A STUDY OF THE FOUNDATION FAILURE

of the

TRANSCONA GRAIN ELEVATOR

A Thesis

Presented to

the Faculty of Graduate Studies

The University of Manitoba

in Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

by

Michael Bozozuk

May, 1954

- ACKNOWLEDGEMENTS -

The author wishes to express his appreciation to Assistant Professor A. Baracos for his helpful suggestions and criticisms. Acknowledgements are also due to the National Research Council of Canada, Division of Building Research, who assisted financially.

ABSTRACT

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The failure of the grain elevator provided an opportunity to check bearing capacity theories as failure load and nature of the foundation are known. Thin walled shelby tubes were used to obtain undisturbed soil samples from the seven test holes drilled at convenient locations. The soils tests were performed at the University of Manitoba Civil Engineering Soils Laboratory.

Three bearing capacity theories were checked and compared with the actual failure load. The results:

Prandtl - Buisman	3,025 psf.
Fellenius	5,070 psf.
Terzaghi	6,235 psf.
Actual failure load	6,150 psf.

indicate that only the Terzaghi Theory is applicable to the Transcona Grain Elevator Failure.

This thesis was sponsored by the National Research Council of Canada, Division of Building Research. Permission must be obtained from the sponsor before all or part of the thesis may be published.

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CHAPTER I

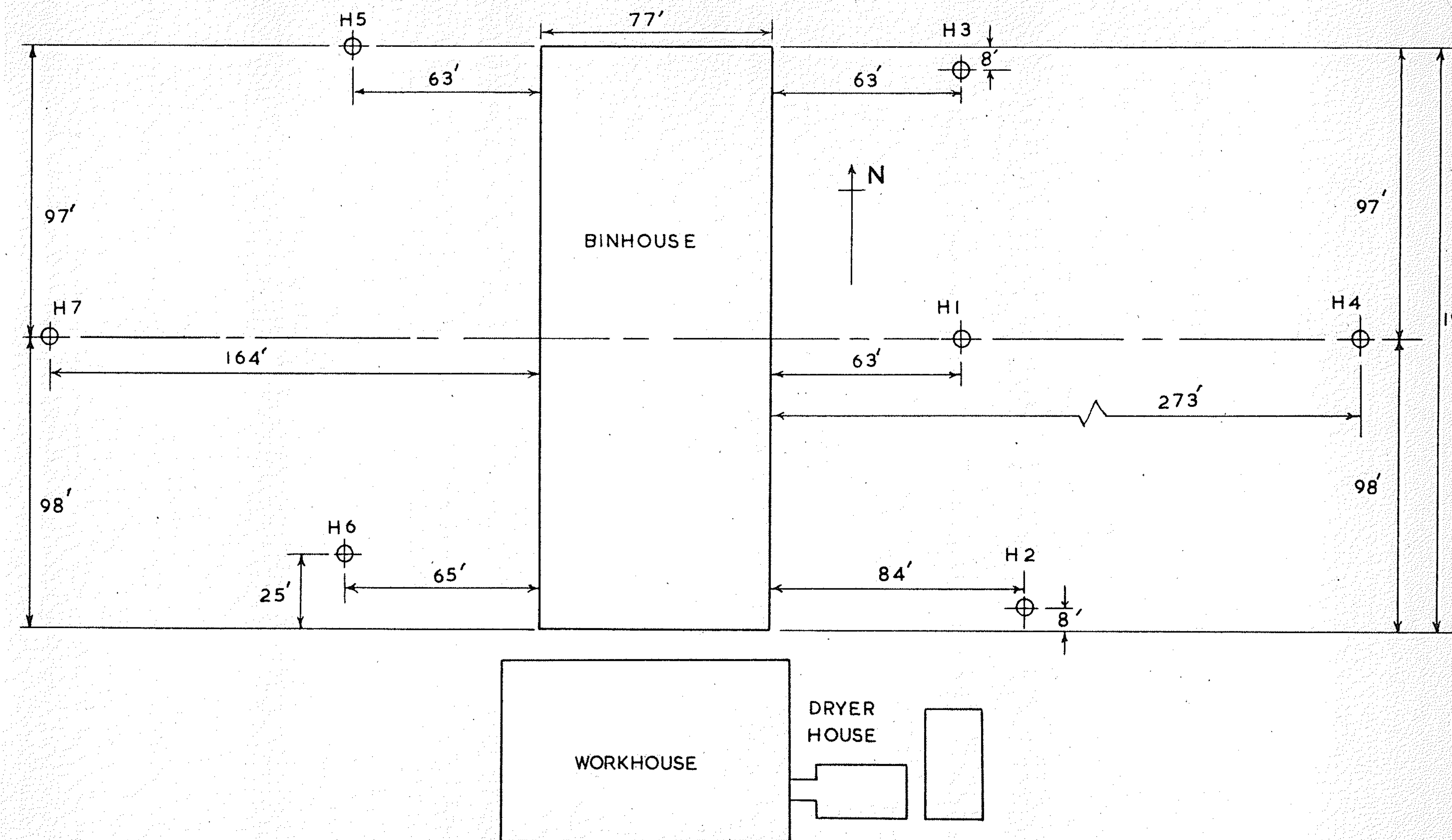
SYNOPSIS

The Transcona Million Bushel Grain Elevator was constructed by the Canadian Pacific Railway Company to store the grain coming from the Western Provinces on route to the Lakehead. Construction was completed and storage started late in September of 1913. On October the 18th, 1913, the binhouse was approximately three-quarters full when settlement was first noticed, followed by a severe listing of the structure. This was a type of failure which permits checking of bearing capacity theories, inasmuch as actual failure load and the nature of the foundation are known. In October, 1952, seven test holes were drilled at convenient locations to a depth of approximately fifty feet, and undisturbed soil samples obtained. Tests were performed at the University of Manitoba Soils Laboratory and calculations made with the object of checking the theory.

DESCRIPTION OF THE ELEVATOR

The general layout of the elevator shown in figure 1 consists of a workhouse, 70' x 96' and 180' high; a binhouse, 77' x 195' and 102' high; a dryerhouse, 18' x 30' and 60' high; and a boiler room equipped with two - one hundred horsepower locomotive boilers.

LOCATION OF BORINGS



PLAN, TRANSCONA GRAIN ELEVATOR

FIG.1

SCALE: 1" = 50'

M.B.

The workhouse is a reinforced concrete structure with brick curtain walls enclosing the top. Its floor is constructed of thirty inch reinforced concrete. The basement of the workhouse, which is 16 feet high, contains belts for transporting grain from cars to conveyor boots and from binhouse to workhouse. The ground floor houses the cleaning and drying machinery, above which are fifteen bins 13 feet in diameter and 70 feet high. Above these bins are floors carrying the rest of the machinery necessary for the operation of the elevator.

Sixty-five circular bins arranged in five rows of thirteen, rest on a twelve inch concrete floor slab making up the binhouse. Each of the bins have a diameter of 14 feet 4 inches and are 92 feet high. The bin walls are of six inch thick concrete with normal steel reinforcing. Total capacity of the elevator is one million bushels. Diamond shaped spaces between the bins are also used for storage.

Extending for the full length of the structure, the bins are surmounted by a cupola housing top conveyors and trippers. This equipment is used for the purpose of filling the tanks with grain. Below the floor slab supporting the bins are four conveyor-belt tunnels which are seven feet wide and run the full length of the binhouse. These tunnels are formed by sixteen inch concrete walls seven feet high, and they in turn rest upon a two foot foundation mattress of concrete.

Crosswalks run transverse to the main tunnels under the bin contacts. They are approximately fifteen inches wide and are spaced about fifteen feet on center. The original excavation for the structure had been made to a depth of twelve feet below prairie level.

NATURE OF FAILURE

When storage was started in September 1913, considerable care was taken to regulate the filling of the different bins. Large buildings supported on floating foundations settled to some extent in this area and it was reasoned that settlement could be controlled through loading. Noticeable settlement began on October 18th. The bins then contained approximately 875,000 bushels. Within an hour after settlement was first noticed, a vertical sinking of about one foot occurred. This was followed by a steady listing towards the west, and at the end of twenty-four hours the binhouse rested at an angle of 26° 53' from the vertical.

After all movements had ceased, it was found that the settlement had caused an upheaval, of the earth surrounding the bins, exceeding five feet. However, no cross sections were taken of the affected area. In addition, the north end of the building had settled some four to five feet more than the south end; while the west side was twenty-nine feet below and the east side five feet above the original position. Figure 2 shows the position of the building after failure, while photographs of the failure and of the righted binhouse

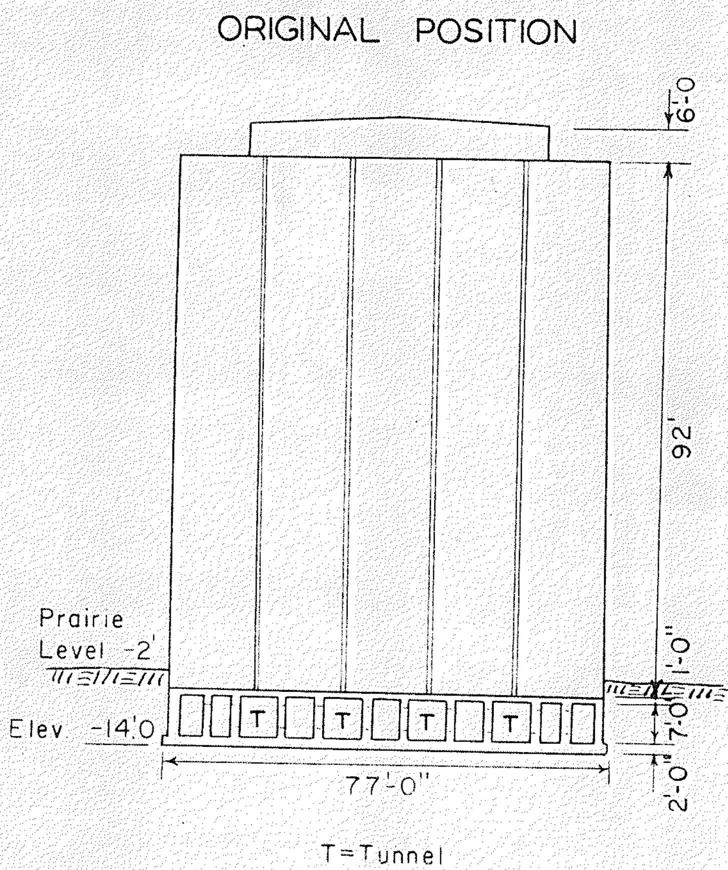


Fig. 2a

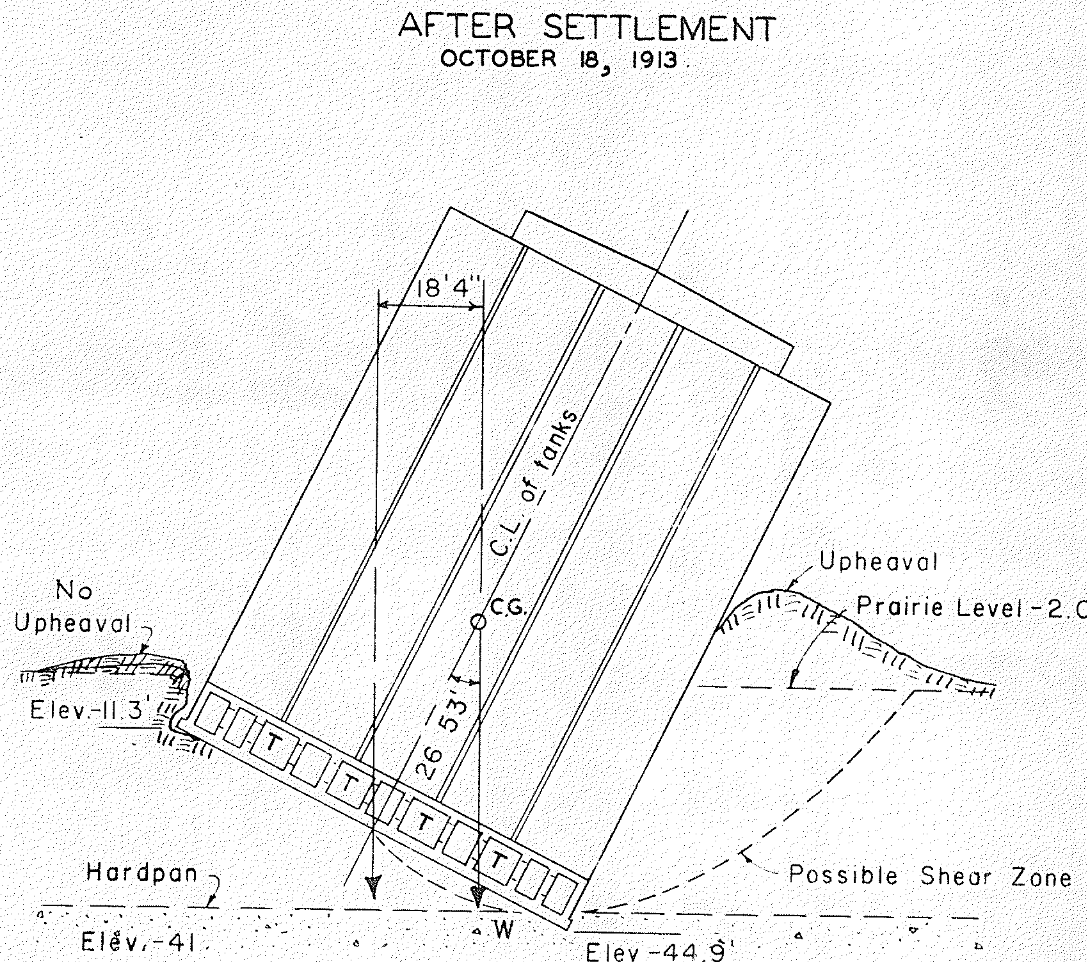


Fig. 2b

SECTION THROUGH BINS, LOOKING SOUTH

are included in figure 3.

Calculation. As the dimensions of the binhouse and the volume of stored grain were known at the time of failure, it was possible to calculate the bearing value of the soil.

Area of mat footing = 77 x 195 = 15,015 square feet

Total weight of structure including footings = 20,000 tons[1]

Volume of stored wheat = 875,000 bushels

$$\text{Weight of wheat} = \frac{875,000 \times 60}{2,000} = \underline{26,250} \text{ tons}$$

Total weight on soil at failure 46,250 tons

∴ Maximum bearing stress of soil

$$= \frac{46,250 \times 2,000}{15,015} = \underline{6,150} \text{ lbs/sq. ft.}$$

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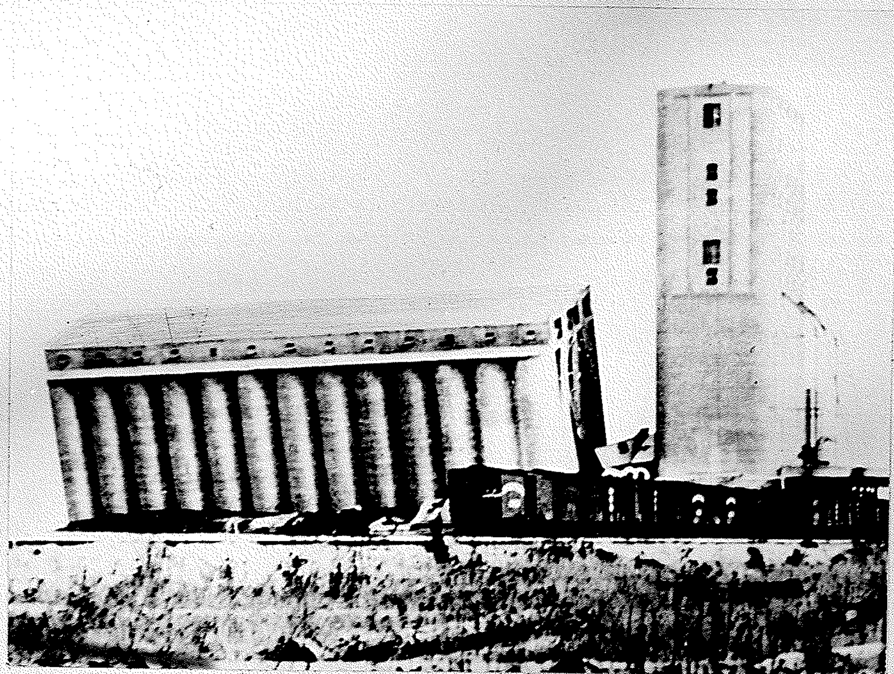
1 A. Allaire, The Failure and Righting of A Million Bushel Grain Elevator. Transactions of the A.S.C.E. Vol. 80, December, 1916. Pages 799 to 832.



TRANSCONA GRAIN ELEVATOR AFTER FAILURE, 1913.

.a.

CLOSEUP OF NORTH END OF BINHOUSE.

.b.

FIG. 3.

CHAPTER II

THEORY: <u>BEARING CAPACITY OF COHESIVE SOILS</u>

An ultimate bearing capacity of a soil may be determined on the basis of a shear failure occurring. Such failures may occur in the form of a logarithmic spiral (fig. 4), a semi-circle (fig. 5), or some combination of both. With reference to the logarithmic spiral as the form of failure, it is assumed that three zones exist after failure is reached. Zone I is the active zone, which acts in unison with the footing. Zone II is plastic, where radial shear exists, and Zone III, represents the passive state zone. (fig. 4a).

<u>Prandlt</u> - <u>Buisman</u> [2] This solution assumes that the most dangerous surface of sliding is the logarithmic spiral (figure 4). Assuming that the soil is incompressible and that the shearing strength of the soil is given by:

$$S_s = c + S_n . \tan . \phi \text{ with c constant,}$$

the expression for ultimate bearing capacity for any soil becomes:

$$q_u = N_b P_b + N_c C + N_g j b$$

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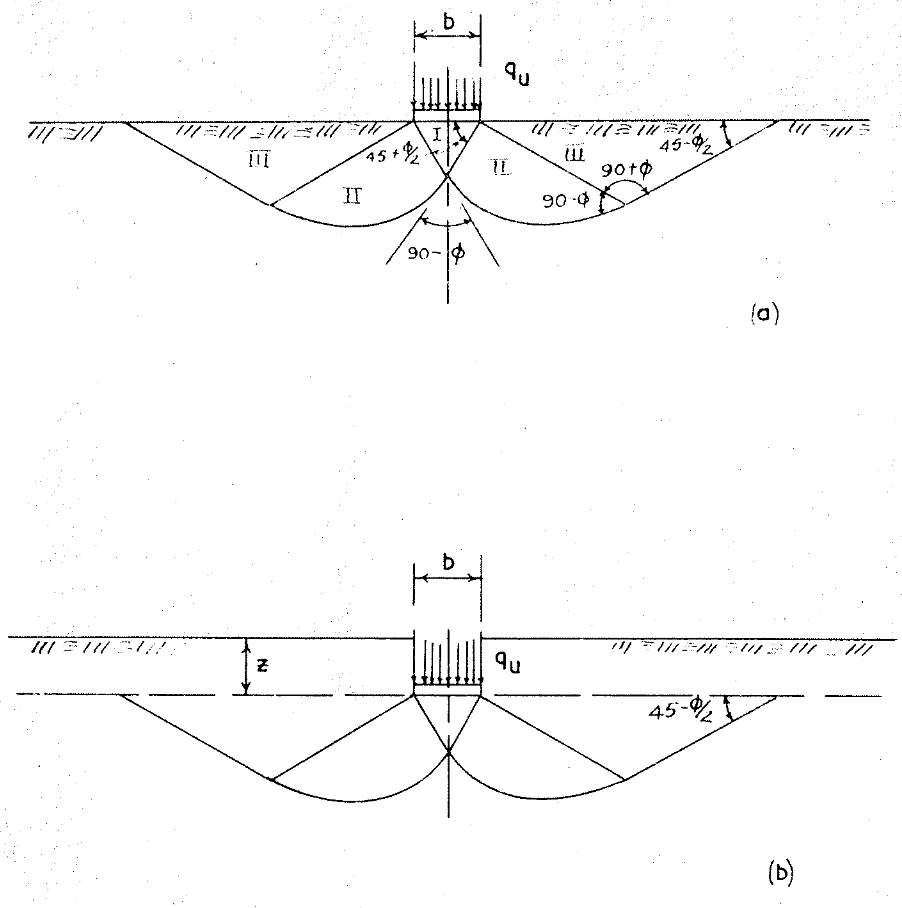
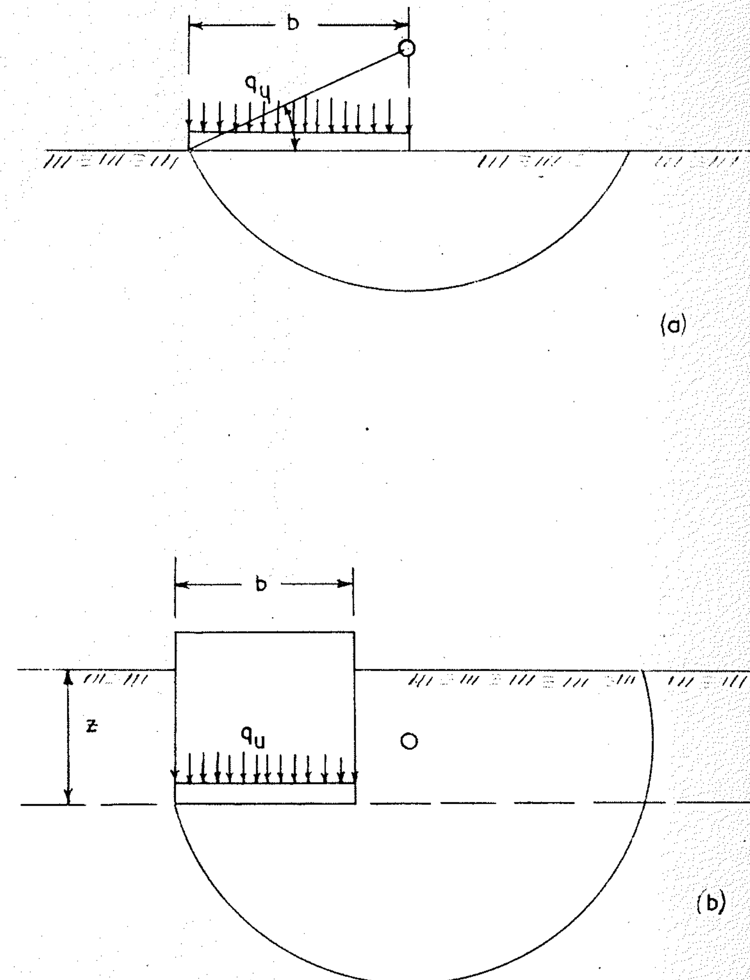
2 Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering. pp 63-65, Vol. 1, 1948.



PRANDTL– BUISMAN SOLUTION

FIG. 4



FELLENIUS SOLUTION

FIG. 5

Where:

q_u = Ultimate bearing capacity

P_b = Surcharge beside footing

C = Cohesion

j = Unit weight of soil

b = Width of footing

z = Depth of footing

N_c, N_b, N_g, are three functions of ϕ

Where:

$$N_b = e^{\pi \tan.\phi} \tan.^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

$$N_c = e^{\pi \tan.\phi} \frac{2 \cos.\phi}{1 - \sin.\phi} + \left(e^{\pi \tan.\phi} - 1\right)\cot.\phi$$

$$N_g = \frac{1}{8}\left[\frac{1 + \tan.^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)}{1 + 9\,(\tan.^2\phi)}\left(\left[3 \tan.\phi \tan.\left(\frac{\pi}{4} + \frac{\phi}{2}\right) - 1\right] e^{\frac{3}{2}\pi \tan\phi} + 3 \tan.\phi + \tan.\left(\frac{\pi}{4} + \frac{\phi}{2}\right)\right) + 2e^{\frac{3}{2}\pi \tan.\phi} \tan.^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) - 2 \tan.\left(\frac{\pi}{4} + \frac{\phi}{2}\right)\right]$$

for $\phi = 0^o$, $N_b = 1$, $N_c = 2$, $N_g = 0$, and equation (1) becomes:

$$q_u = P_b + 2C \qquad (1\text{-a})$$

If $P_b = 0$,

then $q_u = 2C$ (1-b)

If the foundation is at a depth z beneath the surface of the soil then:

$$P_b = jz \text{ and}$$

$$q_u = N_b jz + N_c C + N_g jb \qquad (2)$$

again, if $\phi = 0^o$, $N_b = 1$, $N_c = 2$, $N_g = 0$ and equation (2) becomes

$$q_u = jz + 2C \qquad (2a)$$

<u>The Fellenius Solution</u>[3] assumes that the surface of sliding is circular, fig. 5. Fellenius thus obtained an expression for the ultimate bearing capacity of a soil. For long surface footings and highly cohesive soils he obtained the simplified expression:

$$q_u = 5.5\ C \qquad (3)$$

For footings below the surface of the ground, the resulting simplified equation becomes:

$$q_u = 5.5\ C(1 + 0.38\ \tfrac{z}{b}) \qquad (4)$$

The advantage of the circular arc method is that it is simple and gives reasonable results.

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3 Fellenius Solution. "Fundamentals of Soil Mechanics" by Taylor p. 573, 1948.

Terzaghi Solution[4], presents the commonly used ultimate bearing capacity for soils. He assumes that failure occurs as a logarithmic spiral (figure 4) and derives his formulae on that basis. A distinction is made between local and general shear failure and respective formulae are developed. The type of failure is indicated by the kind of settlement undergone. A general shear failure occurs when settlement is gradual up to a certain point; then very rapid until failure occurs. Local shear failure occurs when there is no distinction between rates of settlement. Settlement continues to increase at a more or less uniform rate from time of loading until failure occurs. See fig. 7 (b).

Because failure at Transcona was by general shear, formulae pertaining to this type of failure are considered only.

The notation is as follows:

q = surcharge

z = depth of footing

S_s = $c + S_n \text{ Tan.} \phi$ (Coulomb's Equation)

where ϕ = angle of shearing resistance

P_p = passive earth pressure

P_{pn} = normal component of passive earth pressure (Zone I)

C = cohesion

H = height of contact face on which P_{pn} acts (Zone I)

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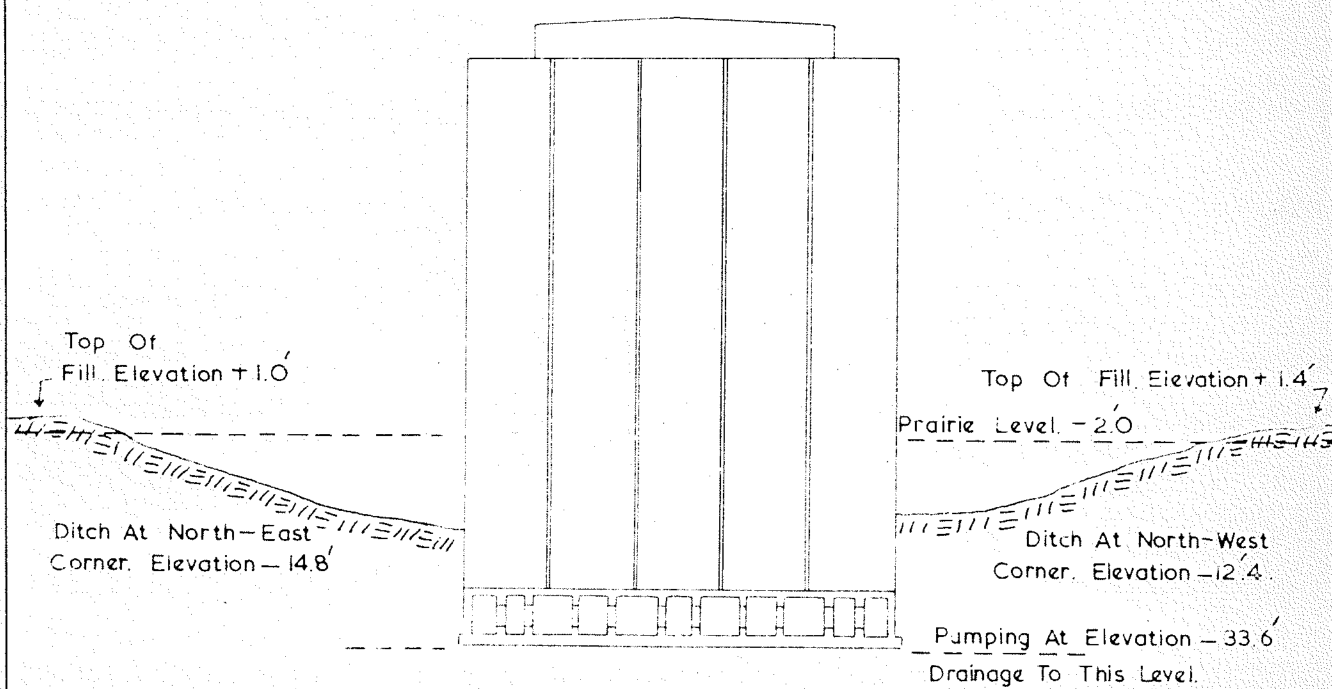
4 Terzaghi Solution. "Theoretical Soil Mechanics" pp. 118-133, May 1947.

PRESENT POSITION — AFTER UPRIGHTING



DEPTH AFFECTED BY DRAINAGE SYSTEM ; 33.6'

SECTION THROUGH BINS —— LOOKING SOUTH

FIG. 6.

M.B.

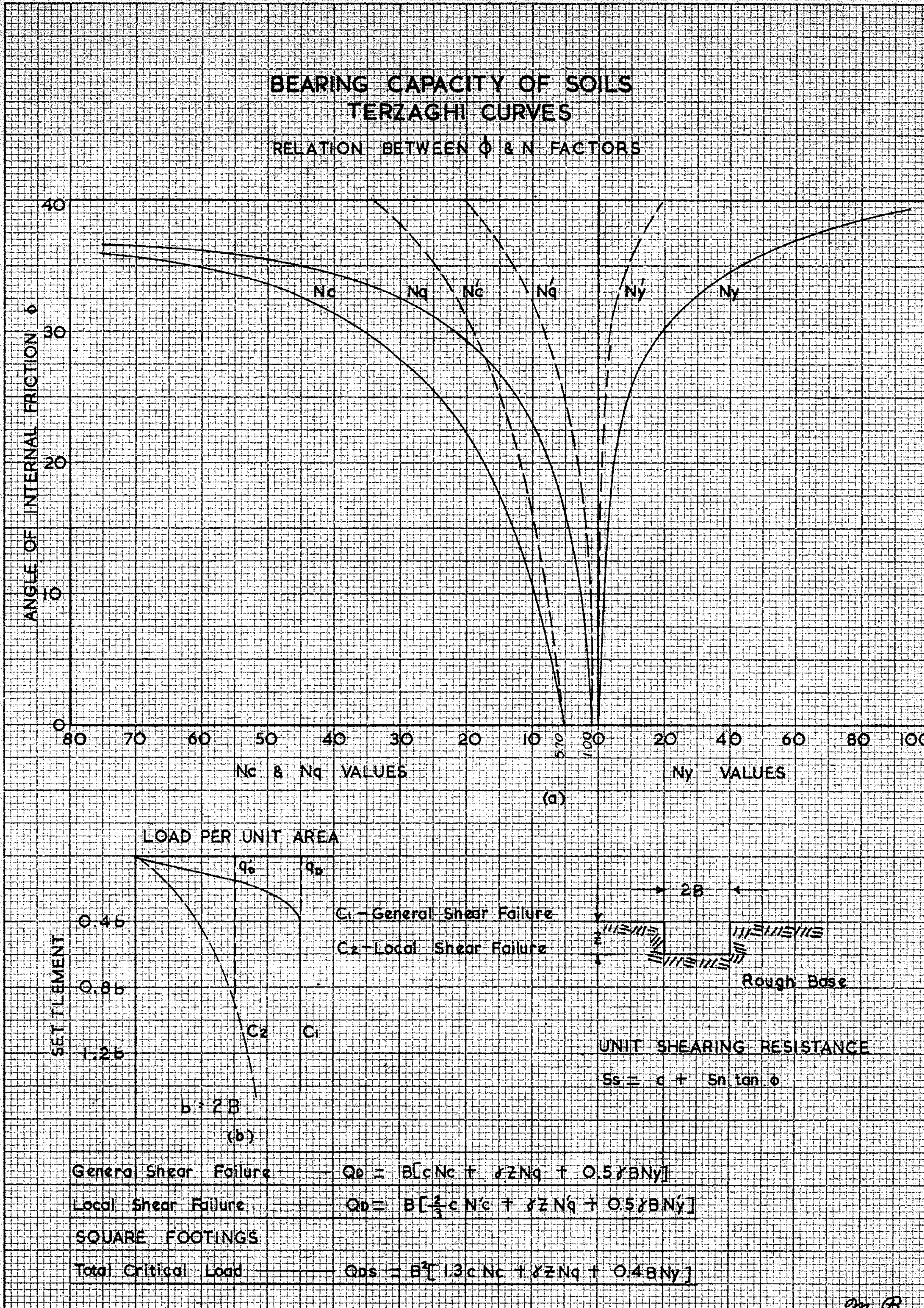

BEARING CAPACITY OF SOILS
TERZAGHI CURVES
RELATION BETWEEN Φ & N FACTORS
ANGLE OF INTERNAL FRICTION Φ
40
30
20
10
0
Nc
Nq
N'c
N'q
N'γ
Nγ
80 70 60 50 40 30 20 10 5.70 1.00 0 20 40 60 80 100
Nc & Nq VALUES
Nγ VALUES
(a)
LOAD PER UNIT AREA
q'o
qo
SETTLEMENT
0.4b
0.8b
1.2b
C2
C1
C1 – General Shear Failure
C2 – Local Shear Failure
b = 2B
(b)
2B
Rough Base
UNIT SHEARING RESISTANCE
Ss = c + Sn tan Φ
General Shear Failure ——— Qo = B[cNc + γZNq + 0.5γBNγ]
Local Shear Failure ——— Qo = B[⅔cN'c + γZN'q + 0.5γBN'γ]
SQUARE FOOTINGS
Total Critical Load ——— Qos = B²[1.3cNc + γZNq + 0.4BNγ]
M.B.

FIG. 7.

α = slope angle of the contact face (Zone I)

j = unit weight of soil

Consider a shallow footing of width 2B resting at a depth z beneath the ground surface (fig. 4). The shearing resistance (S_s) of the soil is determined by Coulomb's equation. At the instant of failure, the shear stresses on the contact face of Zone I, and Zone II are:

$$P_{pn} = C + P_{pn} \tan.\phi.$$

The passive earth pressure P_p on each of the contact faces consists of two components, P_{pn} acting at an angle ϕ to the normal on the contact face and the adhesion component:

$$C_a = \frac{B\ C}{\cos\cdot\phi.}$$

The equilibrium of the mass of soil located within Zone I must satisfy certain equilibrium conditions. That is, the summation of vertical forces including the weight $jB^2\tan\phi$ of the earth in the zone must equal zero.

Then $Q_d + jB^2\tan.\phi - 2P_p - 2BC\tan.\phi = 0$ (5)

and, $Q_d = 2P_p + 2BC\tan.\phi - jB^2\tan.\phi$ (6)

gives the solution if P_p is known. The value of P_p for simplified work is given by: $P_{pn} = \frac{H}{\sin\alpha}(CK_{pc} + qK_{pq}) + \frac{1}{2}jH^2\frac{K_{pj}}{\sin\alpha}$ (7)

Theoretical Soil Mechanics (7)

$\therefore H = B\tan.\phi, \alpha = 180^\circ - \phi, \delta = \phi, C_a = C$

also $P_p = \frac{P_{pn}}{\cos\delta} = \frac{P_{pn}}{\cos\phi}$

then (7) becomes:

$$P_p = \frac{B}{\cos^2\phi}(CK_{pc} + qK_{pq}) + \tfrac{1}{2}jB^2 \frac{\tan\phi}{\cos^2\phi} K_{pj}$$

Combining with (6),

$$Q_d = 2BC\left(\frac{K_{pc}}{\cos^2\phi} + \tan.\phi\right) + 2B_q \frac{K_{pq}}{\cos^2\phi}$$

$$+ jB^2 \tan.\phi \left(\frac{K_{pj}}{\cos^2\phi} - 1\right) \qquad (8)$$

where K_{pc}, K_{pq}, K_{pj}, are whole numbers independent of 2B, the footing width.

for j = 0, we get

$$Qc + Qq = 2B\left(\frac{K_{pc}}{\cos^2\phi} + \tan\phi\right) + 2Bq \frac{K_{pq}}{\cos^2\phi} \qquad (9)$$

$$= 2BCN_c + 2BqN_q$$

where:

N_c and N_q are pure numbers dependent on ϕ in Coulomb's equation

Q_c = load the weightless soil would carry if q = 0,

Q_q = load the weightless soil would carry if the bearing capacity were due to surcharge only.

If C = 0 and q = 0 while j is greater than zero, the critical load due to this is:

$$Qj = jB^2 \tan\phi \left(\frac{K_{pj}}{\cos^2\phi} - 1\right)$$

$$= 2B.jBN_j$$

If C, z, j are greater than zero, then

$$Q_d = Q_c + Q_q + Q_j$$

$$= 2BCN_c + 2BqN_q + 2B^2jN_j$$

but q = jz, then

$$Q_d = 2B(CN_c + jzN_q + jBN_j) \qquad (10)$$

The coefficients N_c, N_q, N_j are the bearing capacity factors for shallow continuous footings and depend only on the angle of shearing resistance ϕ. Terzaghi has plotted these values on a graph simplifying computations of bearing values. Such a graph is shown in fig. 7(a).

For highly cohesive soils and $\phi = 0$, we get

$$N_c = \frac{3}{2}\pi + 1 = 5.7,\ N_q = 1,\ N_j = 0$$

with z = 0, $Q_d = 2B \times (5.7C + jz)$

and bearing capacity per unit of area $q_d = 5.7C + jz$ (11)

For a perfectly smooth footing base,

$$N_c = \pi + 2 = 5.14,\ N_q = 1,\ N_j = 0$$

for z = 0, $Q_d = 2B.(5.14C + jz)$

and bearing capacity $q_d = 5.14C + jz$ (12)

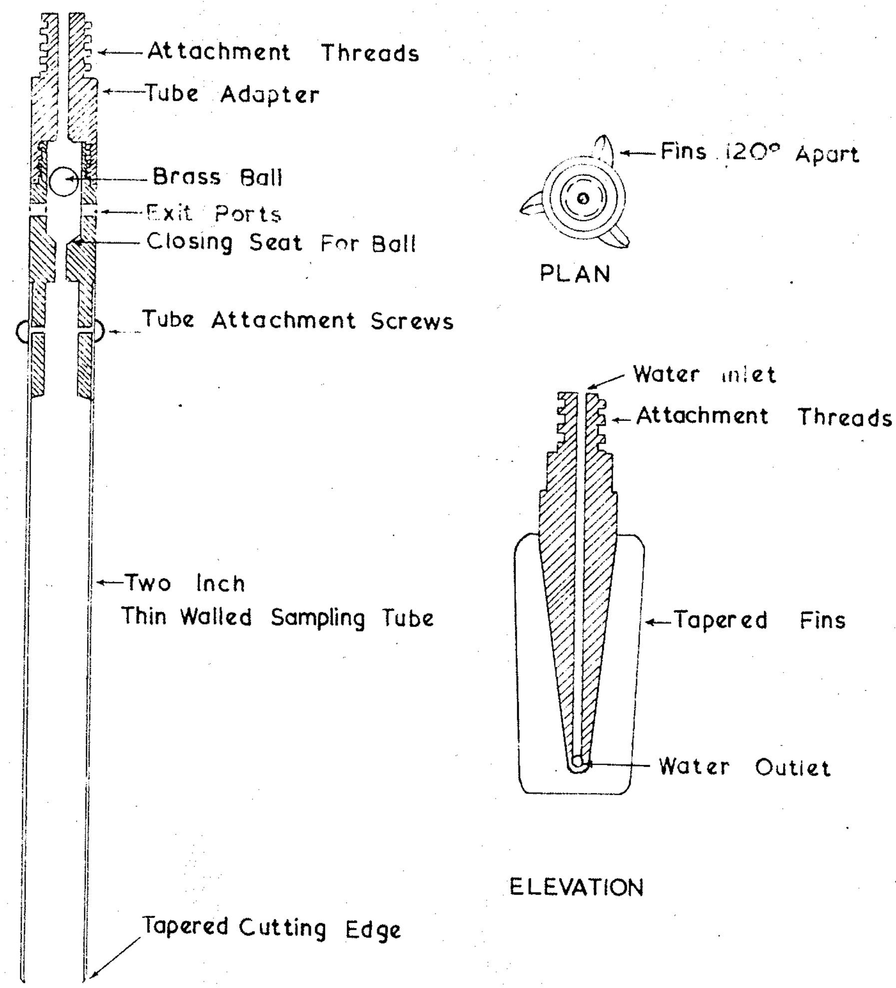
CHAPTER III

NATURE OF INVESTIGATION

A programme of obtaining undisturbed soil samples and laboratory testing was undertaken to permit verification of bearing capacity theories of large footings on clay. Permission was granted by the Canadian Pacific Railway Company to drill on their Transcona Elevator property while the National Research Council of Canada, Division of Building Research assisted financially. Equipment from the Manitoba Government Highways Branch was used and soil sampling commenced on October 21, 1952.

FIELD WORK

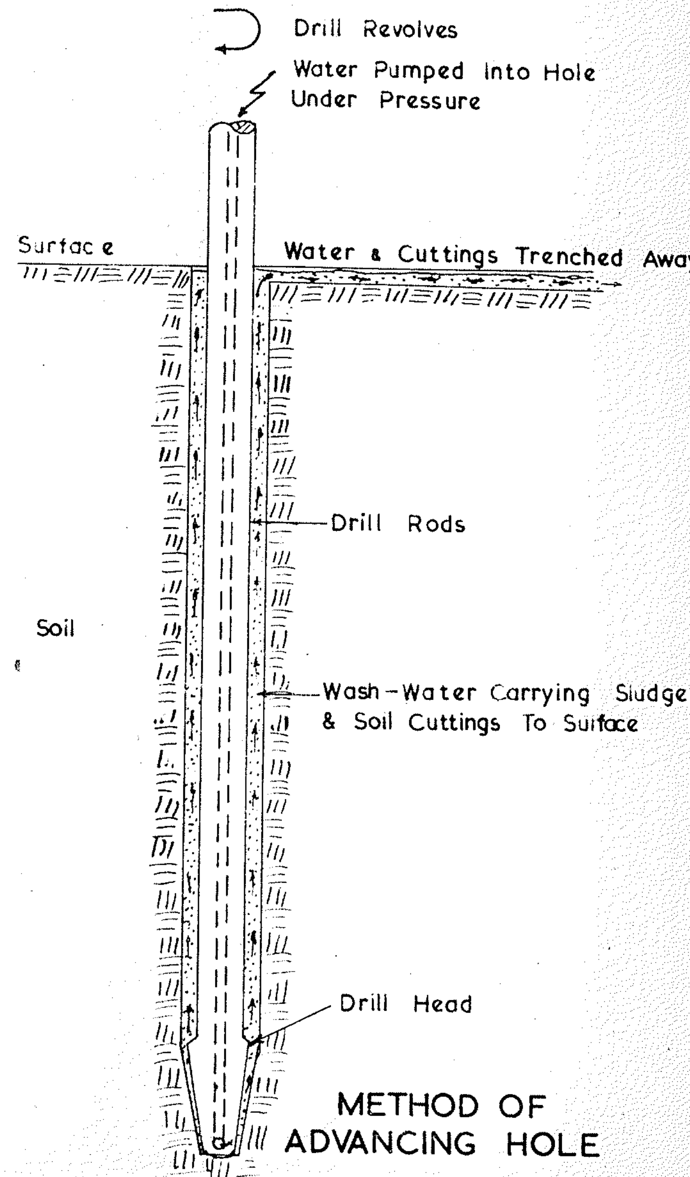
The equipment consisted of a diamond drill (DD-1) converted to a soil sampling machine, a water truck with pump to supply the necessary water, and accessories for sealing and packing the soil samples. For drilling the hole, a drill head known as a "fish-tail" was used (fig. 8b). This fish-tail was threaded at the upper end providing a means of attachment to rods, while the bottom or lower end contained three holes evenly spaced between the three cutting fins providing an outlet for water. All rods come in ten foot lengths and are easily assembled or dismantled during the drilling operations. A one inch diameter axial hole runs through these rods, through which water is pumped under pressure.



SAMPLING TUBE & BALL VALVE
(a)

'FISHTAIL' DRILL HEAD
(b)

FIG. 8.



METHOD OF
ADVANCING HOLE

FIG. 9.

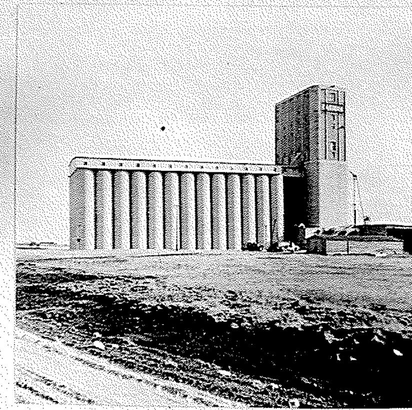
m.B.

To obtain undisturbed soil samples, two inch thin walled samplers with ball check valves were used. The check valve allowed waste sludge to escape through the ports and not into the rods when the tube was being forced into the ground; and formed an airtight seal above the sample within the tube when the tube was being pulled out. See figure 8(a). This prevented the loss of any samples due to suction.

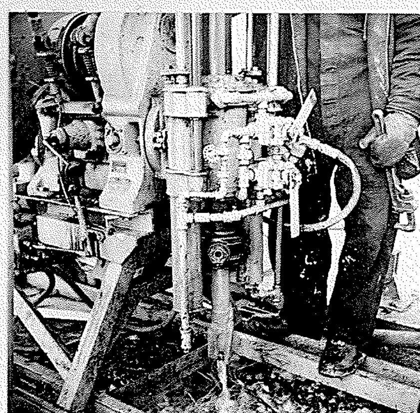
The drill was set up at the required location (see fig. 1 for the location of borings) and the topsoil around the required hole was spaded away. During the drilling operation, water pumped under pressure through the shaft core into the hole washed the cuttings to the surface where it was led away by means of a trench (fig. 9). Inspection of the wash water and cuttings at frequent intervals made it possible to log the hole in the field. This was very important in detecting the exact depths of any subsoil changes. Samples were obtained at five foot intervals of depth or oftener when required; the drilling and sampling continuing until refusal was reached. All tubes containing soil samples were carefully sealed with wax to prevent moisture loss, capped, labelled, and shipped to the University of Manitoba Soils Laboratory for testing.

Photographs of the drilling operations are shown in figure 10.

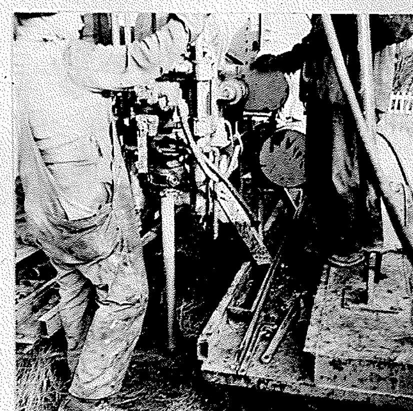
HOLE. 1



FINAL RIGHTED POSITION

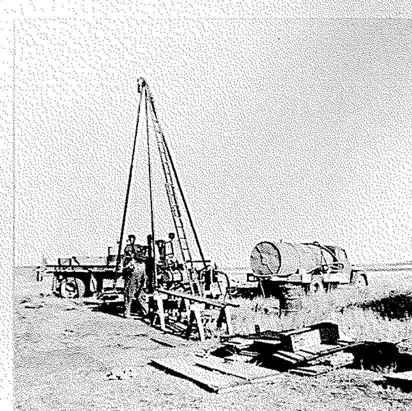


DRILL IN PLACE



SAMPLING TUBE IN PLACE

HOLE. 7



HOLE. 4.

FIG. 10.

LABORATORY WORK

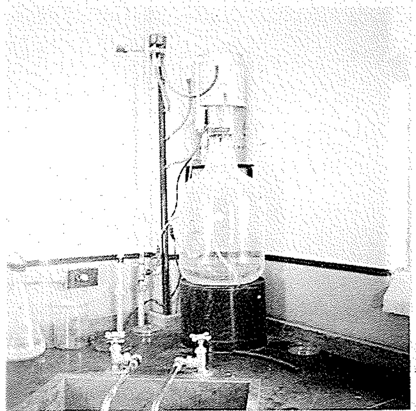
The soil samples were sealed in the field to prevent moisture loss and shipped to the university where they were stored in a cool moist place until tests could be performed. Tests were begun early in November. It was noticed that some remolding had occurred around the outer edge of the samples. To remove the remolded portions, a sample trimmer was obtained from the National Research Council and the tests continued. As all tests were performed in accordance with standard procedure, the tests and their procedures are as follows:

The moisture content, specific gravity, triaxial compression and consolidation tests are outlined in the University of Manitoba, Civil Engineering Soil Testing Laboratory Manual, 1952.

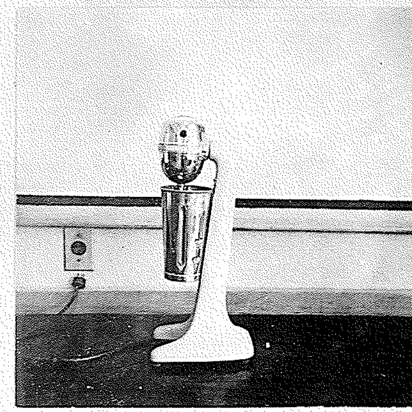
Mechanical Analysis of Soils (Hydrometer Method)
ASTM Designation D422-39

Atterberg Limits ASTM Designations
D423-39, D424-39, D427-39.

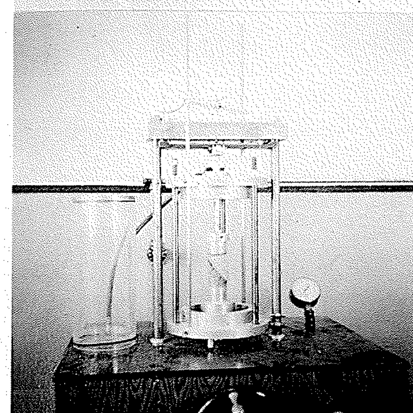
Photographs of equipment used and of some tests are shown in figure 11.



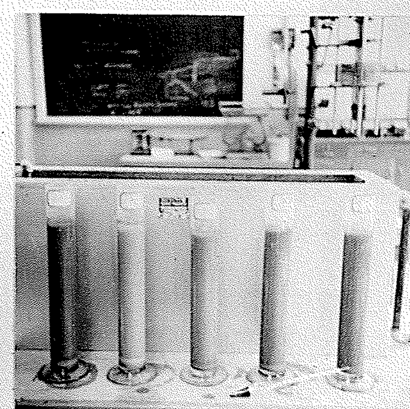
DISTILLED WATER STILL



SOILS DISPERSER



TRIAXIAL COMPRESSION



HYDROMETER ANALYSIS

ATTERBERG LIMITS.

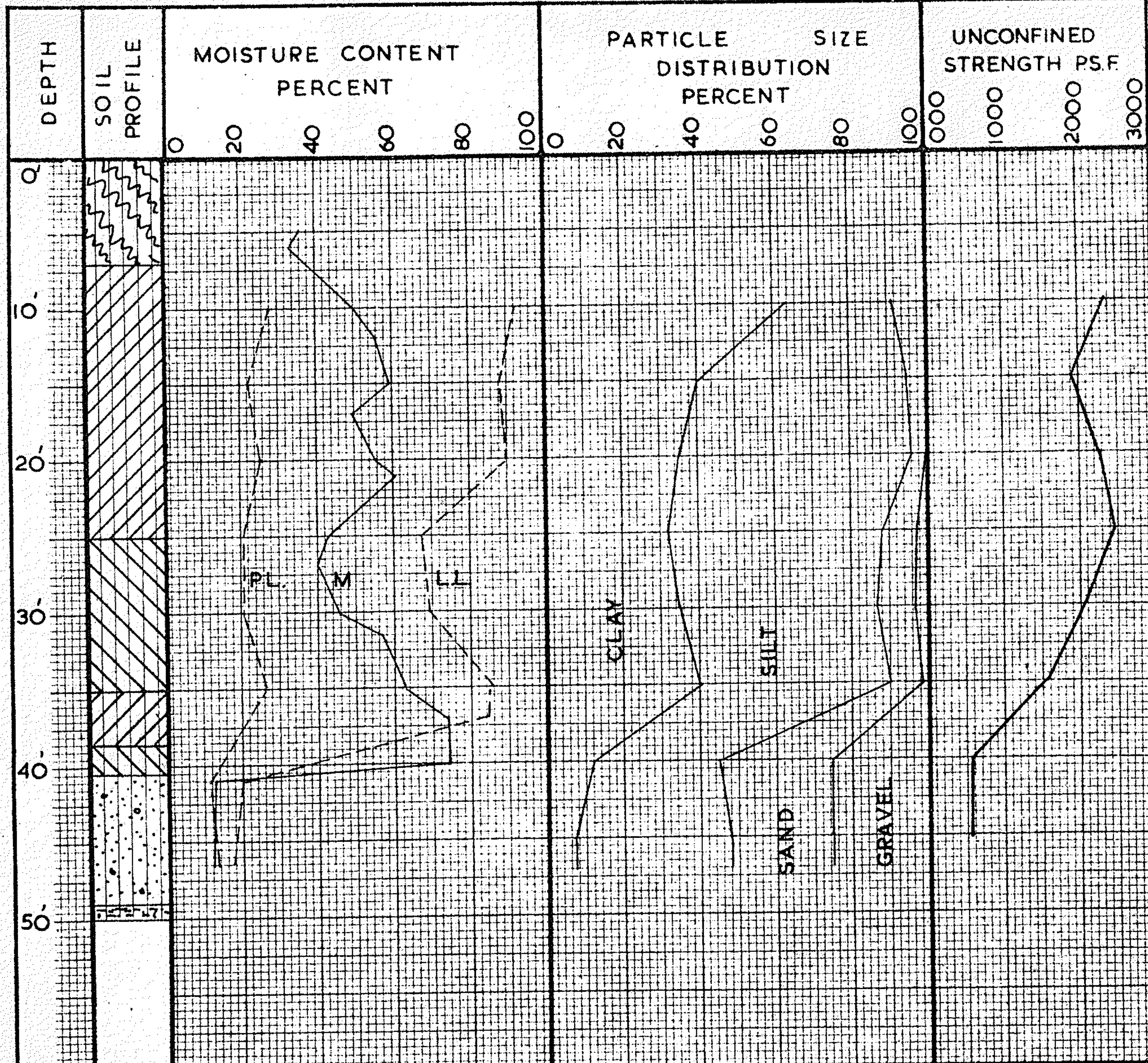
CONSOLIDATION TESTS

FIG. 11.

CHAPTER IV

RESULTS

The test results for each bore hole in both graphical and tabular form are as follows:



DESCRIPTION AND REMARKS

Fill Material
Black & Brown Organic Clay

Stiff Dark Brown Clay
Some Stratification Visible
Contains Some Gypsum & Small
Silt Pockets

Dark Grey Silty Clay
Numerous Large Pockets of Brown Silt
No Stratification Visible
Contains Some Limestone Pebbles

Dark Grey Silty Clay With Silt Pockets
Also Some Sand, Limestone-Gravel

Grey "Slushy" Silty Clay

Grey-White Sand, Silt, & Limestone Gravel

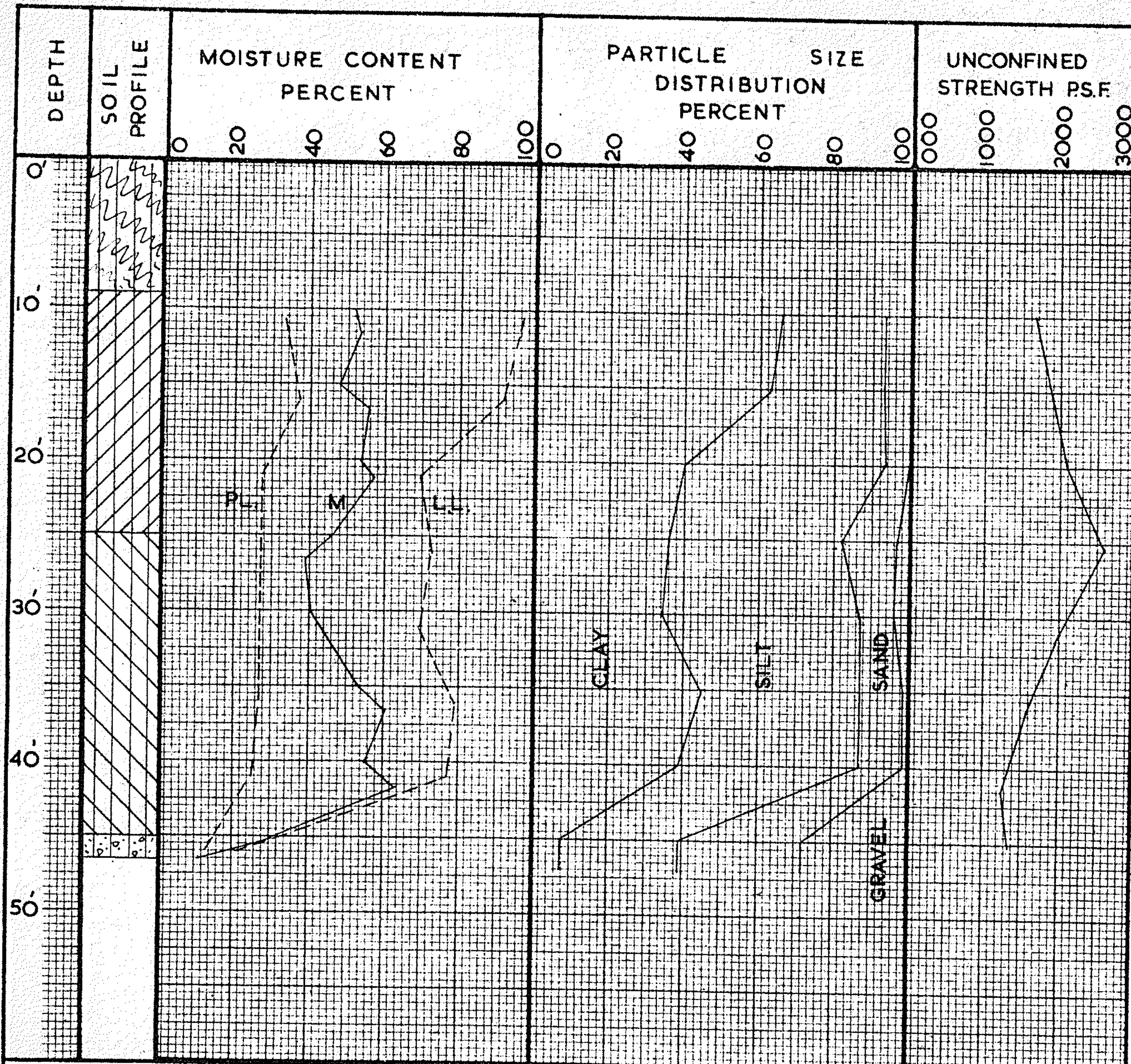
Locally Known as Hardpan

REFUSAL

NOTE: Hardpan Layer Overlies
Limestone Rock Found at El:-52'
During Investigations of 1914

SUMMARY SHEET TEST HOLE NO: 1 PROJECT Foundation Investigation
LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.
SURFACE ELEVATION: +0.4' NOTE: LL=Liquid Limit, PL=Plastic Limit
PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

SOIL MECHANICS LABORATORY
CIVIL ENGINEERING DEPARTMENT
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA



DESCRIPTION AND REMARKS	DEPTH
Fill Material Containing Mixed Black Organic, Grey & Brown Clay & Silt	0'
Stiff Dark Brown Varved Clay Contains some Brown & Grey Silt Pockets. Some Thin Layers of Silt Also Visible Between the Clay Layers.	10' – 20'
Dark Grey Medium Stiff Clay With Considerable Silt Found in Numerous Pockets. Some Sand Pockets & Small Limestone Pebbles Were Found. No Stratification Visible.	30' – 40'
Silt, Sand, & Limestone Gravel.	
Refusal Layer, Top of Hardpan.	50'

SUMMARY SHEET TEST HOLE NO. 2 PROJECT Foundation Investigation
LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.
SURFACE ELEVATION: +0.2' NOTE: L.L. Liquid Limit, P.L. Plastic Limit.
PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

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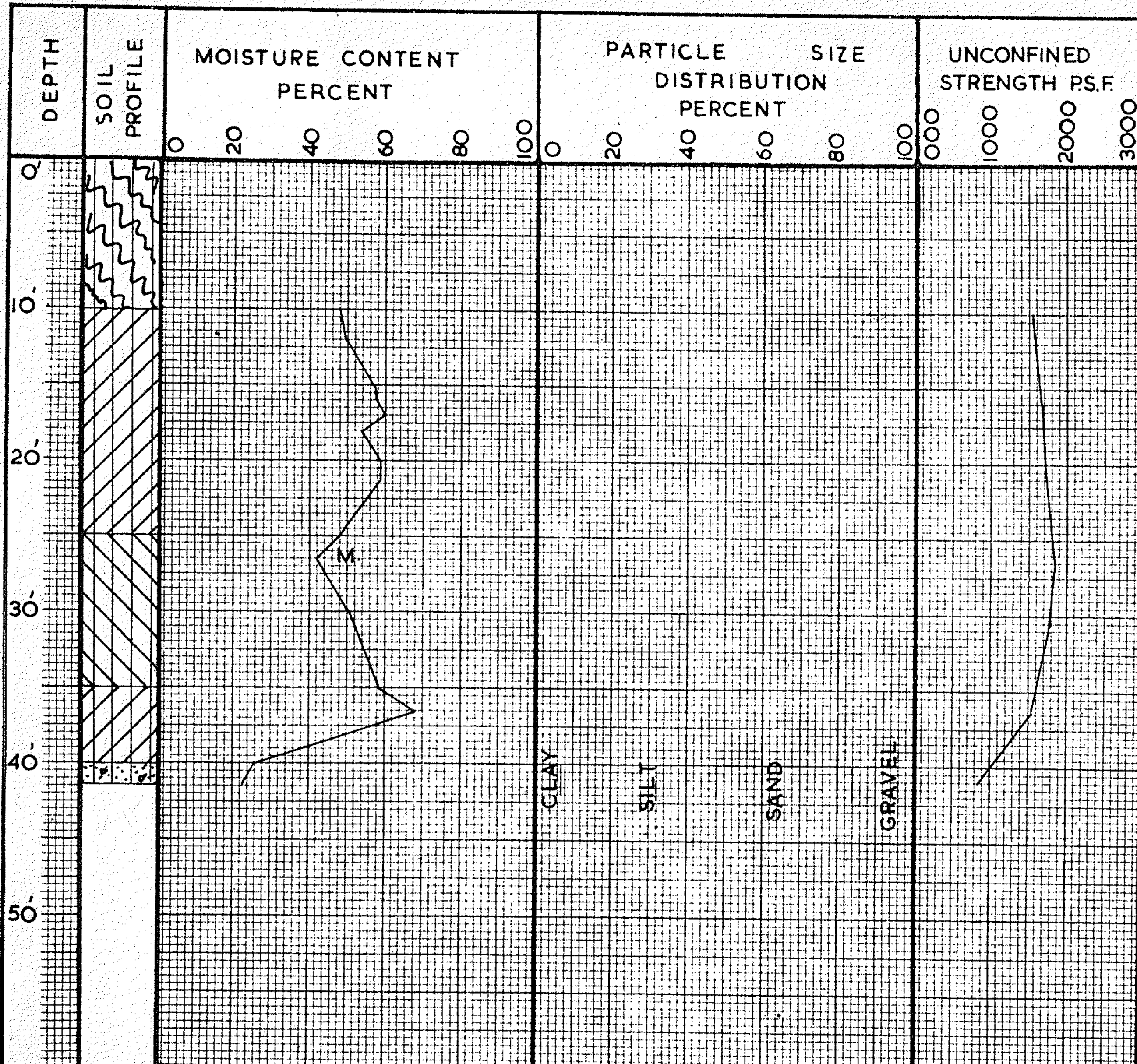

DESCRIPTION AND REMARKS
Fill Material Consisting of Organic and Inorganic Clays. Some Decayed Organics Also Present.
Dark Brown Stiff Stratified Clay (Locally Known As Blue Clay) Some Pockets Of Brown Clay, Silt, And Gypsum.
Grey Clay Containing Silt, Some Sand And Small Pebbles Stratification Visible.
Dark Grey Clay. Numerous Large Silt Pockets & Some Very Fine Sand.
White Clay, Sand, Silt, And Stone
Refusal.

SUMMARY SHEET TEST HOLE NO: 3 PROJECT Foundation Investigation
LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.
SURFACE ELEVATION: 0.0'
PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

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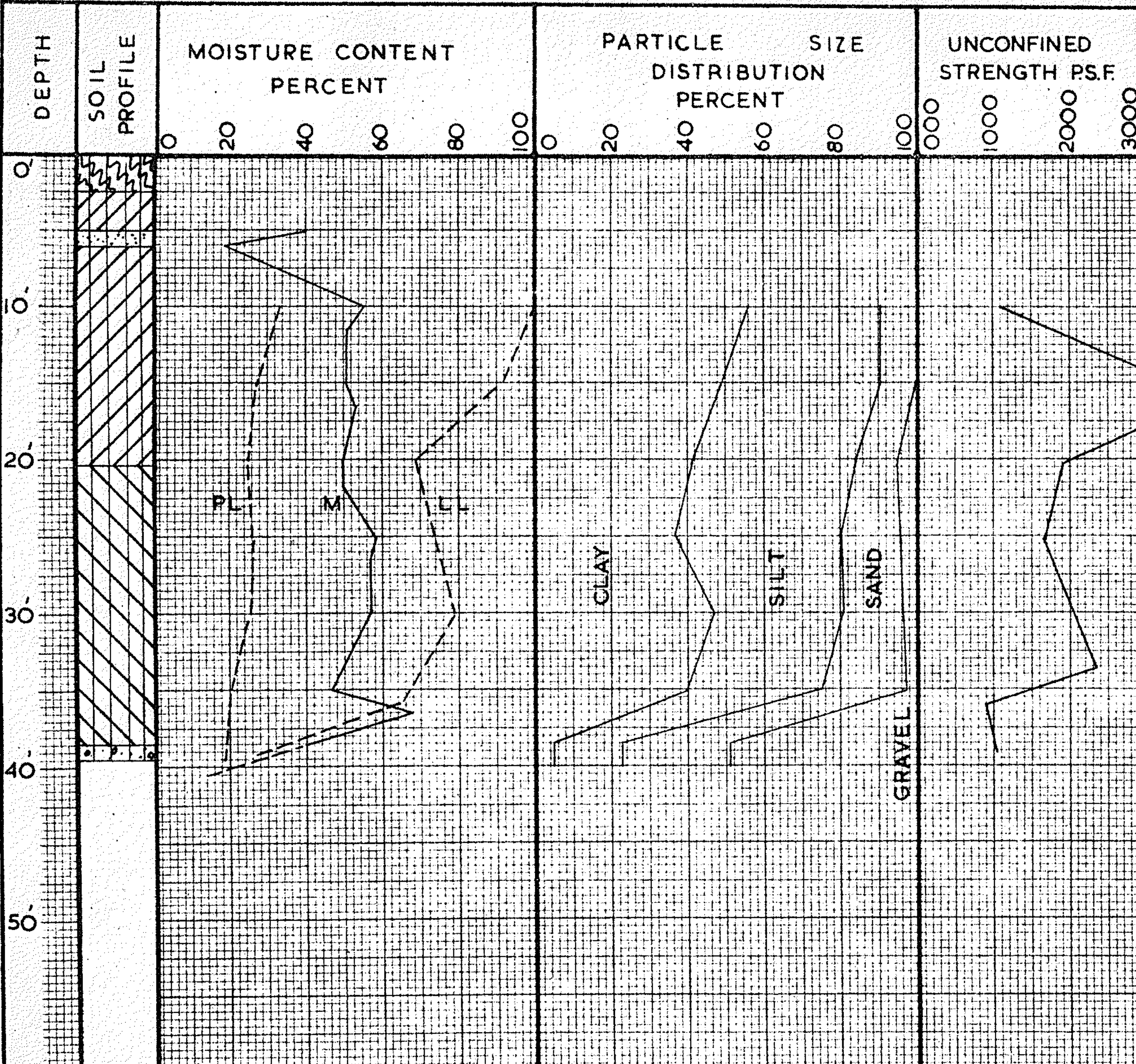

DESCRIPTION AND REMARKS

Depth	Description and Remarks
0'	Black & Brown Organic Clay Fill.
	Light Brown Silty Clay
	Yellow Layer Of Fine Silty Sand.
10'	Stiff Brown Clay With Horizontal Stratification. Pockets Of Silt, Grey Clay, Gypsum, Some Sand, And Limestone Gravel Present In Some Quantities.
20'	Dark Grey Medium Stiff Clay. Contains Considerable Silt Found In Numerous Pockets; Also Sand And Angular Limestone Pebbles Forming The Gravel. Horizontal Strata Visible In Places.
30'	
40'	Silt, Sand, & Gravel. Very Little Clay.
	Refusal To Penetration Of Shelby Tube. This Layer Covered With Very Soft Grey Silty Clay.
50'	

SUMMARY SHEET TEST HOLE NO. 4 PROJECT Foundation Investigation

LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.

SURFACE ELEVATION: -0.6' NOTE: L.L. Liquid Limit, PL. Plastic Limit.

PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

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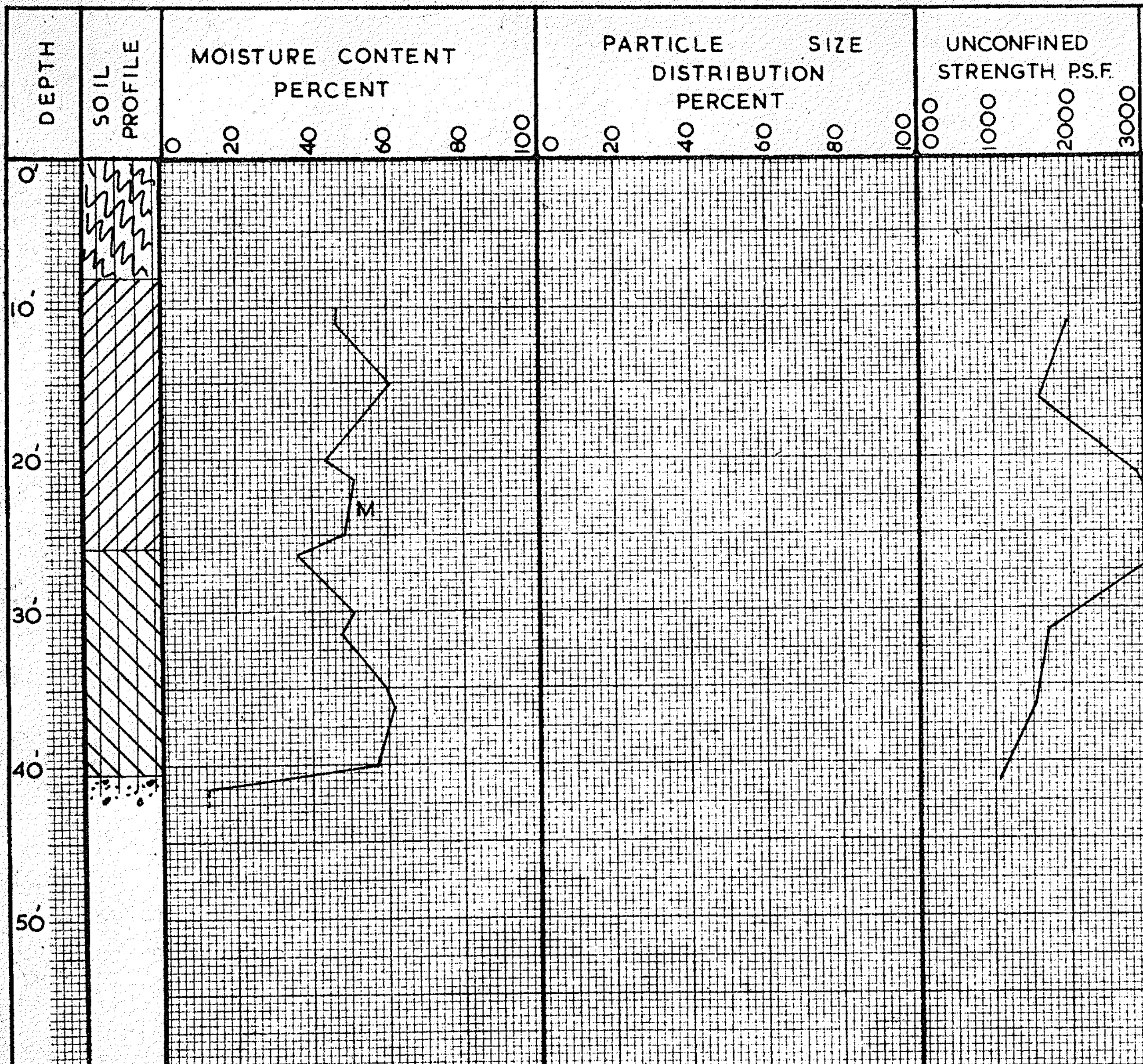

DESCRIPTION AND REMARKS

Fill Material Consisting Of Organic Clay, Mixed Brown & Grey Clays.

Stiff Brown Varved Clay With Silt Pockets And Some Small Stones. Alternate Layers Of Clay And Silt Exposed In Places.

Dark Grey Clay Containing Numerous Silt Pockets, Some Sand And Pebbles. The Pebbles Are Angular Limestone.

White Silty Sandy Gravel. Hardpan.

SUMMARY SHEET TEST HOLE NO: 5 PROJECT Foundation Investigation
LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.
SURFACE ELEVATION: + 1.4'
PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

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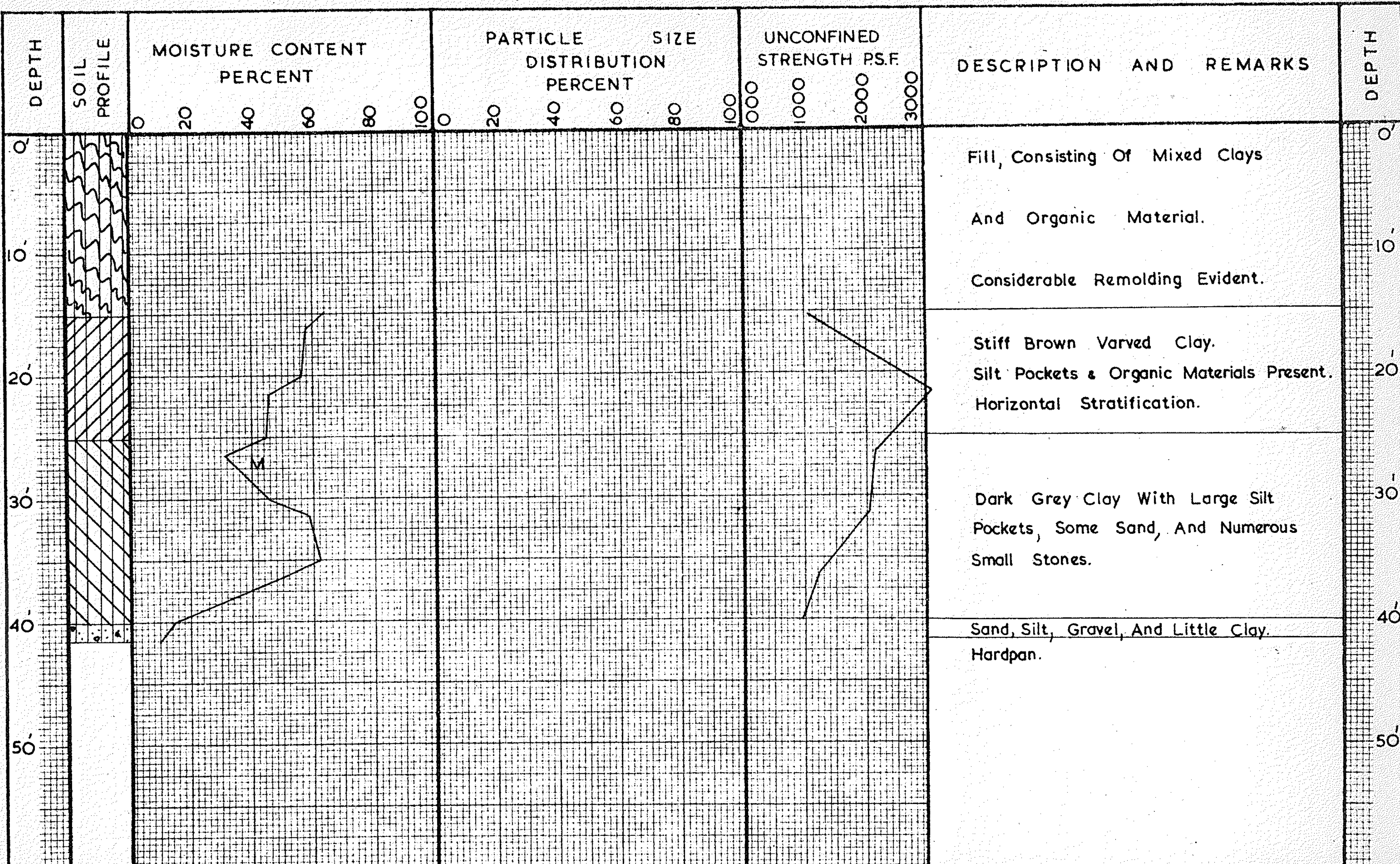

SUMMARY SHEET TEST HOLE NO: 6 PROJECT Foundation Investigation
LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.
SURFACE ELEVATION: +0.9'
PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

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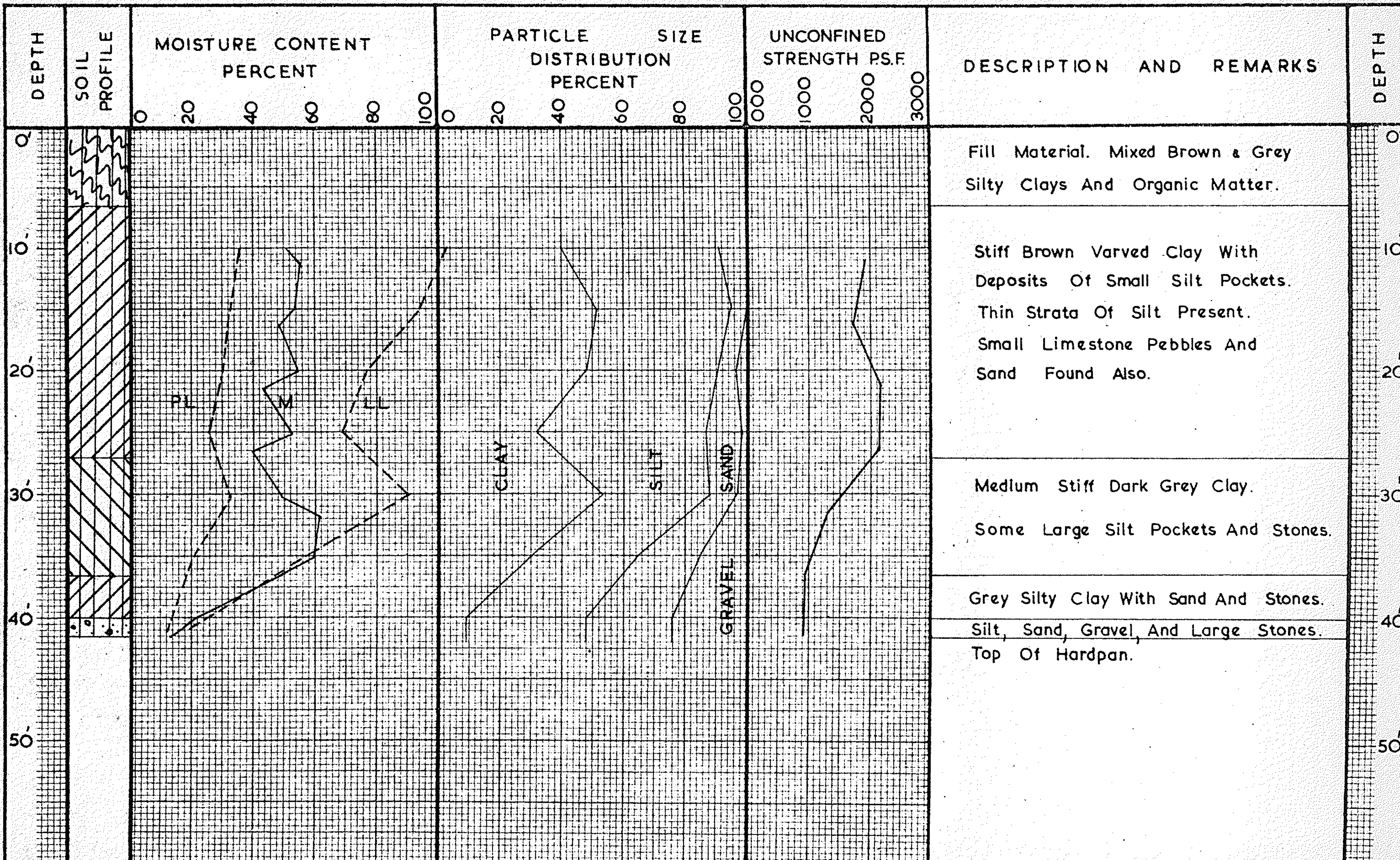

SUMMARY SHEET TEST HOLE NO: 7 PROJECT Foundation Investigation
LOCATION: TRANSCONA GRAIN ELEVATOR, See Plan.
SURFACE ELEVATION: + 1.7' NOTE: LL. Liquid Limit; PL. Plastic Limit.
PLOTTED BY: M. BOZOZUK DATE: APRIL CHECKED BY: A. BARACOS DATE: 1953

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File No. ..1..................
Project Foundation Investigation
Test Hole ..1..............
Date Submitted .1953..........

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft		Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution				Atterberg Limits %			A.C. Grouping	Comments Description
		Moist	Dry					% Clay	% Silt	% Sand	% Gravel	L.L.	P.L.	P.I.		
	5			37.9												Fill.-Mixed Clay
	6			33.6												Fill.-Mixed Clay
	10 / 11.5	108.0	72.0	50.1 / 56.2	100.0	16.4	0*	63.0	28.0	9.0	0	92.4	27.8	64.6		Brown Clay Brown Clay
	15 / 17	106.4	81.4	59.6 / 49.7	77.0	13.0	0*	40	54.5	5.5	0	87.7	26.3	61.4		Brown Varved Clay-some Silt Pockets.
	20 / 21	108.9	73.3	55.8 / 59.9	100.0	15.5	0*	34.5	62.0	3.5	0	90.1	26.0	64.1		Brown Clay & Silt Pockets Brown Clay & Silt Pockets
	25 / 26.5	113.8	81.3	42.9 / 40.2	99.4	17.1	0*	32.0	56.0	9.5	2.5	67.5	21.1	46.4		Dark Grey Clay & Silt Pockets Dark Grey Clay & Silt Pockets
	30 / 31.3	115.0	81.5	45.9 / 56.8	100.0	13.9	3°*	36.0	50.0	11.0	3.0	69.3	20.8	48.5		Dark Grey Clay & Silt Pockets Dark Grey Clay & Silt Pockets
	35 / 36.7	107.1	69.9	63.4 / 73.9	100.0	10.9	3°*	40	50.5	9.0	0.5	85.7	26.8	58.9		
	40 / 41	149.1	135.0	74.3 / 13.2	100.0	4.0	3°*	9.0	36.0	29.5	25.5	20.0	12.0	8.0		Very Soft Dark Grey Clay Grey White Sand & Silt
	45 / 46.2			12.9 / 12.4				7.5	40.5	27	25	18.0	13.4	4.6		Silt, Sand & Gravel Silt, Sand & Gravel
																* Assumed Values

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File No. 2
Project Foundation Investigation
Test Hole 2
Date Submitted 1953

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft		Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution				Atterberg Limits %			A.C. Grouping	Comments
		Moist	Dry					% Clay	% Silt	% Sand	% Gravel	L.L.	P.L.	P.I.		Description
	10	106.2	72.0	51.9	100.0	11.7	0	66.0	27.5	6.5	0	97.0	32.9	64.1		Brown Clay (Stratified)
	11.5	107.3	72.7	53.6												Brown Clay (Stratified)
	15	107.0	70.5	48.1	96.5	13.6	0	63.0	30.0	7.0	0	92.0	37.2	54.8		Brown Varved Clay
	16.3	103.3	67.2	55.9												Brown Varved Clay
	20	107.8	72.2	53.8	98.7	15.2	0	40.0	53.5	6.5	0	69.5	27.7	41.8		Brown Varved (Remolded) Clay with Silt
	21.1	109.2	74.7	57.3												Brown Varved (Remolded) Clay with Silt
	25	111.6	78.3	45.9	98.8	18.1	0	36.0	50.0	11.0	3.0	72.5	26.4	46.1		Dark Grey Silty Clay
	26.2	110.0	78.2	38.5												Dark Grey Silty Clay & Stones
	30	113.6	82.5	40.3	99.3	13.6	3°	34.5	52.5	9.0	4.0	69.5	26.3	43.2		Dark Grey Silty Clay & Stones
	31.7	111.2	77.7	45.6												Dark Grey Silty Clay & Stones
	35	104.0	70.8	53.8	95.4	11.0	3°	45.0	42.0	12.0	1.0	79.5	26.7	52.8		Dark Grey Silty Clay & Stones
	36.3	102.8	66.4	60.3												Silty
	40	106.0	71.1	55.3	97.2	8.7	4°	39.0	48.0	12.0	1.0	77.0	25.1	51.9		
	41.6	103.8	66.8	61.9												
	45	154.8	140.9	31.1	100.0	9.5		7.5	31.5	33.0	28.0	20.8	13.0	7.8		Silt, Stones, Sand
	46.5			10.9												Silt, Stones, Sand

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File No. 3
Project Foundation Investigation
Test Hole 3
Date Submitted 1953

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft Moist	Density Lbs/cu ft Dry	Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution % Clay	% Silt	% Sand	% Gravel	Atterberg Limits % L.L.	P.L.	P.I.	A.C. Grouping	Comments Description
	10 / 11.7	109.2	75.8	47.4 / 49.2	96.8	11.0	0*									Stratified Brown Clay
	15 / 16	107.0	72.2	56.9 / 57.3	96.6	11.6	0*									Stratified Brown Traces of Remolding
	17 / 18			59.6 / 53.7			0*									Varved Brown Silty Clay
	20 / 21.2	103.0	65.0	57.6 / 57.6	98.3	11.8	0*									
	25 / 26.4	116.4	85.3	48.1 / 42.3	100.0	12.8	0*									Grey Silty Clay & Pebbles
	30 / 31.3	103.8	66.8	50.4 / 52.7	97.4	12.1	0*									
	35 / 36.4	100.0	62.0	59.2 / 68.1	96.6	10.8	3°*									Soft Dark Grey Clay & Silt Find Sand Layer Intercepted.
	40 / 41.2	111.0	76.0	45.8 / 22.8	100.0	5.8		7.0	41.0	36.0	16.0					Silt, Sand, Gravel & Some Clay Silt, Sand, Gravel & Some Clay
																* Assumed Values

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File No. ....4...............
Project Foundation Investigation
Test Hole ...4...............
Date Submitted ..1953........

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft Moist	Density Lbs/cu ft Dry	Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution % Clay	% Silt	% Sand	% Gravel	Atterberg Limits % L.L.	P.L.	P.I.	A.C. Grouping	Comments Description
	5	110.5	88.5	40.0	73.8	3.73	0									Light Brown Silty Clay
	5.9			19.2												Very Fine Yellow Sand
	10	109.4	73.8	51.6	100.0	7.9	0	56.0	34.5	9.5	0	100.5	33.0	67.5		Varved Light Brown Silty Clay
	11.4	105.9	67.6	55.4												Varved Light Brown Silty Clay
	15	111.2	76.6	51.1	100.0	26.7	0	49.0	41.5	9.5	0	91.4	27.5	63.9		Varved Light Brown Silty Clay
	16.3	105.6	68.8	52.6												Varved Light Brown Silty Clay
	20	108.7	73.2	49.8	99.6	13.5	0	41.5	42.5	10.5	5.5	69.0	25.4	43.6		Varved Brown Silty Clay
	21.7	106.8	71.2	50.0												Brown Silty Clay & Stones
	25	107.9	71.8	58.0	100.0	11.5	0	37.0	43.5	15.0	4.5	73.8	26.2	47.6		Brown Silty Clay & Stones
	26.4	109.2	75.8	57.2												Dark Grey Clay & Stones
	30	105.0	67.2	53.2	100.0	15.0	3°	47.0	34.0	15.5	3.5	79.1	25.5	53.6		
	31.5	106.8	69.2	42.2												
	35	109.6	76.3	46.8	100.0	6.3	3°	40.0	35.0	22.5	2.5	65.6	20.4	45.2		Grey Silty Sandy Clay
	36.2	110.4	75.5	53.9												Grey Silty/Sandy Clay
	38.5	135.2	115.8	40.0	99.0	7.52	3°	4.5	17.5	29.5	48.5	26.7	18.2	8.5		Soft Grey Clay
	39															Gravel, Sand, Silt

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File No. ....5..............
Project Foundation Investigation
Test Hole ....5.............
Date Submitted ...1953......

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft		Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution				Atterberg Limits %			A.C. Grouping	Comments
		Moist	Dry					% Clay	% Silt	% Sand	% Gravel	L.L.	P.L.	P.I.		Description
	10 / 11	112.9	79.0	46.9 / 46.7	100.0	13.6	0*									Brown Varved Clay
	15 / 16.4	107.7 / 105.2	72.2 / 68.4	60.7 / 53.4	97.0	11.1	0*									Brown Varved Clay &Silt.
	20 / 21.3	107.7 / 112.1	71.2 / 79.8	43.6 / 51.2	100.0	19.2	0*									Remolding Evident
	25 / 26.3	119.3 / 117.0	92.0 / 86.8	48.3 / 35.7	95.9	23.1	0*									Brown Clay Silt & Stones Dark Grey Silty Clay & Stones
	30 / 31.5	110.0	75.5	50.6 / 47.8	100.0	11.8	0*									
	35 / 36.4	102.3	63.7	59.6 / 56.4	99.0	10.5	3°*									Ditto with Large stones
	40 / 41.3	156.7	138.0	56.7 / 12.2	100.0	7.33										Soft Dark Grey Clay Silt, Sand & Gravel

*Assumed Values

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File No. 6
Project Foundation Investigation
Test Hole 6
Date Submitted 1953

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft Moist	Density Lbs/cu ft Dry	Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution % Clay	% Silt	% Sand	% Gravel	Atterberg Limits % L.L.	P.L.	P.I.	A.C. Grouping	Comments Description
	15 / 16.1	104.2 / 106.8	67.0 / 70.7	63.5 / 57.9	98.6	8.3	0*									Brown Clay (Remolded Varved) Remolded
	20 / 21.5	106.1 / 109.9	70.5 / 76.1	56.2 / 45.8	97.9	21.9	0*									Brown Silty (Remolded) Clay (Varved)
	25 / 26.4	112.2	79.0	45.0 / 31.5	100.0	14.8	0*									Brown Silty Clay & Stones Grey Silty Clay with Stones
	30 / 31.2	103.6	65.4	45.8 / 52.3	98.9	14.5	0*									
	35 / 36.4	107.8	72.1	62.2 / 37.0	98.7	8.5	3°*									Ditto-with abundant Stones
	40 / 41.3	124.2	98.1	15.4 / 10.0	100.0	6.7										Clay, Stones, Sand & Silt. Clay, Stones, Sand & Silt.

* Assumed Values

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File No. ......7..............
Project Foundation Investigation
Test Hole ....7.............
Date Submitted ..1953.......

LABORATORY TEST SUMMARY SHEET

Sample No.	Depth Ft.	Density Lbs/cu ft Moist	Density Lbs/cu ft Dry	Moisture Content %	Degree of Saturation %	Unconfined Compressive Strength PSI	Internal Friction	M.I.T. Grain Size Distribution % Clay	% Silt	% Sand	% Gravel	Atterberg Limits % L.L.	P.L.	P.I.	A.C. Grouping	Comments Description
	10 / 11.2	108.5	73.8	50.7 / 55.2	98.2	13.2	0*	40.0	51.0	9.0	0	103.0	35.9	67.1		Varved Brown Clay & Silt Varved Brown Clay & Silt
	15 / 16.3	106.2	70.8	51.7 / 48.7	97.2	11.9	0*	51.5	43.5	5.0	0	93.9	32.5	61.4		Varved Brown Clay & Silt (Remolding) Varved Brown Clay & Silt (Remolding)
	20 / 21.5	109.1	76.0	54.4 / 43.7	96.5	15.1	0*	48.0	42.5	5.5	4.0	77.0	29.8	47.2		Varved Brown Clay & Silt Varved Brown Clay & Stones
	25 / 26.5	112.8	81.1	52.1 / 39.9	97.0	14.8	0*	32.0	54.5	12.0	1.5	69.0	25.4	43.6		Varved Brown Clay & Stones (Very Silty) Varved Brown Clay & Stones (Very Silty)
	30 / 31.7	100.3	62.5	49.3 / 60.9	95.7	9.0	0*	53.5	34.5	9.0	3.0	90.0	32.4	57.6		Dark Grey Clay with Silt Dark Grey Clay & Stones
	35 / 36.5	107.9	72.3	59.5 / 49.1	98.5	6.7	3°*	30.0	34.5	21.0	14.5	56.7	20.9	35.8		Dark Grey Clay & Stones Dark Grey Clay & Stones & Sand
	40 / 41.5	142.6	126.1	21.6 / 13.0	100.0	6.4		9.0	39.0	27.0	19.4	19.4	12.8	6.6		Light Grey Silt with Clay, Stones & Sand Silt, Sand, Gravel

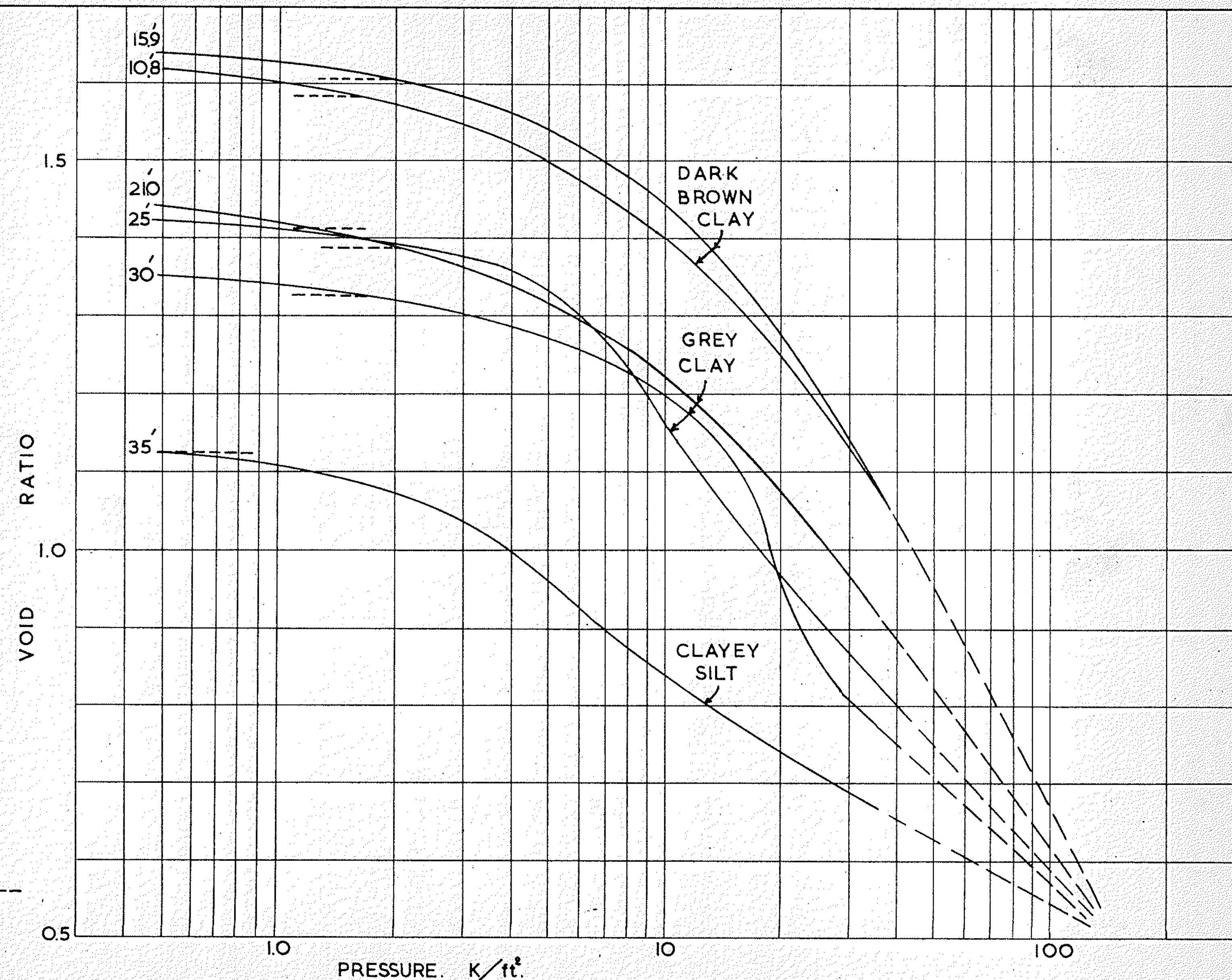
* Assumed Values

DEPTH	COMPRESSIVE INDEX	SWELLING PRESSURE ∵	PRE CONSOLIDATION LOAD ∵
10.8	0.63	1.55	3.5
15.9	0.75	1.95	3.5
21.0	0.66	1.17	8.7
25.0	0.54	2.05	6.0
30.0	0.45	1.42	10.4
35.0	0.28	0.56	3.5

∵ NOTE:

All. Pressures. K/ft².

Initial Void Ratio. -------



CONSOLIDATION RESULTS. HOLE No. 4.

FIG. 12

m.B.

TABLE 8

Summary of Results

Depth Feet	COHESION P.S.I. hole 1	2	3	4	5	6	7
0							
5				1.9"			
10	7.2	5.8	5.5	5.5	6.8		6.1
15	6.5	6.7	5.8	8.2	5.6	4.1"	6.0
20	7.7	7.6	5.9	6.8	9.6"	7.7	7.6
25	8.5	7.6	6.4	6.4	9.6"	7.7	7.4
30	6.9	6.2	6.1	6.9	6.0	7.7	4.5
35	5.4	5.0	5.4	3.1	5.2	4.3	3.3
40	2.0	4.0	2.9	3.0	3.0	3.3	3.0
45							
Average of Top 35′	7.03	6.48	5.85	6.15	5.90	6.85	5.82

" - These figures excluded from calculations because they are too extreme.

(1) Minimum cohesion (hole 7) 5.82 p.s.i.

(2) Cohesion (average holes 4 and 7) 6.04 p.s.i.

(3) Cohesion (average all holes) 6.30 p.s.i.

(4) Cohesion (average of holes 1,2,3,5,6) 6.42 p.s.i.

(5) Maximum cohesion (hole 1) 7.03 p.s.i.

Average wet density of top 12 feet of soil = 107 lbs/ft. 3

TABLE 9

Comparison of Bearing Values

Hole Grouping	Average Cohesion p.s.i.	Prandtl-Buisman p.s.f.	Fellenius p.s.f.	Terzaghi p.s.f.
7	5.82	2962	4880	6065
4,7	6.04	3025	5070	6235
1,2,3,4,5,6,7	6.30	3100	5280	6455
1,2,3,5,6	6.42	3135	5390	6550
1	7.03	3310	5900	7060

Actual failure load = 6150 p.s.f.

Sample Calculations for Ultimate Bearing Capacity

for c = 5.82 p.s.i.

(a) Prandtl-Buisman

$$q_u = 2c + jz \qquad (2a)$$

$$= 2 \times 5.82 \times 144 + 107 \times 12 = \underline{2962} \text{ p.s.f.}$$

(b) Fellenius

$$q_u = 5.5\, c(1 + 0.38\, \frac{z}{b}) \qquad (4)$$

$$= 5.5 \times 5.82 \times 144\, (1 + 0.38 \times \frac{12}{177}) = \underline{4880} \text{ p.s.f.}$$

(c) Terzaghi

$$q_u = 5.7c + jz \qquad (11)$$

$$= 5.7 \times 5.82 \times 144 + 107 \times 12 = \underline{6065} \text{ p.s.f.}$$

CHAPTER V

DISCUSSION

SOILS DESCRIPTION AND CLASSIFICATION

A soils description and classification was obtained for the Transcona Elevator site from tests performed on samples taken from seven drill holes. The description extends from ground level to "hardpan".

Except for a black and brown organic clay and fill ranging from 2½ feet to 10 feet deep, the top 25 feet consists of a varved, stiff dark brown clay with an average moisture content of 55%.

Forty-five per cent of this material is clay, 50% is silt, and 5% is sand. Plastic and liquid limits are 25% and 90% respectively. These properties correspond to an inorganic clay of high plasticity according to the Casagrande plasticity chart.

A dark grey silty clay, softer than the overlay material, is found within the following 15 feet. It is composed of 40% clay, 50% silt, 8% sand, and 2% gravel. The moisture content varies from 40% to 70%. On the plasticity chart it comes under the same classification as the brown clay, that is, an inorganic highly plastic clay. Values of plastic limit and liquid limit are 25% and 75% respectively.

Underlying the brown and grey clays which are Lake Agassiz deposits, are found glacial deposits locally known as "hardpan". The top of this layer may be quite soft. Composition of this material is as follows: 10% clay, 40% silt, 25% sand, and 25% gravel. The moisture content of 13% falls within the plastic limit and liquid limit range for the depth tested. Plastic limits of 12% and liquid limits of 20% are in accordance with the properties of a slightly plastic inorganic silt. Drilling extended 5 feet into the "hardpan".

REMOLDING

Careful examination of the core samples failed to disclose the slip plane at which the failure occurred. However, the examination revealed that the soil in the immediate vicinity had been subjected to some remolding, probably when the elevator was righted. Holes 1, 2, 3, 5, and 6 were affected to a depth of 20 feet. This was an important discovery because remolding affects unconfined compressive strengths. As the damage to most of the samples was slight, the strengths were not affected to any extent as can be seen in table 8. Only holes 4 and 7 represent soil conditions as they probably existed before failure occurred in 1913. Therefore, the results obtained from them should furnish the actual ultimate bearing capacity.

COHESION

Bearing capacity of soils beneath footings depend upon the cohesion, angle of internal friction, and a depth correction factor for the material. Because of the nature of the loading, which was rapid, and the fact that the clays were saturated, it was possible to assume an angle of internal friction for the clays of zero, thereby excluding this term from the calculations. A limited number of quick triaxial tests verified this value. Consequently the cohesion governs the bearing capacity as the depth factor was quite small. Listed in table 8 at five foot intervals are the cohesion values. A representative cohesion was obtained for each hole using an average for the top thirty-five feet. This figure was selected on the basis of depth to "hardpan" which limited the depth of the bottom of the sliding. Most of the failure occurred in the clay.

Five arrangements of these values considered for the calculation of ultimate bearing capacity are listed below table 8. No explanation is required for choosing the minimum cohesion given by hole 7, and the maximum by hole 1. However, results from holes 4 and 7 were selected for one combination knowing that any affects of the uprighting would be negligible. (See figure 1 for the location of borings). Using an average cohesion for all holes is evident, but the average for 1, 2, 3, 5, 6, necessitates some explanation. Moisture contents for the holes show a marked decrease as the thirty foot level is approached. (See summary sheets). Holes 4 and 7 have moisture contents which are uniform for nearly the full depth to "hardpan". It seems that the ditch and drainage system installed during the uprighting in 1914 (see figure 6) drained the soil surrounding the elevator resulting in a decrease in moisture content with consolidation occurring, affecting the unconfined compressive strength. The average cohesion for these holes (6.42 psi) is considerably higher than from holes 4 and 7 (6.04 psi). Table 9 contains bearing values using the cohesion resulting from the above assortment.

ULTIMATE BEARING CAPACITY

A comparison is obtained in table 9 between actual failure load and theoretical ultimate bearing values from the Prandtl-Buisman, Fellenius, and Terzaghi solutions.

The Prandtl-Buisman solution produces results which are considerably lower than the actual failing load. If maximum cohesion is used giving a value of 3310 psf, it is seen that it falls far short of the 6150 psf at which the elevator failed.

The Fellenius solution also gives a low value. If the cohesion from holes 4 and 7 are used, a value approximately one thousand pounds less than the failure load of 6150 psf is obtained. The maximum load of 5900 psf resulting from the cohesion of hole 1 approaches the failure load; however, minimum values govern designs. Therefore, the solution although superior to Prandtl-Buisman, must be considered as unsatisfactory for this case.

The Terzaghi solution: Of the three bearing capacity theories for rapid loading on saturated clays, the test results give best agreement with the Terzaghi analysis. (see table 9). Neglecting the values of 6455 psf, 6550 psf, and 7060 psf which are slightly high due to consolidation resulting from 40 years of drainage, the remaining figures of 6065 psf and 6235 psf agree very closely with the failure load of 6150 psf. It would appear that the Terzaghi solution is not seriously affected by the presence of the stronger underlying layer at the depth of the failure plane. It may be seen from the summary sheets that the layer existed 33 feet below the original foundation.

CAUSE OF FAILURE

Reasons for failure are apparent. The clay subsoil was loaded to failure as a result of underestimating the bearing capacity of the soil. Allaire[1] stated that design was based on a single field load test. The results might have been entirely different had a number of unconfined strength tests been performed which probably would have resulted in a safe design.

CONSOLIDATION CURVES

The purpose of the consolidation tests was to verify the soil characteristics. The curves in figure 12 point out the variation in the compressive index, swelling pressures, and pre-consolidation loads with depth, for a hole located outside the affected area. The swelling pressure and compressive index decreases with increasing depth, indicating that the material is softer and contains more silt. The pre-consolidation load does not follow any apparent pattern. With these curves, it is possible to calculate the settlement the building would have undergone had failure not occurred.

PUMPING

The forty years of continuous pumping from beneath the binhouse effected some consolidation. Average moisture contents and wet densities for holes 4 and 7 were 51.9% and 107.8 lbs/cu.ft. respectively and 48.9% and 112.2 lbs/cu.ft. respectively for holes 1, 2, 3, 5, 6. Strength of soil was also affected, being slightly higher for holes 1, 2, 3, 5, 6, than for 4 and 7. However, the bearing capacities were not materially altered. (See table 9).

CONCLUSIONS

Soils tests performed during the investigation of the failure of the Transcona Grain Elevator, indicated that the Prandtl-Buisman and Fellenius Theories did not apply. However, the Terzaghi Theory for the ultimate bearing capacity of rapidly loaded saturated clay subsoils satisfied this case.

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Photographs of failure, figures 3a and 3b were obtained from The Foundation Company of Canada, Toronto, Ontario.
