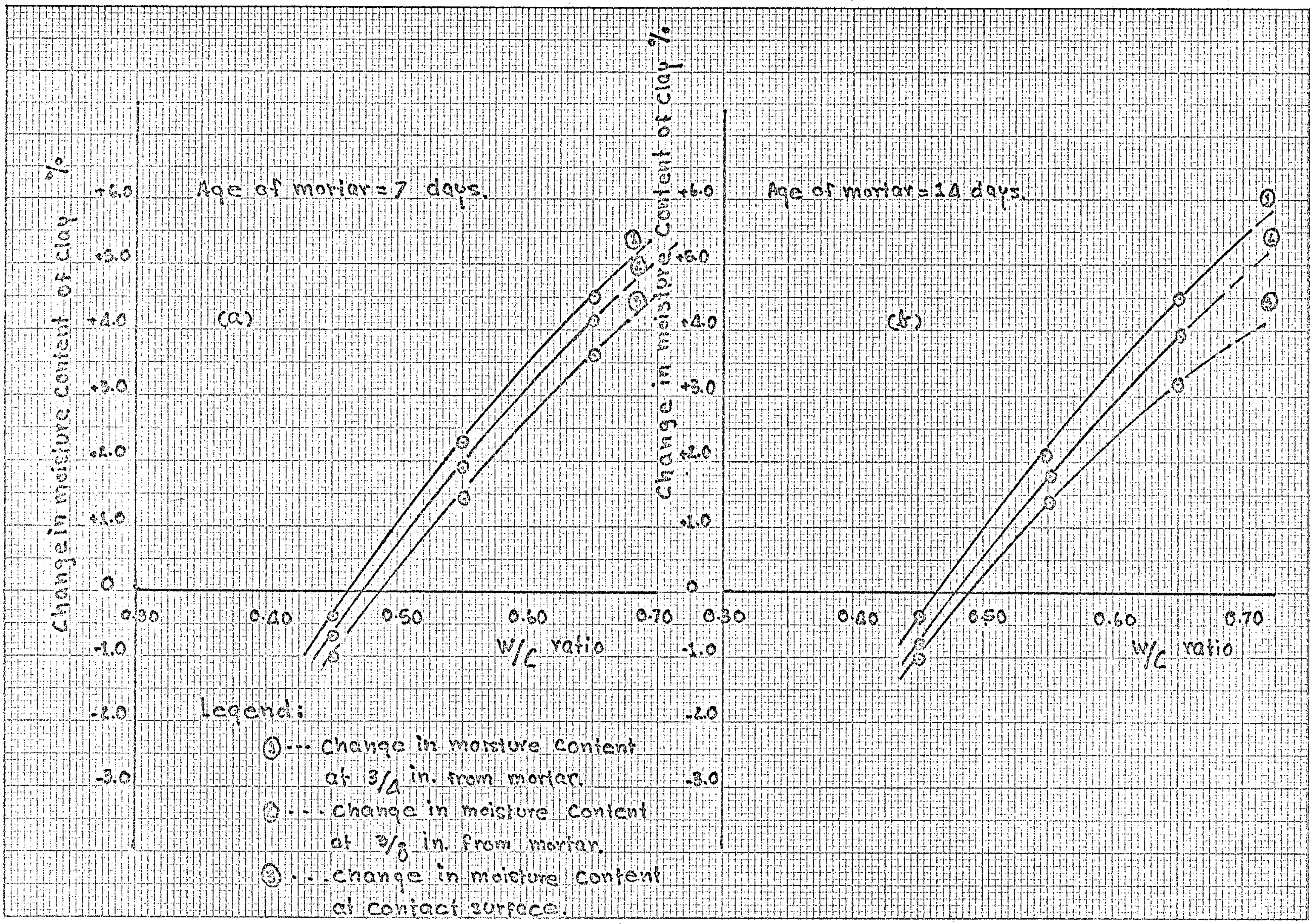


FIGURE 22 CHANGE IN MOISTURE CONTENT IN CLAY.

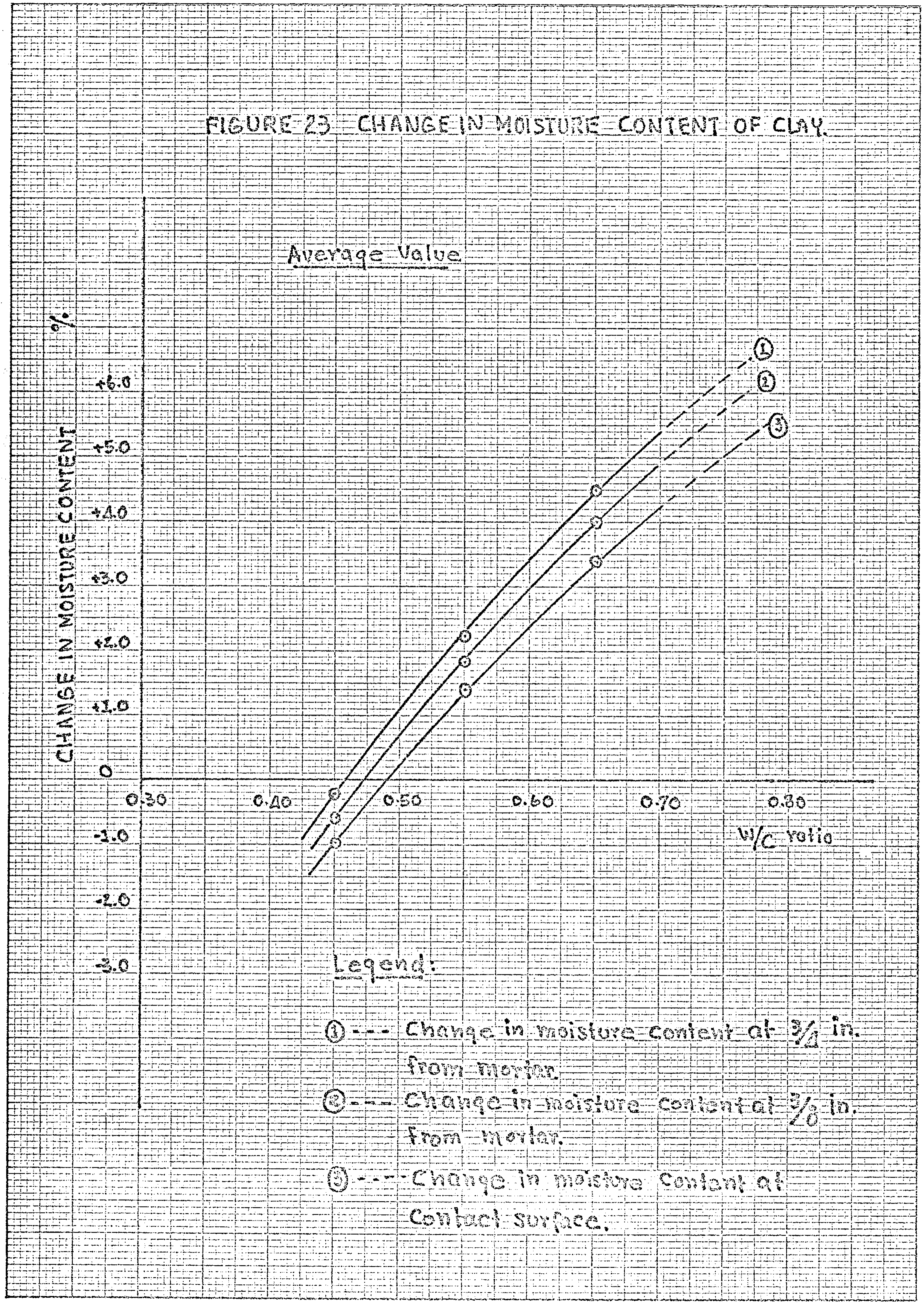


FIGURE 23 CHANGE IN MOISTURE CONTENT OF CLAY.
Average Value
CHANGE IN MOISTURE CONTENT %
+6.0
+5.0
+4.0
+3.0
+2.0
+1.0
0
-1.0
-2.0
-3.0
0.30
0.40
0.50
0.60
0.70
0.80
W/C ratio
①
②
③
Legend:
① --- Change in moisture content at 3/4 in. from mortar.
② --- Change in moisture content at 3/8 in. from mortar.
③ --- Change in moisture content at contact surface.

CHAPTER V

SAFE FRICTION VALUE AND THE COEFFICIENT OF SOFTENING

5.1 SAFE FRICTION VALUE OF THE CLAY

Skin friction capacity (Q_s) of a pile in clay can be calculated from :

$$Q_s = A_s . c_a$$

The test results indicated $c_a = c$, or the friction value was equal to the undrained shearing strength of the clay. In general practice, the c value is equal to one-half of the unconfined compressive strength (q_u) of the clay.

The laboratory results show that the angle of shearing resistance(ϕ) of the clay was about 7.5 degrees. Thus, its shearing strength will increase with increasing normal pressure. In the case of pile foundations, the skin friction capacity of a pile will increase with length of the pile.

The lateral pressures below the ground surface are generally unknown. These pressures depend on the coefficient of earth pressure which is known only for certain special cases. A very conservative approach is to consider the pile bearing capacity, when it is assumed that, the lateral pressure is equal to zero. From equation (2):

$$s_a = c_a + p.\tan\delta$$

When p = 0;

$s_a = c_a, \quad c_a = c$ (test results).

From Figure 7, when $p = 0$, the shearing strength of the clay was 460 psf at its natural moisture content(56 %), and when the clay had been allowed to be fully softened under zero pressure (Section 3.5.1), the moisture content of the clay was about 64 %. Its shearing strength correspondingly reduced to 315 psf. At increasing normal pressure, the shearing strength was increased, that can be seen from Figure 7 :-

At 64 % moisture content :

for $p = 0$, shearing strength = 315 psf,

for $p = 1000$ psf, shearing strength = 430 psf,

for $p = 2000$ psf, shearing strength = 565 psf,

for $p = 3000$ psf, shearing strength = 715 psf.

The cohesion of the clay was 460 psf at its natural moisture content, so that its q_u strength should be about 920 psf. This indicates a clay of medium strength(5).

From the Metro Winnipeg Building Code, the friction value for calculation of safe pile bearing capacity in Winnipeg area, is 300 psf for piles in firm clay, and 150 psf for piles in soft clay.

For a clay of medium strength a friction value of 200 psf may be used.

At 64 % moisture content (fully softened), when $p = 0$, the safety factor is 315/200 = 1.58 .

The bearing capacity of a friction pile in clay will increase with length of the pile, because of the increase of the lateral pressure. The increase of the lateral pressure can be seen from the equation :

$$\sigma h = K_s . \gamma . z \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Where :

σh = lateral earth pressure on the pile shaft,

K_s = coefficient of horizontal to vertical earth pressure,

γ = average density of the clay within depth z.

The magnitude of the lateral earth pressure depends on the K_s value and depth(z) of the pile.

The coefficient of earth pressure K_s depends on the type of soil, on the stress history and on the temporary loads which have been acted on the surface of the soil.

In the case of a cast-in-place concrete pile, the lateral earth pressure of the soil around the pile lies between the active and the at-rest state, because of the expansion of the soil in the vicinity of the hole. The coefficient K_s will be between K_A and K_o, where :

K_A = coefficient of active earth pressure,

K_o = coefficient of earth pressure at-rest.

After a period of time, the soil around the pile, will return to a state of equilibrium and its volume does not change any further. Then the theory of semi-infiniteelastic solid may be applied by assuming the soil is a incompressible material and no volume change.

From :

$$K_o = \frac{\mu}{1-\mu}$$

Where, μ = Poisson's ratio = 0.5, when no volume change,

then, $K_o = 1$.

From equation (12)

$$\sigma h = K_s . \gamma . z$$

Let; $K_s = K_o = 1$

γ = 100 pcf. If a high water table exists, then the effective stress would have to be considered. The value of γ to use in such case would have to be the submerged unit weight.

Consider the safety factor at 64 % moisture content, (from Fig.7) :

If z = 0, lateral earth pressure = 0,

safety factor = 315/200 = 1.58 .

If z = 10 feet, lateral earth pressure = 1000 psf,

safety factor = 430/200 = 2.15 .

If z = 20 feet, lateral earth pressure = 2000 psf,

safety factor = 565/200 = 2.83 .

If z = 30 feet, lateral earth pressure = 3000 psf,

safety factor = 715/2000 = 3.58 .

These results show that the safety factor increases with increasing length of the pile, and after a period of time, when the excess water diffuses away into the surrounding area, the safety factor of the pile in the clay would be considerably increased. This can be seen by considering the safety factor at 56, 60 and 64 % moisture content, that are given in Table VI.

TABLE VI

RATIO OF SHEARING STRENGTH TO "SAFE" FRICTION AT VARIOUS DEPTH

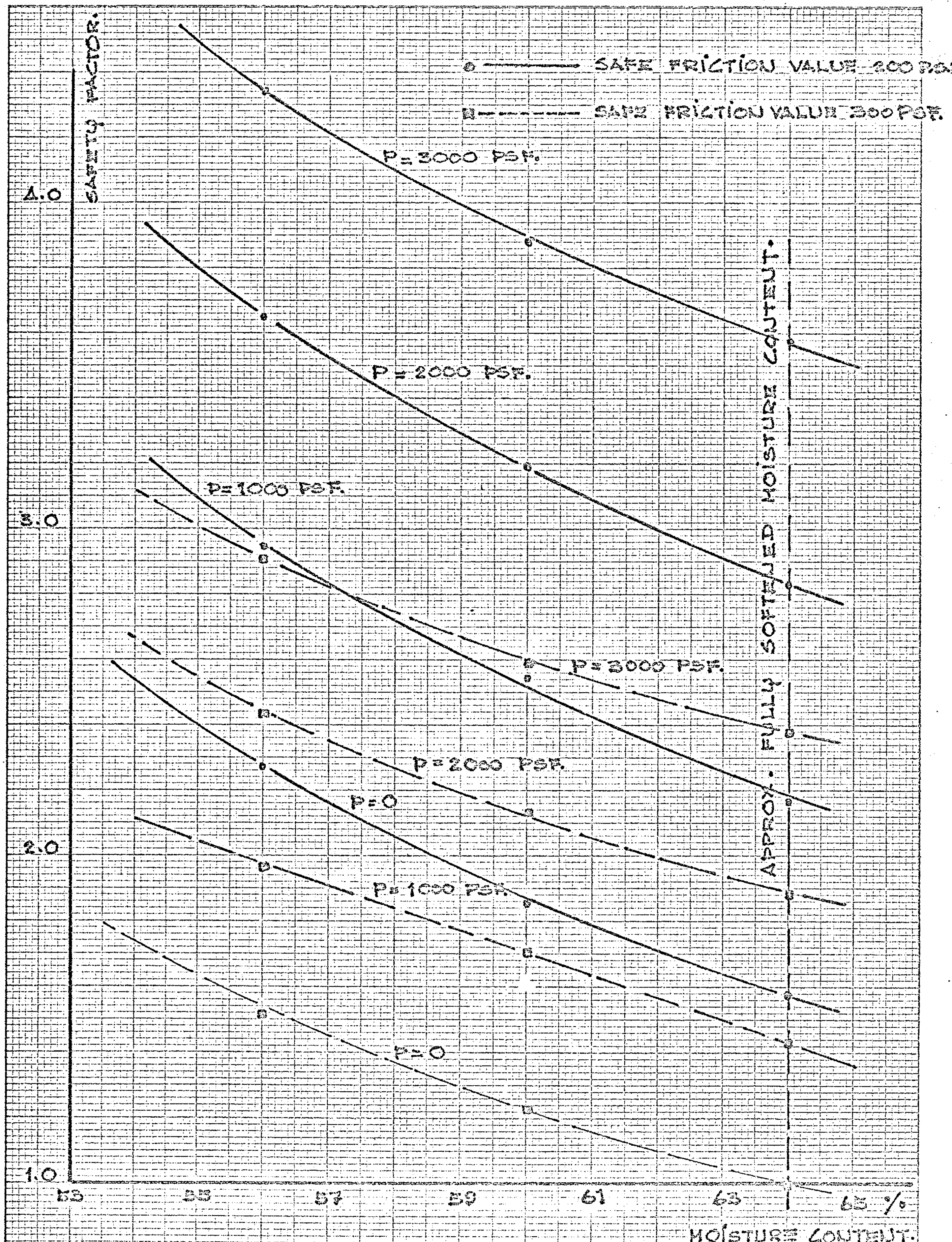
Depth z feet	Lat.pressure p-psf.	Moisture cont. %	Shearing strength psf.	S.F.for safe friction =200 psf.	S.F.for safe friction =300 psf.
0	0	56	455	2.28	1.52
10	1000	"	590	2.95	1.97
20	2000	"	730	3.65	2.43
30	3000	"	870	4.35	2.90
0	0	60	370	1.85	1.23
10	1000	"	510	2.55	1.70
20	2000	"	640	3.20	2.13
30	3000	"	775	3.88	2.58
0	0	64	315	1.58	1.05
10	1000	"	430	2.15	1.43
20	2000	"	565	2.83	1.88
30	3000	"	715	3.58	2.38

Note.

S.F. = ratio of shearing strength to safe friction value,(safety factor).

Figure 24 shows the relationship between the safety factor at various depth and moisture content of the clay.In the case of pile foundations, the safety factor must be the average value.

In general practice, the length of piles would not less



. FIGURE 24 SAFETY FACTORS .

than 20 feet, so that the average safety factor would be 2.20, even though the moisture content of the clay would be very high (64%).

The safe friction value of this clay equal to 200 psf may be used for design practices, so as to give an adequate safety factor of about 2 to 3 for a pile not less than 20 feet long. For a longer pile, a greater safe friction value may be used; say 300 psf.

5.2 COEFFICIENT OF SOFTENING OF THE CLAY (C_s)

The clay will be softened by the excess water from the mortar. The amount of the excess water depends on a number of factors including the W/C ratio. Initially a condition favoring water migration exists depending on whether the soil tends to absorb water from the mortar, or the reverse action when water tends to move from the soil to the mortar. For a given soil and mortar, the results indicate that the softening can be related to the W/C ratio. For different mortars, or concrete, a similar relationship would have to be established. The shearing strength of the clay will be decreased from its natural strength by the effect of the softening.

Let :

Δw = the change in moisture content of the clay due to cast-in-place mortar, %.

w_n = natural moisture content of the clay, %.

w_s = maximum final moisture content of the clay, %.

The maximum final moisture content of the clay can be determined by equation (12),

$$w_s = w_n + \Delta w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(12)$$

Where : Δw can be obtained directly from the curve in Fig.23.

The coefficient of softening (C_s) of the clay can be determined by :

$$K_s = s_s/s_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(13)$$

The values of w_s, s_s, and K_s for different W/C ratios of mortar, are given in Table VII. These values were based on the test results in Figure 7 and Figure 23.

The rate of decreasing of the strength value of the clay with increasing moisture content was about 24 psf for each percent increase in moisture content. Thus, the approximate softened shearing strength (s_s) of the clay can be calculated by the equation (14).

$$s_s = s_n - 24\Delta w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(14)$$

Figure 25 shows the relationship between the coefficient of softening (C_s) and W/C ratio of mortar. From this figure, if the natural shearing strength of the clay (s_n) and W/C ratio of the mortar are known, the softened shearing strength (s_s) can be easily determined by equation (13).

The strength value of the clay for calculation of bearing capacity of a friction pile is generally known in term of "Friction" or "Adhesion" value.

The test results indicated that the friction,or the adhesion value,was actually the clay strength itself, and was equal to the softened shearing strength of the clay after it had been softened by water from the mortar.

In design practice, the more accurate safe friction value of the clay can be estimated from the softened shearing strength (s_s) of the clay, by using the C_s value for each W/C ratio of the concrete mix, under the considerable lateral earth pressure as shown in Figure 25.

Let :

F_s = safe friction value of the clay,

n = required safety factor.

Then :

$$F_s = s_s/n \qquad \text{.........................(15)}$$

The W/C ratio of the concrete is one of the important factors influencing the friction value of the clay. The W/C ratio for 2000 to 3000 psi concrete would range from about 0.50 to 0.60 . The higher W/C ratio, the more, the water penetrates into the clay, and the greater is the clay strength loss. For W/C ratio of 0.45 or less the effect is to dry the clay, but this is usually below the practical limit.

TABLE VII

COEFFICIENT OF SOFTENING (C_s)

W/C ratio	Δw %	$w_s = w_n + \Delta w$ %	s_n psf.	s_s psf.	$C_s = s_s/s_n$	p psf.
0.45	-0.2	55.8	460	460	1.00	0
0.50	+1.0	57.0	"	430	0.94	"
0.55	+2.3	58.3	"	400	0.87	"
0.60	+3.4	59.4	"	380	0.83	"
0.65	+4.5	60.5	"	360	0.78	"
0.45	-0.2	55.8	595	600	1.01	1000
0.50	+1.0	57.0	"	570	0.96	"
0.55	+2.3	58.3	"	545	0.91	"
0.60	+3.4	59.4	"	520	0.87	"
0.65	+4.5	60.5	"	500	0.84	"
0.45	-0.2	55.8	730	735	1.01	2000
0.50	+1.0	57.0	"	708	0.97	"
0.55	+2.3	58.3	"	675	0.92	"
0.60	+3.4	59.3	"	650	0.89	"
0.65	+4.5	60.5	"	620	0.85	"
0.45	-0.2	55.8	875	878	1.00	3000
0.50	+1.0	57.0	"	850	0.97	"
0.55	+2.3	58.3	"	820	0.94	"
0.60	+3.4	59.4	"	798	0.91	"
0.65	+4.5	60.5	"	775	0.89	"

Note.

w_n = 56 % = average natural moisture content of samples tested,

Δw = change in moisture content due to cast-in-place mortar from Figure 23,

s_n, s_s - obtained from Figure 7.

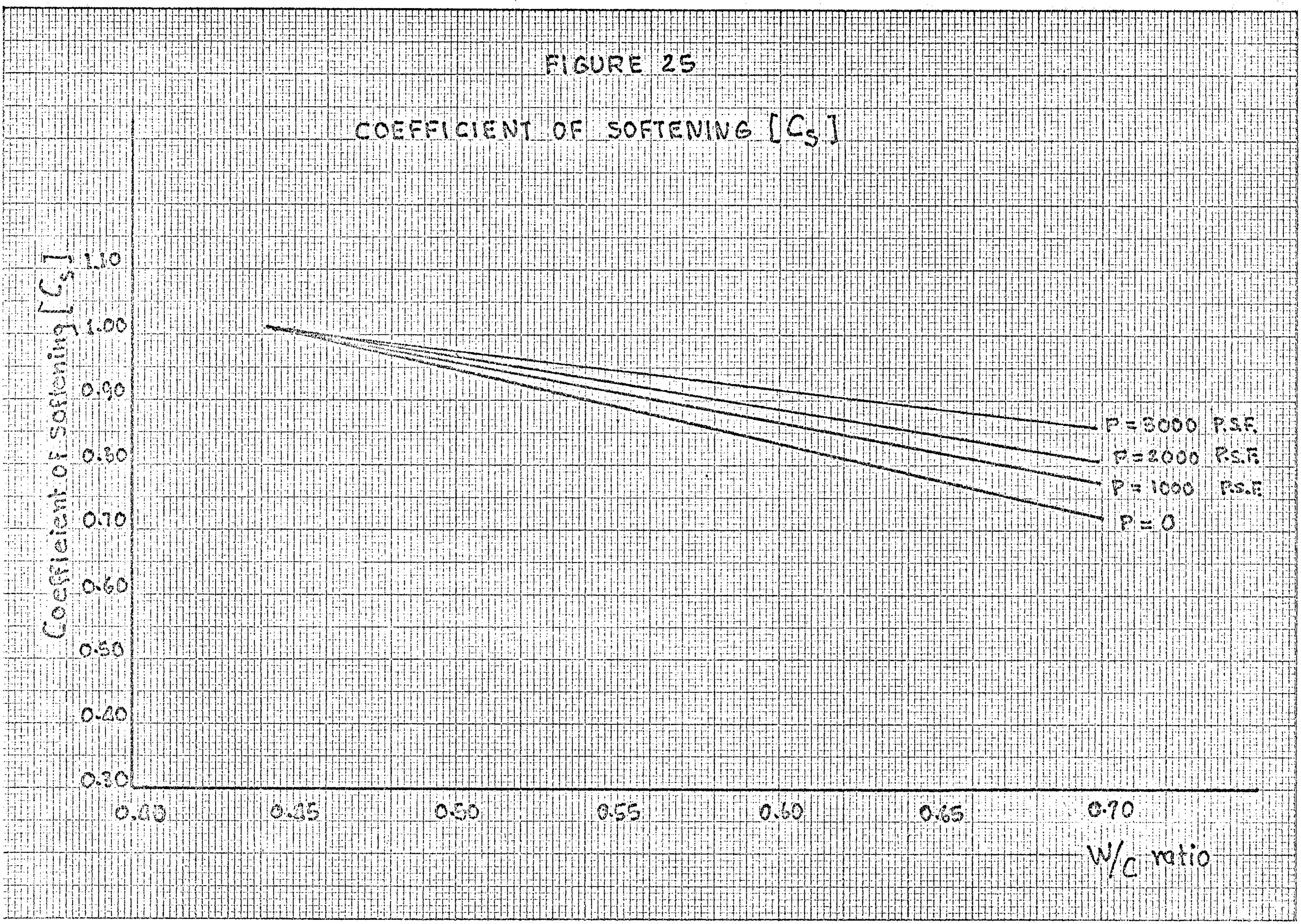


FIGURE 25
COEFFICIENT OF SOFTENING [C_s]
Coefficient of Softening [C_s]
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.40
0.45
0.50
0.55
0.60
0.65
0.70
W/C ratio
P = 3000 P.S.F.
P = 2000 P.S.F.
P = 1000 P.S.F.
P = 0

CHAPTER VI

CONCLUSIONS

The samples tested can be considered typical of many Winnipeg clays. The following conclusions can be made from the test results :

1. Major function influencing the undrained shear strength parameters of the clay is moisture content. The cohesion(c) decreases with increasing moisture content. The angle of shearing resistance(ϕ) does not change significantly with the change in moisture content.

2. Friction between the clay and cast-in-place mortar also depends on moisture content. It decreases with increasing moisture content at the shear plane and increases with increasing normal pressure. The strength of the clay-mortar is increased by the effect of cementing of the mortar to the clay. The true skin friction at the contact surface(T =0) is about 2.2 times the shear strength of the clay. The clay-mortar strength decreases with increasing distance from the mortar, and at a distance of about 0.25 inches from the mortar, the strength value decreases to the shear strength of the clay.

From equation (1), $c_a = \alpha c$.

The test results indicate :

The coefficient α at 0 inch from mortar = 2.2

The coefficient α at 0.10 inches from mortar = 1.3

The coefficient α at 0.25 inches from mortar = 1.0.

The angle of skin friction(δ) is approximately the same as the angle of shearing resistance(ϕ) of the clay. Also it does not change with a change in moisture content.

3. The failure plane between a clay and a cast-in-place mortar block is not exactly at the contact surface. It will occur at a place where the shear strength of the clay-mortar is equal to the undrained shearing strength of the clay. That is the "friction" value of a clay for a cast-in-place mortar, is the clay shearing strength.

4. Clay will be softened by water from cast-in-place mortar. This softening depends on the moisture equilibrium condition for mortar and soil being considered. The test results for the clay and mortar tested in the laboratory showed that the softening depends on the W/C ratio. For other soils and concrete similar relationships can be established. For low W/C ratios drying of the soil will result.The coefficient of softening(C_s) obtained in the laboratory was unity for a W/C ratio of 0.45, it decreased with increasing W/C ratio.

5. In design practice, the friction value of the clay tested, equal to 200 psf may be used for a pile of about 20 feet long. The safety factor will increase with increasing length of the pile, and with decreasing moisture content of the clay when the excess water diffuses away.

For a cast-in-place concrete pile, a bored hole should be made by an auger using no water and the concrete must be placed at a fairly dry mix. It is important to avoid excessive water to soften the clay in the bored hole. In time it is likely that the excess moisture content would return to a normal condition with a corresponding increase in strength. This has conservatively been neglected in the preceding discussion.

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