A STUDY OF FOUNDATIONS AND SOIL CONDITIONS ON THE UNIVERSITY OF MANITOBA CAMPUS

A Thesis

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by

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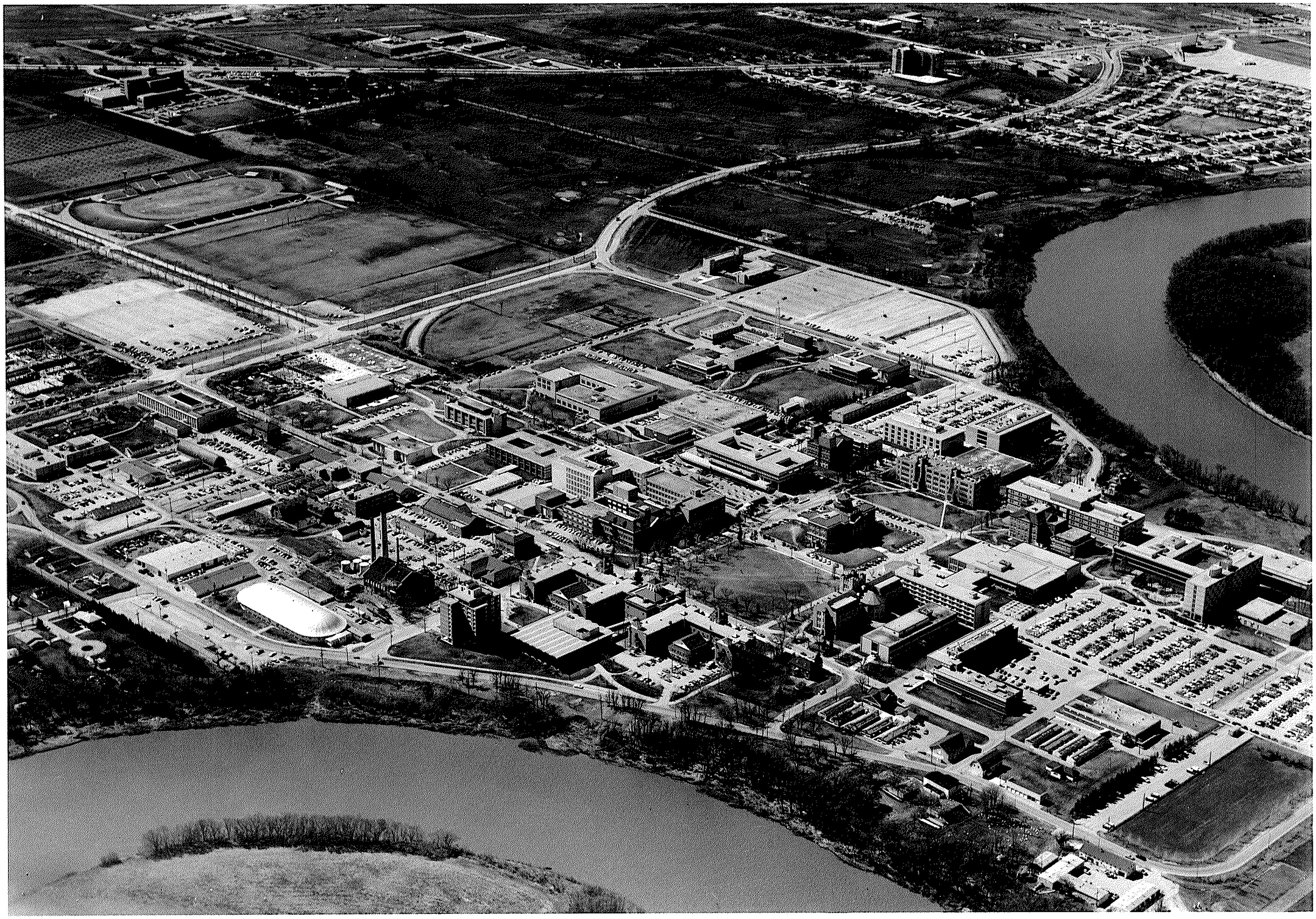

AERIAL VIEW UNIVERSITY OF MANITOBA CAMPUS

TABLE OF CONTENTS

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LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

A	Area
A_p	Base area of pile or pier
A_s	Perimeter area of the pile shaft
B	Width of footing
C_c	Compression index
c	Cohesion of soil
c_a	Adhesion between soil and pile
D_f	Depth of foundation
d_c	Depth factor
En	Manufacturer's maximum rated energy
e	Void ratio, coefficient of restitution
e_o	Void ratio under effective overburden pressure p_o
H	Thickness of stratum, height of fictitious footing measured from the bottom of the piles
H_p	Depth of soil shrinkage zone
I	Influence number
i_c	Inclination factor
K_o	Coefficient of lateral to vertical earth pressure
L_p	Effective length of pile
N	Standard penetration resistance
N_c	Bearing capacity factor for cohesion

N_γ	Bearing capacity factor for depth
N_q	Bearing capacity factor for surcharge
P	Resultant pressure, normal force
P_o	Present overburden pressure
P_o'	Maximum consolidation pressure on soil in field
Δ_p	Change in pressure
Q_a	Allowable load on pile
Q_{dn}	Net dead load
Q_1	Live load on footing
Q_p	Load carried in point bearing
Q_s	Load carried by friction along perimeter of pile
Q_t	Total effective load
Q_{ult}	Ultimate load on soil, ultimate pile capacity
q	Uniformly distributed load; surcharge per unit area
q_a	Allowable soil pressure
q_t	Total pressure on footing
q_{ult}	Ultimate bearing capacity of soil
q_{ult}'	Bearing pressure reduced due to eccentricity
q_d	Bearing capacity of the soil beneath the base of footing
q_{dr}	Ultimate bearing capacity of circular footing
q_{ds}	Ultimate bearing capacity of square footing

q_u	Unconfined compressive strength
R_d	Computed design capacity of pile
R_I	Reduction factor
r	Radius of circular footing
S	Settlement, penetration of pile under hammer blow
S.F.	Factor of safety
s	Sharing resistance
s_c	Shape factor
t	Thickness of footing
W	Weight
W_p	Weight of pile
W_r	Weight of ram of pile driver
z	Depth under centre of footing where q is to be evaluated
α	Reduction factor on strength of clay adjacent to shaft of pile
γ	Unit weight of soil
Δ	Increment
ϕ	Angle of internal friction
ϕ_a	Friction angle between soil and pile
σ_h	Lateral pressure along the pile shaft

ABSTRACT

This thesis contains a summary of data regarding engineering characteristics of the subsoils on the University of Manitoba Campus, together with such explanatory material as appears necessary for a general understanding of subsoil conditions on the Campus. The records of 52 test borings and many hundreds soil tests of various types are assembled and summarized in the quantitative terminology of modern soil mechanics.

The result of the study shows that the soil profile at the University of Manitoba Campus conforms generally to the typical Lake Agassiz deposit in the Winnipeg Area. The subsoils for the most part consist of a series of glacial clays of medium to stiff consistencies for thickness of about 50 feet underlain by 10 feet of glacial till on limestone bedrock.

The study also indicated under what condition it is practical to support light to medium weight structures on spread footings subject to certain precautions. Heavier

buildings and those structures where very little settlement or heave can be tolerated, were shown to be more appropriately supported on deep foundations, for example friction piles, and end-bearing piles or caissons resting on "hardpan" or bedrock. Furthermore, to make certain of successful design of any engineering structures on the Campus, precautions must be taken against the possible detrimental effects of soil volume change, frost heave, high percentage of harmful sulphates, and severe seepage which may occur in some locations.

CHAPTER I

INTRODUCTION

For at least nearly a hundred years, the local engineers have been exploring the subsoil of the Winnipeg area to obtain information for the design and construction of engineering works in the area. In 1937, the Committee on Foundations in Winnipeg recommended that research work on foundation problems both in the field and laboratory be carried out. The establishment of the Committee on Foundation in 1937 in Winnipeg marked the local transition period from traditional methods of describing the subsoil by physical outlooks to those now accepted as more suitable and scientific approaches. The advances in the concepts of soil mechanics and foundation engineering over the last sixty years are reflected in the design of foundations on the Campus.

1.1 Objective

This thesis examines subsoil conditions at the University of Manitoba Campus, with the primary objective to collect relavant data pertaining to design and construction of foundations and related engineering structures on the Campus. A more limited aim of this thesis is to compile and correlate data on the local soil properties in sufficient detail for use for new construction.

1.2 Scope

In order to accomplish the above mentioned purposes, a total of 52 test boring records and many hundreds soil tests of various types were assembled and summarized in the quantitative terminology of modern soil mechanics. The results are expressed in the form of test hole logs (Appendix A) and cross-sections (Figure 1.2, 1.3, and 1.4). This information is supplemented by a statement of the geological origin and summary of the properties of the subsoils.

In addition, foundation plans of some existing buildings on the Campus were studied with particular reference to each type of foundation used, including spread footings, friction piles, end-bearing piles, and caissons. Analysis of performance of selected foundations of each design were made using the data gathered by the author. Other related areas of foundation problems were also considered, such as basement floor upheaval due to soil swelling, frost heave, corrosion of metallic pipes, and attack on concrete by the chemical reaction of soluble sulphates.

1.3 Soil Profile

The University of Manitoba Campus, Figure (1.1), is built on the bed of the old glacial Lake Agassiz. Based on the test hole logs data, and typical cross-sections as shown in Figures (1.2), (1.3), and (1.4), the thickness of the clay strata between the surface grade and limestone bedrock is about 50 feet. These subsoil strata are divided into typical layers as follows:

(a) Roughly two feet of organic top soil.

(b) Four feet or so of brown silty clay,

(c) Tan silt which varies in thickness from a few inches to three or four feet.

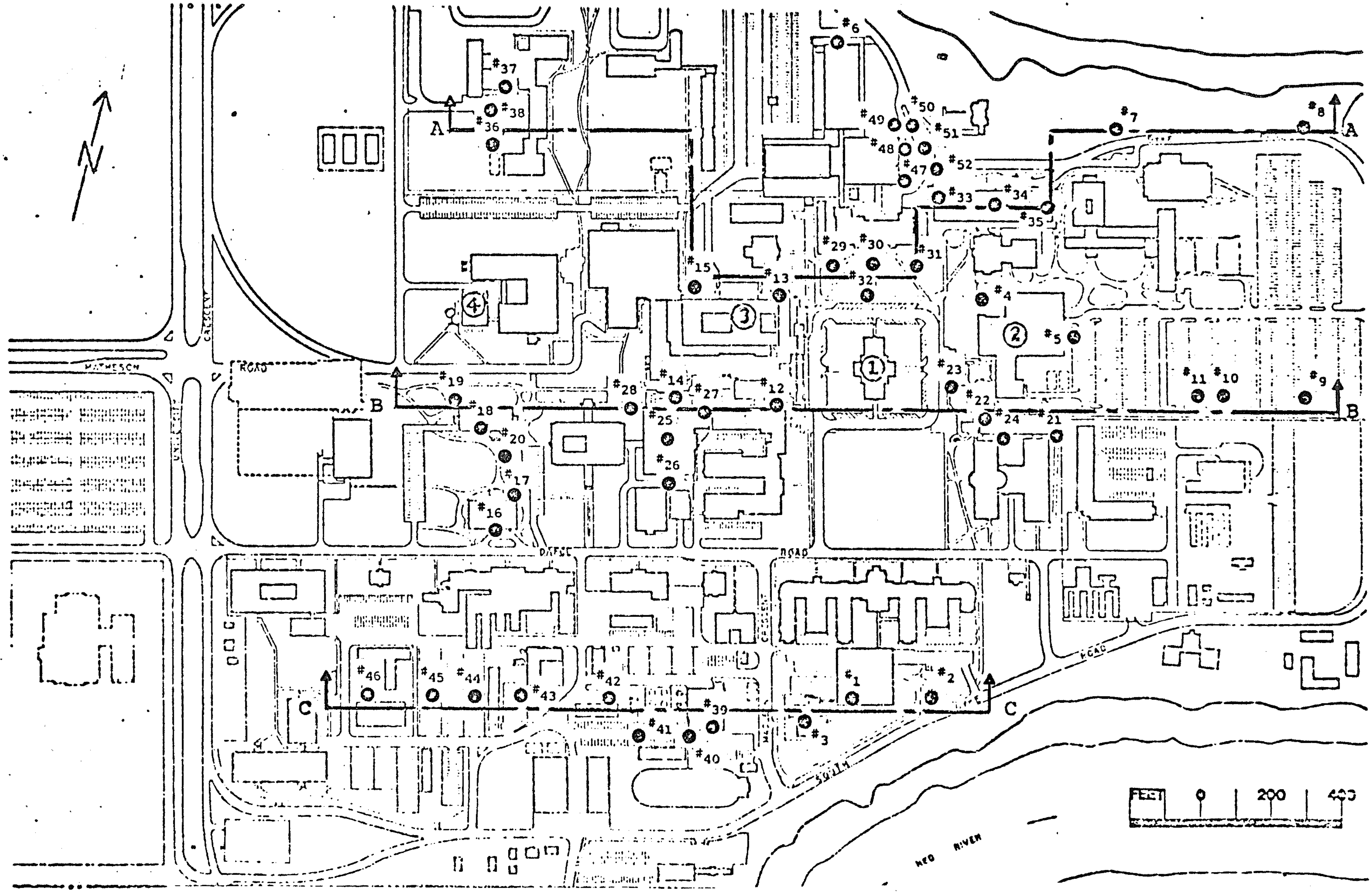
(d) Approximately twenty feet of dense brown clay.

(e) Some twenty-six feet of dense grey clay,

(f) Five to ten feet of glacial till.

(g) Ordovician limestone bedrock.

The typical soil profiles on the Campus are shown in Figures (1.2),(1.3),and (1.4).



LEGEND: ● Test Hole ① Administration Bldg. ② Elizabeth Dafoe Library
③ University Centre ④ Education Bldg. Addition

Figure 1.1 Location of Test Holes on University of Manitoba Campus

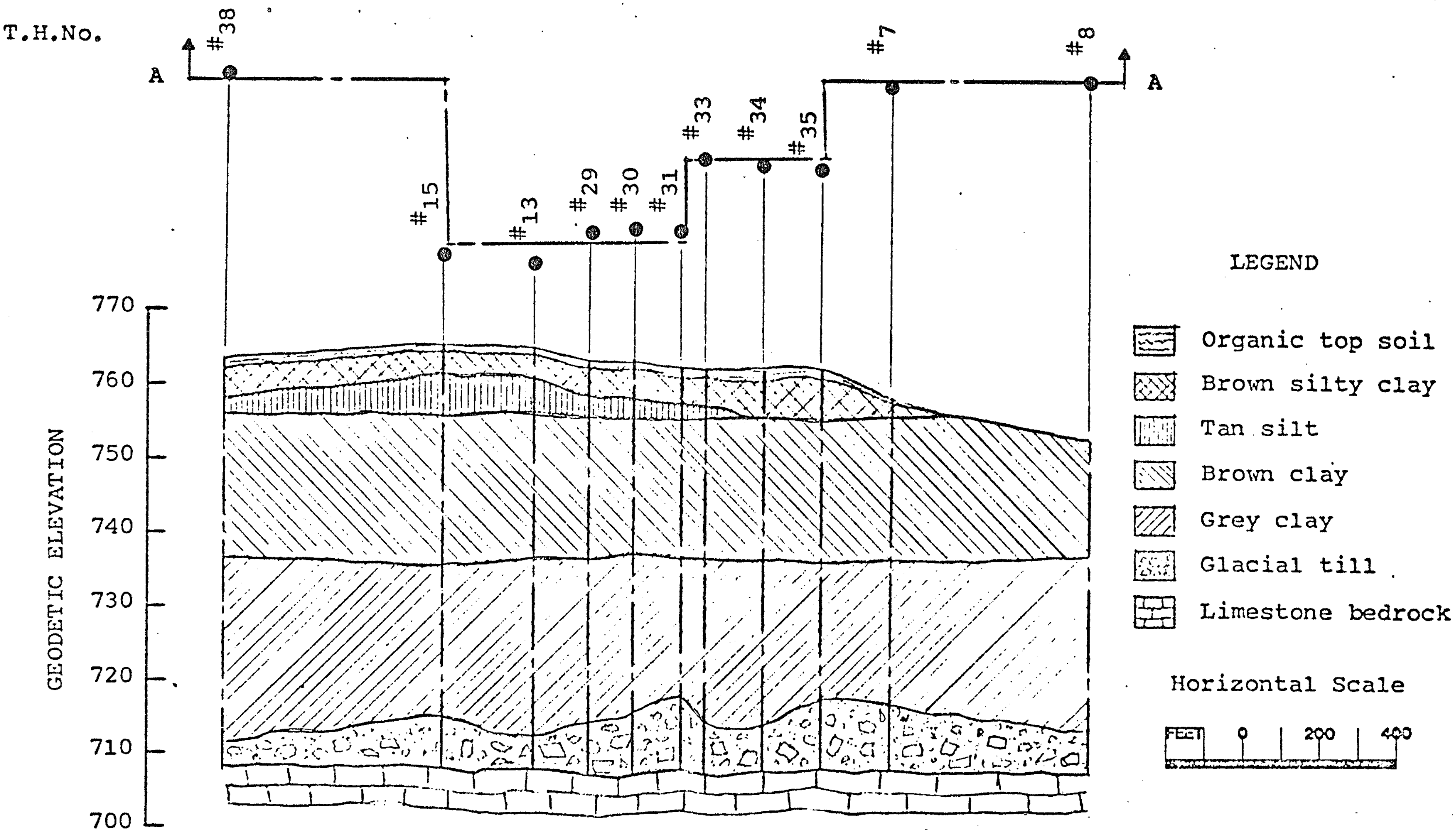


Figure 1.2 Soil Profile: section A - A

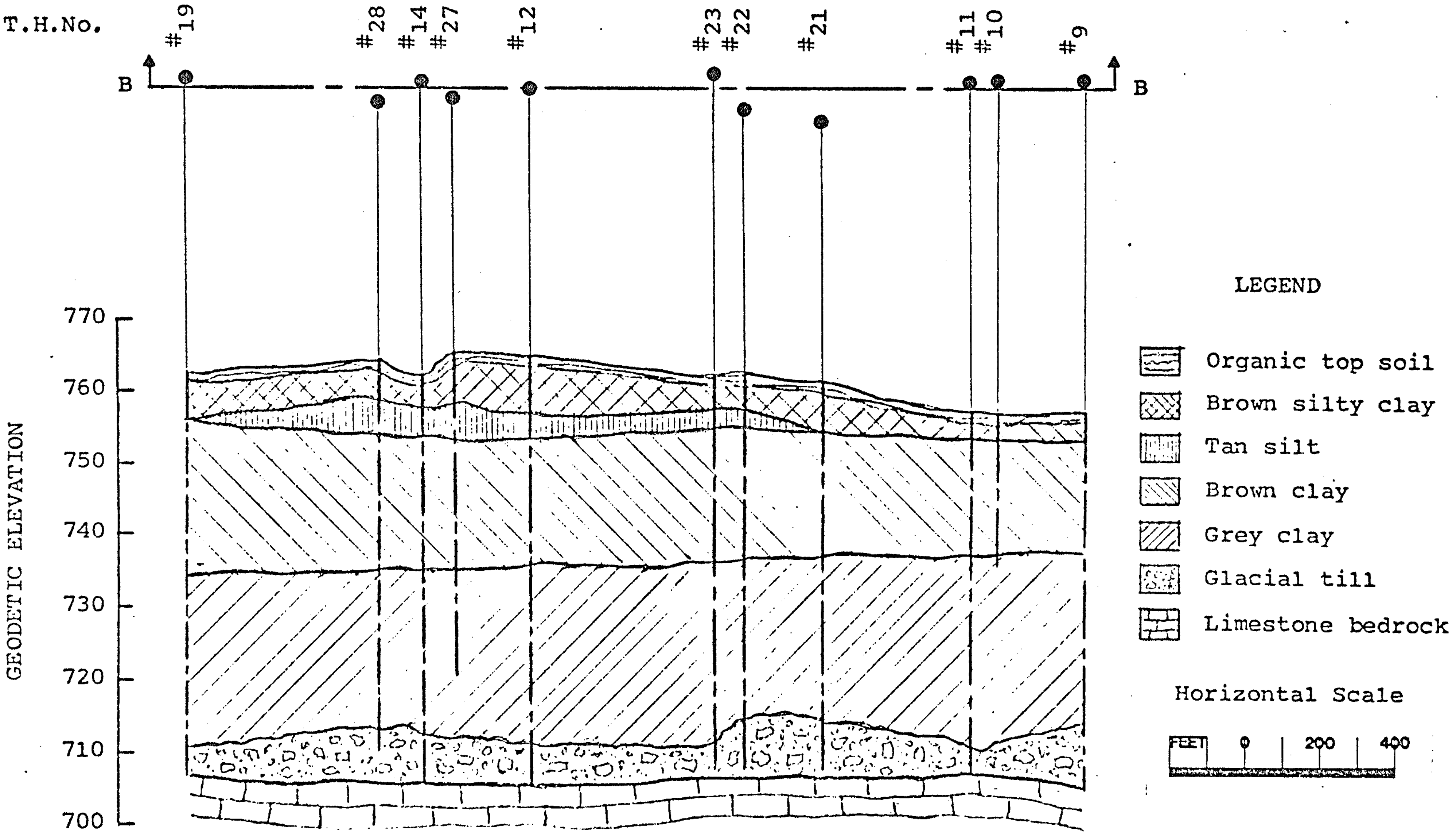


Figure 1.3 Soil Profile:section B - B

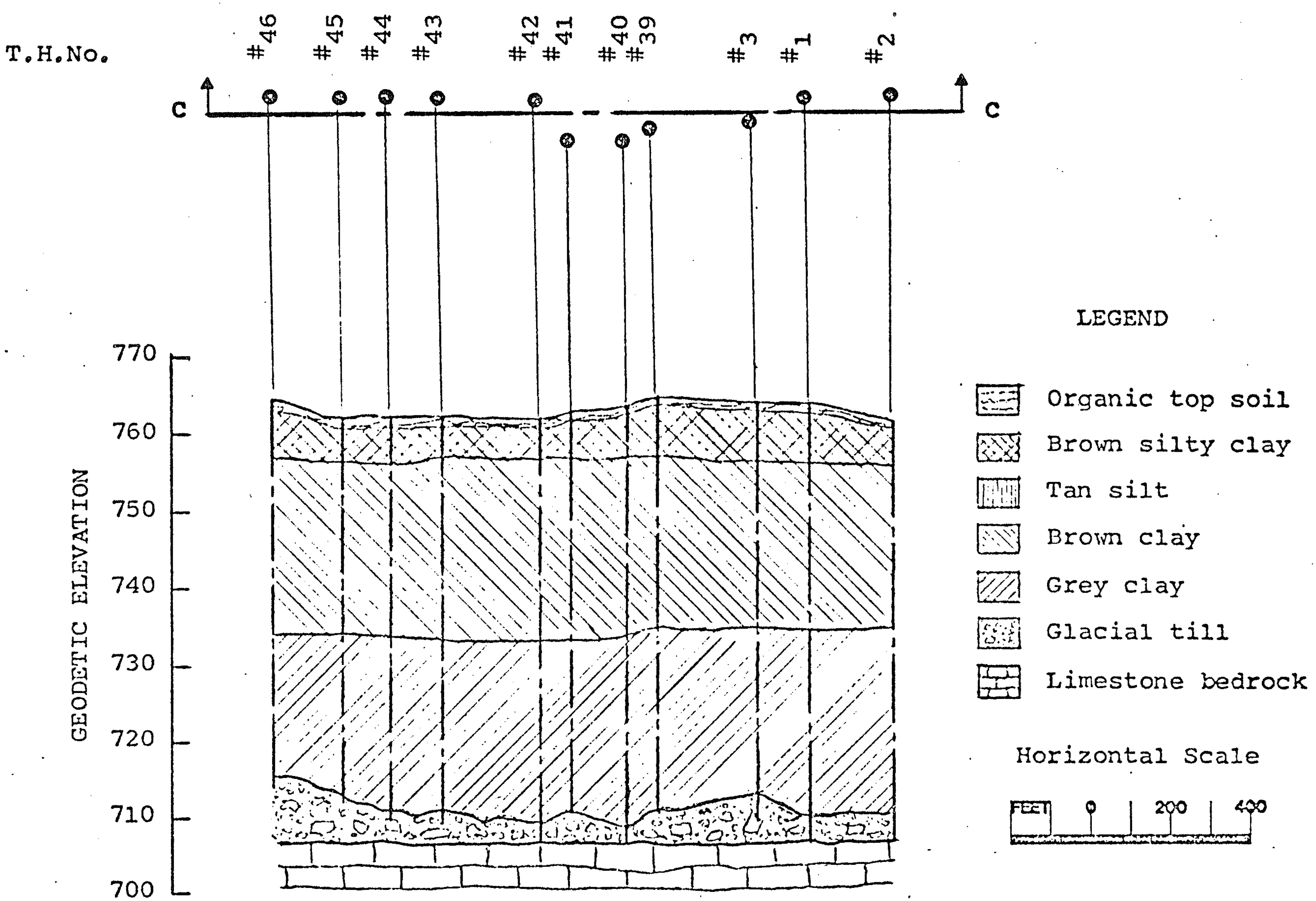


Figure 1.4 Soil Profile: section C - C

CHAPTER II

GEOLOGY OF THE WINNIPEG AREA

An understanding of the geological history of Manitoba provides a useful insight to the foundation conditions on the University of Manitoba Campus. The geology of the Winnipeg area has been studied and reported by: Upham,[39] Klassen,[21] Cherry,[10] Elson,[14] and Davies.[13] Only those features that have had an apparent significant influence in creating the present Winnipeg subsoils will be discussed here.

2.1 Bedrock

Greater Winnipeg lies in the Lake Agassiz basin which is known locally as the Red River Valley. This whole region rests upon the Pre-Cambrian granitic rock or the Canadian Shield, which out-crops and marks the limit of bedrock to the north and the east. To the west of the Shield lies a series of low gradient Paleozoic and younger sedimentary rock formation dipping westward. At the site of Greater Winnipeg itself, the underlying bedrock consists chiefly of Ordovician limestone. The present Winnipeg subsoils, which consists of thick layers of clay and silty clay, and about ten feet of till, occurred as a result of Pleistocene glaciation which in turn created, in its closing stage, the glacial Lake Agassiz and its deposits.

2.2 Pleistocene Geology of Manitoba

Elson[14] states that only the advance and retreat of the Wisconsin glacier of the most recent ice age has left deposits in sufficient quantity to affect foundation design in the Winnipeg Area. The flow of the Wisconsin glacier in Manitoba Area was southward as marked by the directions of striation on bedrock, drumlins, and boulder trains. As the ice sheet advanced, it smoothed the bedrock as well as adding shale and clay to the stones and rock-flour, and when the ice melted the glacial drift was left in a deposit of mixed, confused mass of clay, sand, gravel, and boulders, which is called till. Since the deposits of a glacier are determined largely by the material picked up in the vicinity, those in the Winnipeg Area are composed primarily of silt, sand, rockflour, gravel, and boulders, the derivatives of the Precambrian granite of the Canadian Shield and the Ordovician limestone outcroped near Winnipeg. As the Wisconsin glacier retreated northward and eastward, the meltwater from the glacier became impounded and formed the glacial Lake Agassiz between the high ground to the south and the ice to the north. Rivers also flowed into Lake Agassiz and resulted in the deposition of silts and clays.

2.3 History of Lake Agassiz

Lake Agassiz occupied the basin of the Red River and a wide region to the north. The earliest outlet was south-eastward through the Minnesota River Valley into the Mississippi. Deglaciation opened lower outlets across Ontario and Lake Superior and drained the Lake. Re-advance of glacier blocked those outlets and recreated the Lake, which was later drained and recreated a second time in a similar manner. The Lake grew to great size and ultimately disappeared as a result of drainage through newly opened northern outlets into the Hudson Bay, the last of which was the Nelson River system.

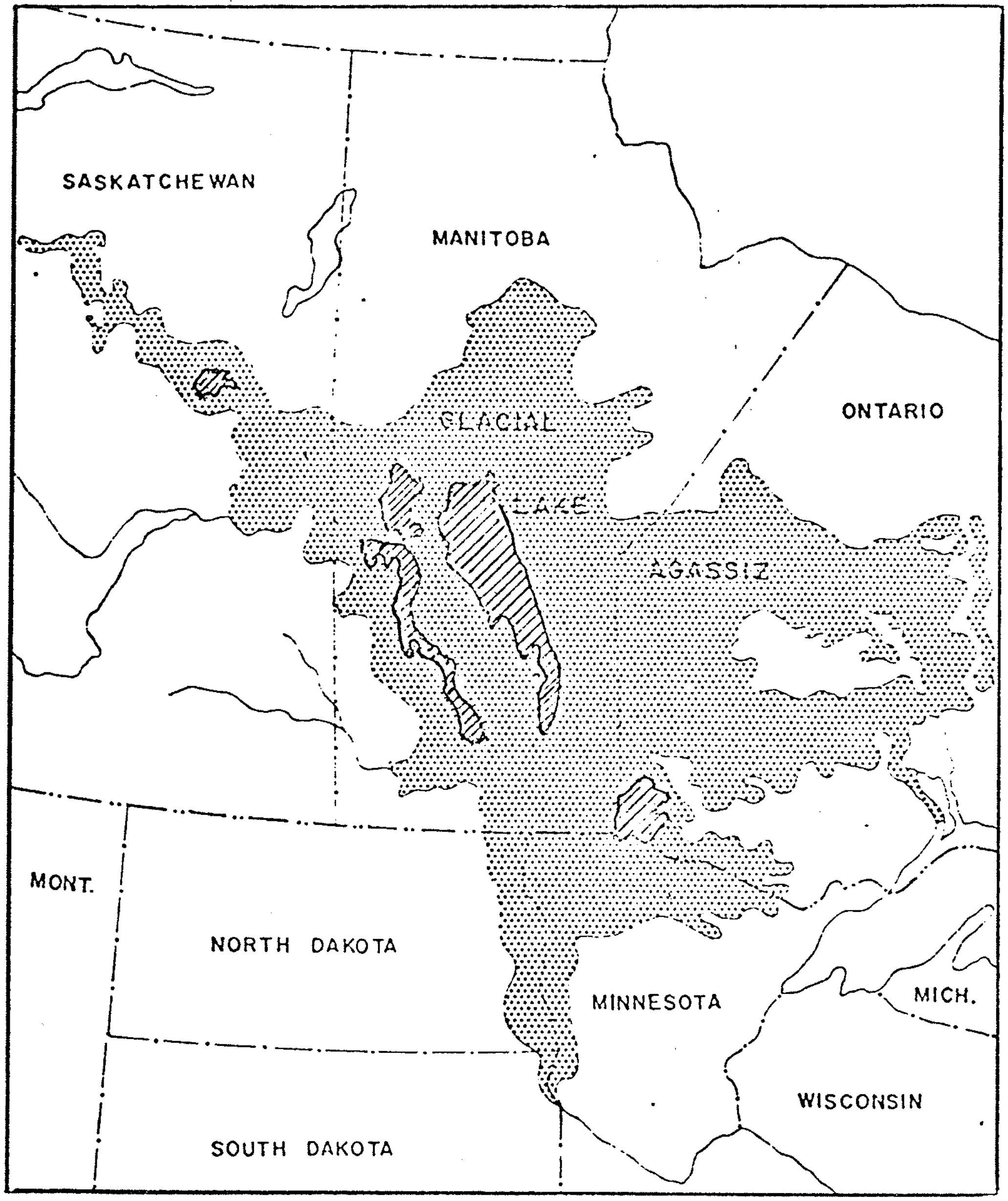


Figure 2.1 Schematic drawing, Glacial Lake Agassiz
After Elson[14]

2.4 Origin of the Winnipeg Subsoils

The formation of Lake Agassiz enabled fluvial and lacustrine materials to be deposited over the glacial till, which resulted in local smoothing of the Lake's bed. On the site of the University of Manitoba Campus the thickness of the fine grained lacustrine material is about fifty feet. In order of increasing depth, the material consists of four distinguishable layers: the brown silty clay, the tan silt, the brown clay and the grey clay. The origin of each individual stratum mentioned will be discussed in the following paragraphs. A few feet of organic topsoil and some fill, which sometimes cover the surface, are not discussed because these materials are generally removed before any construction, hence having no significant bearing on foundation designs.

The part of the Red River basin that contains deep water lake sediments is about 300 miles long from north to south, and 30 to 60 miles wide, with the north part the widest. Maximum water depths ranged from less than 200 feet south of Fargo to about 700 feet near Winnipeg. Lake

Agassiz received appreciable sediment from land areas, particularly to the west.

The grey and brown varved clays: During the high water level phases of Lake Agassiz, in a long episode of abundant sediment, the thin laminated grey clay and later the brown clay were deposited. The wave action eroded the ice-laid deposits along the shorelines, removed the finer material, transported and redeposited it in the Lake bottom. As the deposits increased in thickness, stratification occurred, and the clays were interbedded with thin layers of rockflour and silt or very fine sand. This episode of deposition took a long time, from about 13,000 to 9,000 B.P., as reflected by the thickness of the clay strata. The papery laminae of the clay structure suggest a remote ice margin and deep water during the entire period of deposition of the clays.

The tan silt: The "tan silt" bed occurs in many areas of Greater Winnipeg. The deposition of the silt represent an event of relatively short duration when there was an influx of sediment coarser than clay size into the northern

Red River basin.

Elson[14] interprets that possibly a lowering of the Lake level at the close of Lake Agassiz phase III by a few tens of feet caused incision by rivers and erosion along the newly exposed shores by active wave action. Later the Lake level rose and the silt unit was deposited.

The brown silty clay: These deposits overlying the tan silt are of more recent origin and suggest deposition in a period of rapid change when Lake Agassiz was being drained. The brown silty clay is quite variable both in its occurrance and degree of lamination.

Stratification and lamination of the clays in Lake Agassiz basin can be attributed to the summer and winter flows of the rivers which drained into the Lake. During the summer, flow of maximum discharge carried the heavier material into the Lake depositing silt layers. While in winter, flows of lesser discharge carried the lighter, finer material, and the water body was calm allowing clays to be deposited.

CHAPTER III

SIGNIFICANT SOIL PROPERTIES

In this chapter, the pertinent physical properties of the subsoil materials at the University of Manitoba Campus are described. To achieve this goal, the boring records done previously for site investigations were collected. A total of 52 test hole logs were examined and the information completely summarized as shown in Appendix A. Location of test holes are shown in Figure (1.1). Several hundreds of field and laboratory tests on undisturbed samples were also examined and summarized. Because of their lacustrine and fluvial origins, the Winnipeg soils are predominately clays and silty clays. For foundation purposes, the most significant properties are shear strength and compressibility. These and classifcation properties of the various subsoils found on the University of Manitoba Campus were studied and correlated. In so doing it was found that some values of the unconfined compressive strength were very low. This may be attributed to either the disturbance on the sample or the inaccuracy of the hand penetrometer being used on some tests. The significant properties of the subsoils are considered in order of increasing depth as follows:

3.1 Brown Silty Clay

Table (3.1) summarizes the properties of the brown silty clay. This stratum is generally found immediately below the organic top soil. It is generally first encountered at a depth of 2 feet and occurs usually as approximately a 5 feet thick layer. In its normal state, brown silty clay is stiff, fine grained, and practically impervious. It dries slowly, but in the process of dehydration it cracks and shrinks to considerable degree and becomes pervious. If allowed to dry and then subsequently wetted, it will swell. Ground supported floors heave under this action. Several inches of differential heave of floors in the UMSU Building occurred as a result of this cause following the 1950 flood.

The average unconfined compressive strength, based on 16 tests, was 2,250 pounds per square foot ranging from high and low values of 4,630 pounds per square foot and

Table 3.1 Brown Silty Clay

	Range	Average	No. of Tests
Depth of top of stratum (ft)	0-7	1.8	38
Depth of bottom of stratum (ft)	2-15	6.2	38
Moisture content (%)	21-45	33.3	22
Unit weight (lb/cu ft)	115-129	121	4
Degree of saturation (%)	93-100	98	4
Unconfined compressive strength (lb/sq ft)	700-4630	2250	16
Plastic limit (%)	15-30	22.2	4
Liquid limit (%)	35-84	62.2	4
Plasticity index (%)	20-54	40	4

700 pounds per square foot respectively. The very low value of 700 pounds per square foot probably represents damaged or disturbed samples. Otherwise low strength values generally occur near the bottom of the layer probably reflecting the higher water contents.

The average degree of saturation was 98 percent. The majority of the tests revealed complete saturation, but some of the tests had values as low as 93 percent. The lower values correspond to samples obtained at shallower depths. The high degree of saturation shown from tests at different times confirms the belief that this material is usually completely saturated and thus the angle of internal friction in terms of total stress may be taken as being negligible.

The results of four tests yielded an average value of the liquid limit of 62.2 percent. The average plastic limit and plasticity index were 22.2 and 40 percent respectively. On the basis of these limits, the brown silty clay has medium compressibility and plasticity, low permeability, and medium to high volume changes with changing moisture content.

3.2 Tan Silt

The tan silt is generally located immediately below the brown silty clay. It varies in thickness from a few inches to four or five feet. Twelve to eighteen inches is a fair average, but in some localities the material is not found at all. Table (3.2) summarizes the properties of the tan silt.

Because of the shallow depth involved, it is the usual practice not to build foundations in this silt. Worthy of note is its greater permeability in relation to that of clay. Seepage at shallow depth from the silt was shown by a number of the test holes and is probably seasonally affected.

Based on its Atterberg limits, the tan silt is classified as a soil which is frost-heave susceptible material, unsuitable below roadways or to support foundations subject to freezing. Its occurance is responsible for damage on many of the Campus roads, following the thaw each spring.

3.3 Brown Clay

Table (3.3) summarizes the properties of the brown clay stratum. The brown clay is generally laminated with

Table 3.2 Tan Silt

	Range	Average	No. of Tests
Depth of top of stratum (ft)	1-8	4.4	22
Depth of bottom of stratum (ft)	3-11	7.2	22
Moisture content (%)	21-50	35	2
Unit weight (lb/cu ft)	-	-	-
Degree of saturation (%)	-	-	-
Unconfined compressive strength (lb/cu ft)	800-1750	1275	2
Plastic limit (%)	19-20	19.5	2
Liquid limit (%)	24-25	24.5	2
Plasticity index (%)	-	5	2

Table 3.3 Brown Clay

	Range	Average	No. of Tests
Depth of top of stratum (ft)	0-14	6.5	52
Depth of bottom of stratum (ft)	8-35	26.1	52
Moisture content (%)	20-70	48.2	151
Unit weight (lb/cu ft)	104-118	109	17
Degree of saturation (%)	93-100	99	19
Unconfined compressive strength (lb/cu ft)	500-3920	1674	105
Plastic limit (%)	25-40	32	8
Liquid limit (%)	62-110	83	8
Plasticity index (%)	37-80	55	8

layers of silty or sandy material from which light seepage sometimes occurs, and usually contains a fair amount of gypsum pockets. Grain size analyses show that about 80 percent of the material consists of clay sizes and the remainder silt. The clay is somewhat over-consolidated, highly plastic, slightly fissured, highly impermeable and almost fully saturated. It generally lies below the tan silt but where the silt does not occur, it lies below the brown silty clay stratum. It is generally encountered first about 6.5 feet below the ground surface, but has been found on the surface in some localities and as deep as 14 feet in others. Generally, the layer has a thickness of approximately 20 feet.

The brown clay ranges in consistency from soft to stiff as indicated by the unconfined compressive strengths ranging from 500 and 3,920 pounds per square foot. On the basis of 105 tests, the average unconfined compressive strength was 1,674 pounds per square foot, which indicated that the material is generally of medium consistency. A study of the boring records shows that there is no definite variation in strength of the brown clay with depth except that higher strength values occur near the top surface of the layer.

The high average value of 90 percent for the degree of saturation indicates that the material is usually found in a saturated condition for which the undrained angle of internal friction is negligible.

The Atterberg limits indicate that the brown clay is highly plastic, highly compressible, practically impervious and subject to large volume changes with accompanying changes in moisture content.

3.4 Grey Clay

Table (3.4) summarizes the properties of the grey clay. The clay is distinctly laminated with many thin layers of silt spaced between thicker layer of clay. The grey clay is softer and siltier than the overlying brown clay. It generally has numerous calcareous silt pockets and contains limestone gravel at the greater depths. The material has been found as shallow as 8 feet and as deep as 35 feet and is generally first encountered at a depth of 23 feet. The thickness

Table 3.4 Grey Clay

	Range	Average	No. of Tests
Depth of top of stratum (ft)	8-35	23.3	52
Depth of bottom of stratum (ft)	36-55	49.4	48
Moisture content (%)	21-80	51.8	187
Unit weight (lb/cu ft)	102-127	111	28
Degree of saturation (%)	96-100	99.3	28
Unconfined compressive strength (lb/cu ft)	300-5180	1442	118
Plastic limit (%)	25-32	27.8	9
Liquid limit (%)	56-110	76.7	9
Plasticity index (%)	29-80	48.7	9

of the layer is usually about 26 feet.

Based on 188 tests, the average unconfined compressive strength was 1,442 pounds per square foot which corresponds to a medium consistency. The consistency had a range from very stiff to soft. As in the case of the overlying brown clay, there is again no difinite variation in the unconfined compressive strength with depth.

The grey clay is always found completely saturated as indicated by an average degree of saturation of 99.3 percent. This factor generally permits the undrained angle of internal friction to be assumed equal to zero when designing foundations.

Its Atterberg limits indicate a material of high plasticity and compressibility, very low permeability, and one that is subject to large volume changes with moisture changes. These properties, which are similar to those of the brown clay, are generally undesirable for satisfactory foundations. However, since this material generally lies below the depth of seasonal moisture variation, excessive volume changes generally do not occur, and hence where foundations have been rationally designed and conservatively loaded their performances will, in general, be satisfactory.

3.5 Glacial Till and Bedrock

Table (3.5) summarizes the properties of the glacial till. Beneath the clays are found glacial deposits of rock-flour, silt, sands, and gravel. The upper portion, deposited as the glacier receded, is sandy material containing a fairly large percentage of ground-up limestone and some boulders. Seepage from the till often occurs in sufficient volume to cause trouble. The bottom of this material is the limit to which a boring can be carried by the augers commonly used for drilling cast-in-place piles or caissons. The lower portion of the glacial till, sometimes described locally as hard-pan, is believed to have been acted upon by the full weight of the glacier. It is, therefore, highly consolidated and very dense. This material contains a high percentage of crushed limestone and boulders. It can be excavated with a pick only with considerable difficulty and has about the hardness and consistency of a poor quality concrete. When

Table 3.5 Glacial Till

	Range	Average	No. of Tests
Depth of top of stratum (ft)	36-55	49.1	45
Depth of bottom of stratum (ft)	45-58	54.1	45
Moisture content (%)	10-70	23.4	21
Unit weight (lb/cu ft)	110-143	127	2
Degree of saturation (%)	-	100	2
Unconfined compressive strength (lb/cu ft)	500-1610	1020	6
Plastic limit (%)	-	-	-
Liquid limit (%)	-	-	-
Plasticity index (%)	-	-	-

exposed and in contact with water, it disintegrates readily. The total thickness of the glacial till is about 5 feet but the value varies considerably from place to place. This stratum is generally encountered at the depth of about 50 feet below the ground surface.

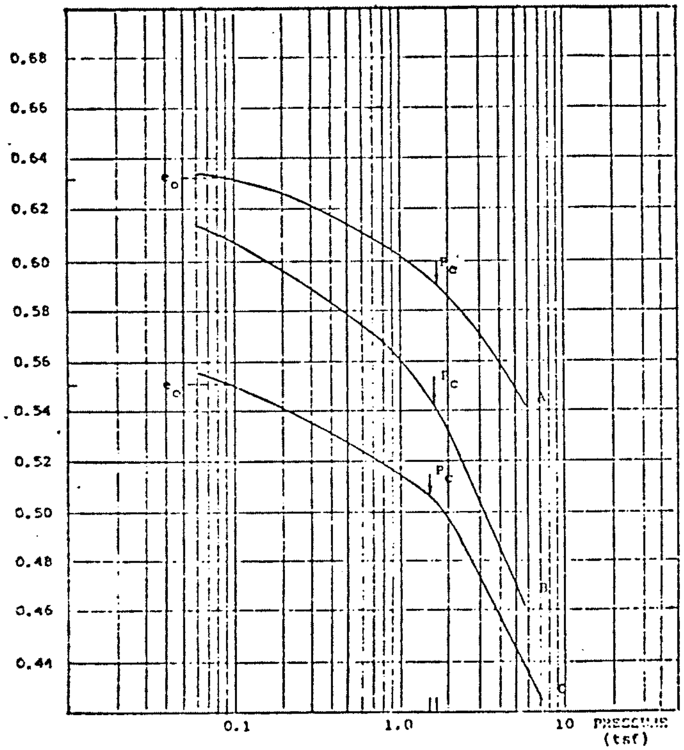
As a general rule the Ordovician limestone bedrock is encountered immediately below hardpan, as indicated by many test holes on the Campus. Sometimes a layer of water bearing sand, gravel, and decomposed limestone is found. This may provide a heavy flow of water under pressure which in turn may make impossible the use of cast-in-place piles in augered holes. Mindess[27] reported that this difficulty was experienced at the Rust Research Building where the former design using cast-in-place caissons had to be abandoned in favour of driven end-bearing piles.

3.6 Consolidation Characteristics of the Clays

The consolidation characteristics of the soils on the Campus were considered separately from the other properties. This was necessary since very few consolidation tests had been performed on Campus soils and recourse had to be made to results from other sites in the Winnipeg Area.

However, a few well performed test results on block samples were available for the tan silt and the brown clay sampled from the Education Building. These are shown in Figures (3.1), (3.2), and (3.3).

Consolidation curves representing the consolidation characteristics of typical Winnipeg Clays are also included, (Figure 3.4 and 3.5). It must be noted here that these tests were performed on soils from outside the University of Manitoba Campus, from the site of the Searle Grain Elevator which lies 1.5 miles east of Greater Winnipeg on the Trans-Canada Highway. Because of the common geological history of the Winnipeg clays discussed in Chapter II, it cannot be far wrong to presume that the discrepancy between these values and those of the clays on the Campus is small. Summaries of the typical consolidation test results for clays in Winnipeg Area are shown in Table (3.6) and (3.7).



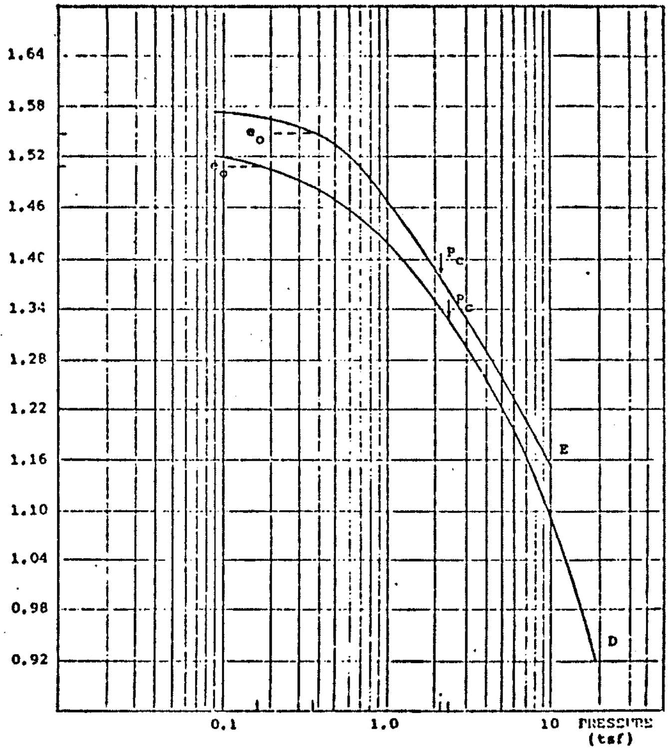
	Curve A	Curve B	Curve C
Natural Void Ratio	0.637	0.624	0.551
Moisture Content (%)	23	22.5	20
Compressive Index	0.11	0.17	0.13
Swelling Pressure (tsf)	0.08	0.0	0.09
Pre-consolidation Load (tsf)	1.80	1.75	1.62

Figure 3.1

Consolidation Test Results:

Tan Silt, Depth 6 ft.

(Education Building)



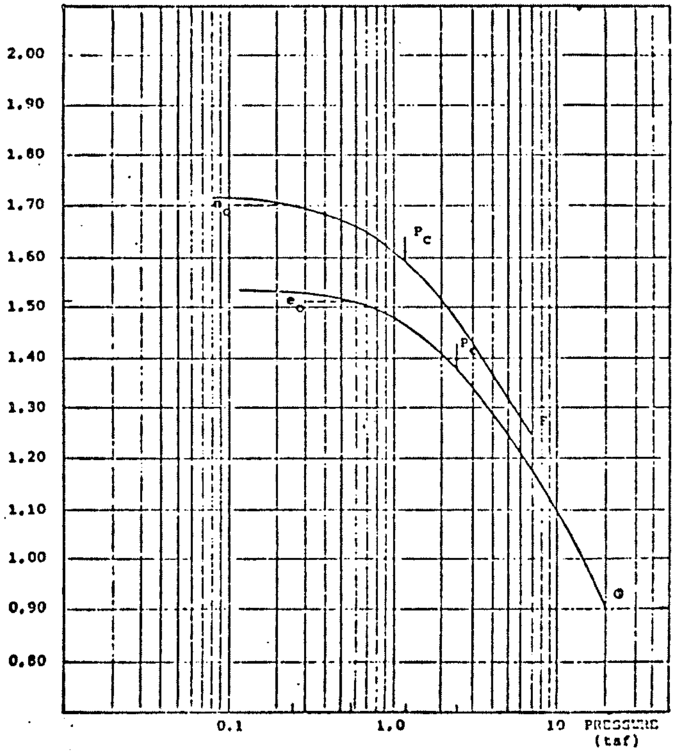
	Curve D	Curve E
Natural Void Ratio	1.512	1.545
Moisture Content (%)	54	56
Compressive Index	0.58	0.54
Swelling Pressure (tsf)	0.14	0.39
Pre-consolidation Load (tsf)	2.40	2.10

Figure 3.2

Consolidation Test Results:

Brown Clay, Depth 8 ft.

(Education Building)



	Curve F	Curve G
Natural Void Ratio	1.698	1.515
Moisture Content (%)	61	55
Compressive Index	0.52	0.61
Swelling Pressure (tsf)	0.24	0.58
Pre-consolidation Load (tsf)	1.2	2.4

Figure 3.3

Consolidation Test Results:

Brown Clay, Depth 16 ft.

(Education Building)

Figure 3.4 Consolidation Test Results: Depths 9 ft to 29 ft.
(Searle Grain Elevator)

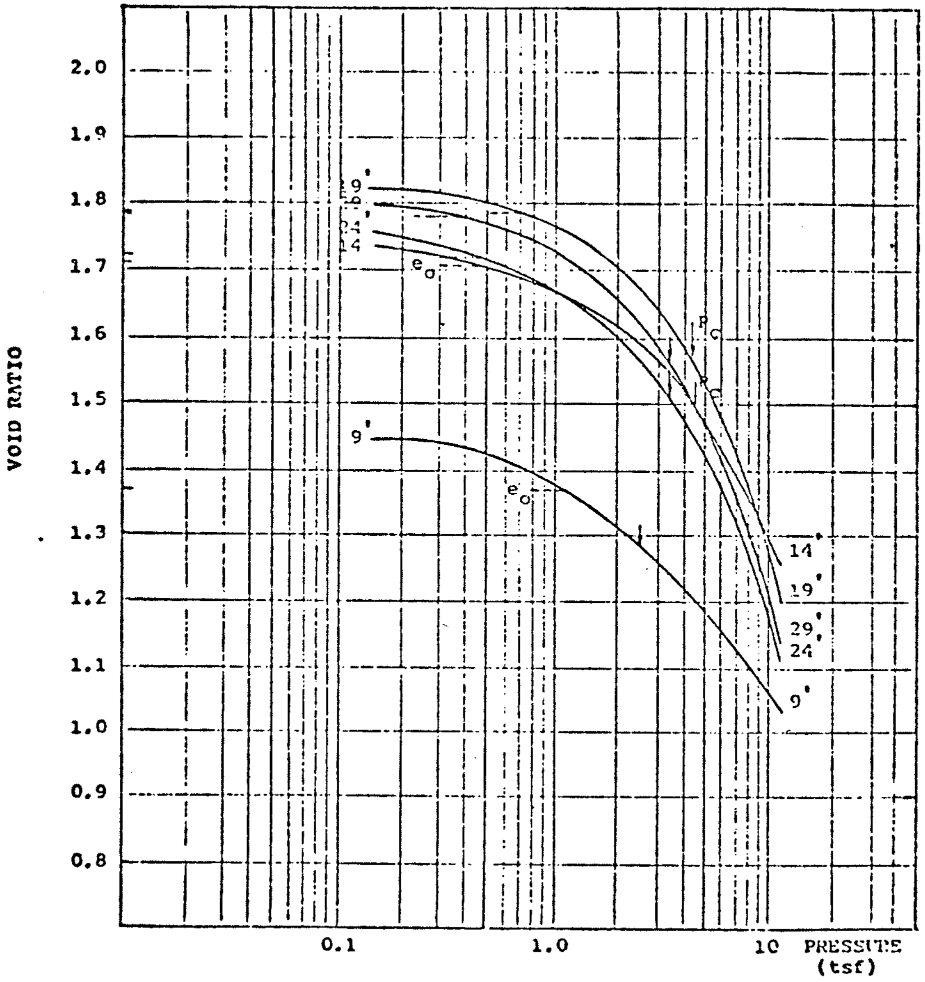


Figure 3.4 Consolidation Test Results
(Searle Grain Elevator)

Figure 3.5 Consolidation Test Results: Depths 34 ft to 54 ft
(Searle Grain Elevator)

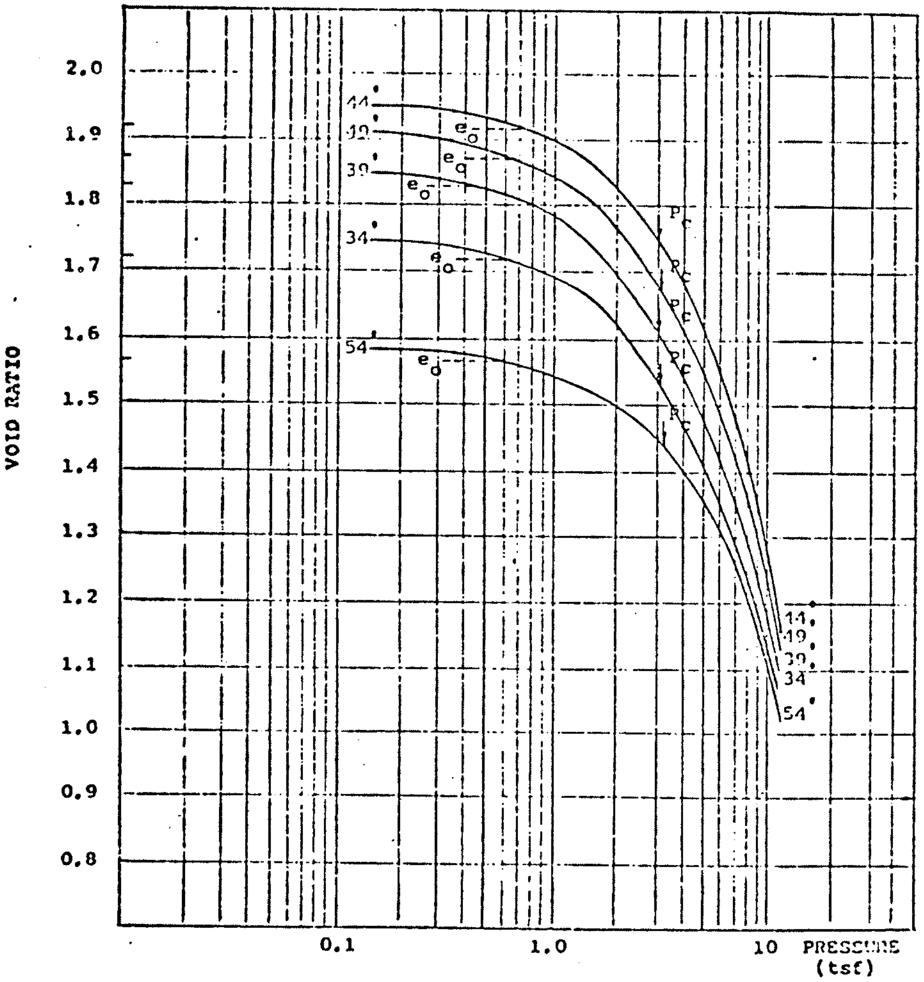


Figure 3.5 Consolidation Test Results
(Searle Grain Elevator)

Table 3.6 Typical Consolidation Test Results - Greater Winnipeg, Searle Grain Elevator.[22]

Depth (ft)	Natural Void Ratio	Compressive Index	Swelling Pressure (tsf)	Preconsolidation Pressure (tsf)
9	1.88	1.40	0.68	3.1
14	1.71	0.62	0.46	4.6
19	1.79	0.90	0.74	4.2
24	1.73	0.78	0.52	3.6
29	1.79	0.98	0.52	3.2
34	1.72	1.10	0.66	3.0
39	1.83	1.24	0.48	3.1
44	1.92	1.44	0.72	3.0
49	1.88	1.40	0.68	3.1
54	1.58	1.02	0.56	3.0

Table 3.7 Typical Consolidation Test Results - Greater Winnipeg,Transcona Grain Elevator.[2]

Depth (ft)	Natural Void Ratio	Compressive Index	Swelling Pressure (tsf)	Preconsolidation Pressure (tsf)
11	1.58	0.63	0.50	2.9
16	1.61	0.75	0.60	5.0
21	1.41	0.66	0.58	4.1
25	1.36	0.54	1.02	4.9
30	1.33	0.45	0.71	4.0
35	1.12	0.28	0.28	3.5

CHAPTER IV

THEORY AND DESIGN OF SHALLOW FOOTINGS

4.1 Historical Background

On the University of Manitoba Campus, those buildings built prior to 1914, such as the Administration Building, Tache Hall, and the Geology Building were all supported on spread footings. The major buildings of the later years, however, have made use of deep foundation types such as piles, pier or caissons, while the smaller or lighter structures still made use of the shallow spread footings as dictated by economy. Table (4.1) gives the types of foundations used in existing buildings on the Campus.

4.2 Bearing Capacity

The term bearing capacity of a soil is the ability of the soil to carry a load without failure within the soil mass and also denotes a loading intensity which the soil can sustain without such deformation as would result in settlement damaging to the structure. Prior to Terzaghi's work, the design of shallow foundations was mainly empirical

Table 4.1 Types of Foundation Being Used in Existing Buildings on the Campus

Name of Building	Date of Construc.	Type of Foundation			
		Spread Ftg.	Pile		Caisson or Pier
			Friction	End Bearing	
Administration Building	1911	X			
Agriculture Engineering Building		X			
Agricultural & Agric. Science Building		X			
Allen Building (Physics)	1959				
Animal Sciences Building	1960			X	
Animal Science Equipment Shed		X			
Architecture Building	1958				X
Armes Building (Science Lectures)					X
Barber Shop & Beauty Parlour		X			
Beef & Cattle Barn		X			
Buller Biological Laboratories	1931			X	
Canada Department of Agriculture					
Research Station				X	
Garages & Storage		X			
Green-house		X			
Phytotion & Service		X			
Rust Research Annex		X			
Volatile Storage		X			
Forestry Research	1959	X			
Receiving Station		X			
Constable's Residence		X			
Cyclotron				X	
Dairy Barn		X			
Diary Science Building		X			

Table 4.1 Continued

Name of Building	Date of Construc.	Type of Foundation			
		Spread Ftg.	Pile		Caisson or Pier
			Fric-tion.	End Bearing	
Education Building	1961		X		
Elizabeth Dafoe Library	1952	X			
Library Addition	1962			X	
Engineering Building		X			
Engineering Addition (North Wing)	1947	X			
Engineering Addition (South Wing)			X		
Engineering Addition	1966			X	
Farm Residences				X	
Feed Mill					
Fetherstonhaugh High Voltage Laboratory	1955		X		
Fletcher Argue Lecture Theatre	1965			X	
Fletcher Argue Building (Arts)	1965			X	
Food Science Building					
Fur & Game Research Station		X			
Geology Core Storage Building		X			
Geology Building		X			
Home Economic Building		X			
Home Economic Addition					
Hut "J" (Environmental Studies)		X			
Implement Shed		X			
Isbister Building (Commerce)	1960			X	
Mary Speechly Hall (Women's Residence)	1962			X	
Music Building				X	
N. E. Multi-purpose Building	1971			X	

Table 4.1 Continued

Name of Building	Date of Construc.	Type of Foundation			
		Spread Ftg.	Pile		Caisson or Pier
			Friction	End Bearing	
Old Animal Science Building		X			
Parker Building (Chemistry)					X
Pembina Hall	1962	X			
Pharmacy Building	1961			X	
Plant Science Garages & Stores		X			
Poultry Building		X			
Power House		X			
New Smoke Stack				X	
Cooling Addition				X	
President's House		X			
Robson Hall (Law)	1968		X		
Rifle Range		X			
Rink		X			
St. Andrew's College	1961				
St. John's College & Residence	1957		X		
St. Paul's College & Residence	1957				X
School of Art	1964			X	
School of Music	1964			X	
Sheep Barn		X			
Soil Science Equipment Shed		X			
Stock Judging Pavilion		X			
Swimming Pool & Athlete Centre				X	

Table 4.1 Continued

Name of Building	Date of Construc.	Type of Foundation			
		Spread Ftg.	Pile		Caisson or Pier
			Friction	End Bearing	
Swine Barns		X			
Tache Hall (Men's Residence)		X			
Tier Building (Arts)	1930				
University Centre	1970			X	
UMSU Gymnesium		X			
University College & Residence	1963			X	
Vice-President's Residence		X			
Zoo-Psychology Building (Duff Roblin)	1969			X	

and it was based largely on allowable bearing pressures for various soil types. From their data of construction, there is no question that the older buildings were designed on this basis. In foundation design, the designer may get the bearing capacity value of the soil applicable to the soil type at the site of construction either from the local building code, or by calculating the ultimate bearing capacity using one of the many bearing capacity theories.

4.2.a Bearing Capacity from Building Codes

The first building code for the City of Winnipeg was passed as early as 1875. Later, By - Law 16187 was passed on June 16, 1947. An excerpt of this by law, dealing with permissible loads on foundations, is presented in Table (4.2). Bearing values are associated with a soil identification based on colour, "dryness or wetness", and generally not on the pertinent properties of strength and compressibility. A rather strange distinction is made between industrial or commercial buildings, and those for human habitation. A higher safety factor was required for the latter. Undoubtedly many of the older buildings on the Campus were designed

Table 4.2 Permissible Load for Foundation Footings and Substructures. (City of Winnipeg Building By-Law, 1947)

	A (tsf)	B (tsf)
Blue Clay with no underlying strata of yellow or brown Clay	2	1.5
Mixed Clay - Moderately dry	1	0.75
Soft wet Clay or Silt	0.5	0.38

Notes: Column A is for industrial and commercial buildings.

Column B is for buildings for human habitation.

on the basis of the 1947 Building Code, or the prior experience which led to the formulation of the Code.

The City of Winnipeg Building code of 1965, was modelled after the National Building Code of Canada. As can be seen from Table (4.3), which summarizes the allowable soil pressures for foundation design. The identification of the soil type is more precise, and the pertinent property of strength is more realistically considered in terms of density for granular soils, and in terms of softness, stiffness, etc. for cohesive soils. In keeping with technical advances, the 1965 Building Code also permits bearing capacities to be based on soil mechanics investigations, including both field and laboratory testing. Newer buildings on campus have employed these modern provisions of the Code, and based bearing values on what can be theoretically justified.

4.2.b Bearing Capacity based on Theoretical Analysis

In recent years, several similar theories have been developed for obtaining the ultimate bearing capacity of soils. They include those by: Terzaghi,[36] Meyerhof,[24] Hansen,[16] Feda,[15] Lee,[22] Balla,[1] and Hu.[20] These theories

Table 4.3 Allowable Soil Pressure to Use in Foundation Designs.

(City of Winnipeg Building Code-1965)

Type and Condition of Soil or Rock	Design Bearing Pressure (psf)
Cohesionless Soils	
Dense sand, dense sand-and-gravel	6,000
Cohesive Soil	
Firm silt	1,000
Soft silt	500
Stiff clay	3,000
Firm clay	2,000
Soft clay	1,000
Hard till or Hardpan	15,000
Limestone Bedrock	
Sound	60,000
Soft or Shattered	20,000

Definitions of words used in Table (4.3)

Cohesive Soil:

"stiff" is a soil difficult to indent with the thumb; with difficulty it can be remoulded by hand.

"firm" is a soil that can be indented by moderate thumb pressure.

"soft" is a soil that can be penetrated several inches with the thumb.

make various assumptions concerning the shape of the failure surface in the soil, and the roughness etc. of the base of the foundation. Factors to account for the depth of footing, shape of footing, and inclination of loads are introduced where applicable. Due to the fact that the General Bearing Capacity Equation proposed by Hansen incorporates more general loading conditions, shape and size of footing; the method will be used for analysing soil bearing value in the subsequent chapter.

According to Hansen, the ultimate soil pressure may be taken as:

$$q_{ult} = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \tfrac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma \qquad (4.1)$$

where: q_{ult} = ultimate soil bearing pressure,

c = cohesion of soil,

ϕ = angle of internal friction of soil,

D_f = depth of footing below adjacent ground surface

B = least lateral dimension of footing,

γ = unit weight of soil,

s_c, s_q, s_γ = shape factors,

d_c, d_q, d_γ = depth factors,

i_c, i_q, i_γ = inclination of load factors.

Equation (4.1) written in terms of net pressure for saturated clay under undrained loading reduces to:

$$q_{ult\ net} = cN_c s_c d_c i_c \quad (4.1.a)$$

The bearing capacity factors are:

$$N_q = \tan^2(45 + \frac{\emptyset}{2})(e^{\tan \emptyset}) \quad (4.2)$$

$$N_c = (N_q - 1)\cot \emptyset \quad (4.3)$$

$$N_\gamma = 1.8\ (N_q - 1)\tan \emptyset\ \text{(approx.)} \quad (4.4)$$

The typical values of these bearing capacity factors are given in Table (4.4)

For a footing eccentrically loaded, according to the concept of useful width, the approach is to apply the reduction factor to the computed ultimate bearing capacity. The ultimate bearing capacity is computed by Equation (4.1) as before, and then multiplied by a reduction factor from Figure (4.1).

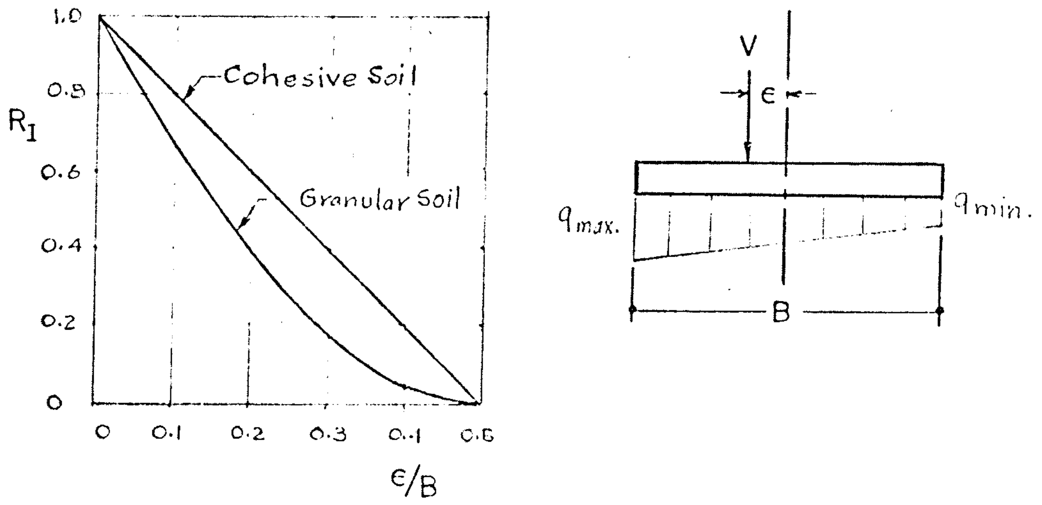


Figure 4.1 Reduction Factors For Eccentric Loading

Table 4.4 Bearing Capacity Factors N_c, N_q, and N_γ (after Hansen)

ϕ	N_c	N_q	N_γ
0^o	5.14	1.00	0.0
5	6.48	1.57	0.09
10	8.34	2.47	0.47
15	10.97	3.94	1.42
20	14.83	6.40	3.54
25	20.72	10.66	8.11
30	30.14	18.40	18.08
35	46.13	33.29	40.69
40	46.13	64.18	95.41
45	138.89	134.85	240.85
50	266.89	318.96	681.84

Table 4.5 Approximate values of the shape, depth, and inclination factors.

Shape factors	s_c	s_q	s_γ
Shape of base			
Continuous strip	1.00	1.00	1.00
Rectangle	$1 + 0.2B/L$	$1 + 0.2B/L$	$1 - 0.4B/L$
Square	1.3	1.2	0.8
Circle	1.3	1.2	0.6*
	Limitation $B \leq L$		
* Use B = diameter.			
Inclination factors	i_c	i_q	i_γ
	$1 - \frac{H}{2cBL}$	$1 - \frac{1.5H}{V}$	i_q^2
	Limitation $H \leq V \tan \delta + cBL$		
where $\tan \delta$ = coefficient of friction between footing base and soil c = cohesion between footing and soil L = length of footing parallel to H			
Depth factors	d_c	d_q	d_γ
	$1 + \frac{0.35D}{B}$	$1 + \frac{0.35D}{B}$	1.00
	Take $d_q = \begin{cases} d_c & \text{for } \phi \quad 25^\circ \\ 1.0 & \text{for } \phi = 0^\circ \end{cases}$		

Hence $q'_{ult} = q_{ult} (R_I)$ (4.5)

Where: q'_{ult} = reduced bearing pressure due to eccentricity,

q_{ult} = ultimate soil bearing pressure,

R_I = reduction factor.

4.3 Settlements

In addition to ensuring adequate safety factor, foundations cannot be permitted to undergo excessive total and differential settlement. The term settlement is used to describe the vertical displacement of the base of a structure. Although the causes of settlement are many and varied and may include the effects due to static loads, moving loads, changes in moisture content and the effects of undermining; the major cause of settlement, however, comes from static or compressive loading such as those imposed by the weight of a structure. The settlement caused by loading may be divided into two kinds: firstly, immediate settlement which is a combination of elastic compression and plastic deformation and occurs without change in volume or water content: secondly, settlement due to consolidation which is the result of the decrease in the volume of the loaded soil caused by the gradual expulsion of water from the voids. In clay soils consolidation settlement usually develops very slowly but may attain considerable magnitude in course of time. If the clay is saturated the settlement may be computed from Equation (4.6) below:

$$S = \frac{C_c H}{1 + e_o} \log \frac{p_o + \Delta_p}{p_o} \qquad (4.6)$$

where: S = consolidation settlement,

C_c = compression index,

H = thickness of stratum,

e_o = void ratio of soil,

p_o = overburden pressure,

Δ_p = change in effective pressure.

For clay soils, settlement calculated by Equation (4.6) seems to include both immediate and consolidation components of settlement. Skempton and Peck[35] showed that the discrepancy between the measured and calculated values of settlement in undrained condition of clay soils is in the neighbourhood of 9 percent. This indicates that the magnitude of settlement, as computed from consolidation theory, normally provides a satisfactory answer for the total expected settlement.

It has been found that when the soil is heavily preconsolidated, the settlement which will occur is so small that settlement analysis is rarely of practical interest. If the soil is slightly preconsolidated, it is found out that the magnitude of settlement computed by Equation (4.6) is theoretically higher than would the actual value due to the neglecting of the effects of preconsolidation, and due to the fact that the compressive index,C_c, used came from the virgin compressive branch of the consolidation curve. A more realistic value of settlement may be computed by using the preconsolidation pressure rather than the existing overburden pressure. Thus Equation (4.6) becomes:

$$S = \frac{H}{1 + e_o} \cdot \Delta e \qquad (4.7)$$

In this case,Δe is the change in void ratio between initial and final pressures taking into account preconsolidation and obtained directly from the curves.

The settlement of spread footings resting on clay increases roughly in direct proportion to the base width, and depends to a considerable extent on the net pressure under the foundation. This pressure can be reduced by excavation and the construction of basements.

The removal of soil tends to produce heave at the bottom of an excavation. In the case of clays, the consolidation component of heaving is a slow process and can continue long after the completion of the structure. Foundations supported on the base of the excavation tend to move downward under the effect of pressure increase from the footings, but upward under the effect of pressure reduction from the excavation. The net effect is a desirable reduction of settlement.

4.4 Conventional Design Method

The design of foundations consists of determining the elevation, size, shape, and structural details of the foundation structure. These aspects of foundation design are given in detail in standard textbooks on the foundation engineering and are therefore not included in this thesis. Use in this thesis has been made of the concept of net loads or pressures as distinguished from total loads or pressures, in determining the size of footings. This practice has been advocated by more recent authors such as Peck, Hansen and Thornburn.[29]

In calculating the net load on a foundation, the equivalent weight of soil of the volume occupied by the foundation members below adjacent ground surface is subtracted from the total of dead and live loads.

The total load, Q_t, is given by:

$$Q_t = Q_d + Q_1 \qquad (4.8)$$

where: Q_d = dead load on foundation including weight of foundation,

Q_1 = live load on foundation.

The net load, Q_n, is given by:

$$Q_n = Q_t - Q_v \qquad (4.9)$$

where: Qv = equivalent weight of soil of volume occupied by the foundation members below adjacent ground surface.

Total and net pressures are obtained by dividing the respective loads by the base area of the foundations.

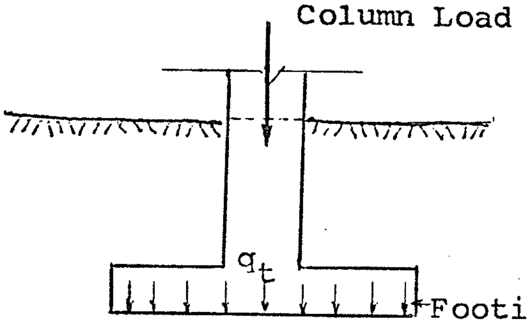
$$q_t = \frac{Q_t}{A} \qquad (4.8.a)$$

$$q_n = \frac{Q_n}{A} \qquad (4.9.a)$$

where: A = base area of footing.

In the case of a concrete spread footing as shown in Figure (4.2), it can be readily shown that Equation (4.9.a), can be approximated by:

$$q_n = \frac{\text{Column Load}}{A} + t\,(\gamma_{conc.} - \gamma) \qquad (4.10)$$



Where: t = thickness of footing,

γ_{conc} = unit weight of concrete,

γ = unit weight of soil.

Figure 4.2 Net Bearing Pressure under footing

The concept of net pressure is also applied to the theoretical ultimate bearing capacity. A term γD_f is subtracted from the ultimate bearing capacity to give the net ultimate bearing value. This value is then divided by a safety factor to give an allowable net pressure.

The determination of the net allowable pressure is a most critical step in the design process. The allowable soil pressures used in design should satisfy the two requirements that the factor of safety against soil failure should be adequate and the settlement produced by the load should be within tolerable limits. The factor of safety against the breaking of a footing through clay depends on the shearing resistance of the clay. In the undrained condition, the clay behaves as if "Ø" were equal to zero and as if the cohesion "c" were approximately equal to one-half the unconfined compressive strength of fairly undisturbed samples.

A primary requirement is that the base of the foundation should be located below the depth to which the soil is subject to seasonal volume changes caused by alternate wetting and drying. Baracos and Bozozuk[5] reported that significant seasonal soil moisture changes to a depth of 10 to 12 feet have been observed in Winnipeg. Withdrawal of water from the ground by the roots of trees has also been responsible for detrimental differential settlement. The base of

Table 4.6 Safety Factor According to Soil Conditions and Types of Loading (After Skempton)[33]

Conditions	Safety Factor
Temporary structure where some risk of a bearing failure can be tolerated	1.5
Where there is a large component of live load that is unlikely to develop	2.0
Where there is reasonably accurate soil and loading information	2.5
Where soil conditions are not well established	2.5
Where there are questionable conditions	4.0

the foundation should also be located below the depth to which soil may be weakened by cavities produced by borrowing animals. In regions of cold weather the foundations of the outside columns or walls should be located below the level to which frost may cause a perceptible heave. The choice of the appropriate depth of foundations may depend on a number of other considerations, such as the basemet depth required to reduce the net pressure on the base of the footing, and limitations of depth due to high ground water level, the presence of rock, and the location of adjacent foundations.

CHAPTER V

ANALYSES OF TYPICAL SPREAD FOOTING DESIGNS

5.1 Method of Analysis

Buildings where spread footings have been used on the Campus were examined. Selected for specific analysis were the Administration Building representing older nonreinforced concrete spread footings, and the Elizabeth Dafoe Library representing more modern reinforced concrete spread footings. In each of these cases, a typical footing was checked for the net pressure acting on the base of the footing, and the theoretical net bearing capacity based on the soil conditions prevailing at the site. The foundation loads were calculated according to the 1965 Winnipeg Building Code. Timber Design Handbook,[9] and Steel Construction Handbook,[8] were used to supply the unit weight of materials. The safety factor with respect to the net bearing capacity was determined. In addition, consolidation test data were examined with regard to possible settlement or heave.

5.2 Spread Footing Bearing Capacity and Settlement Analyses (Administration Building)

The Administration Building was built in 1911 to accommodate the Agricultural College. It is a four-story masonry structure, 58 by 197 feet in plan, and 63 feet high. The structure was founded on a combination of bearing wall footings and individual spread footings. The elevation of each foundation is 10 feet below surface grade excluding 4 feet of fill east and west of the building. The massive outside walls, the thickness of which ranges from 16 inches at the top two floors to 32 inches in the basement, is supported on a continuous footings of 7 feet 8 inches wide. The concrete floor slabs were supported by steel beams, the ends of which were assumed to extend 6 inches into the wall measured from the inner face, and which were assumed to be simply supported. With these assumptions, the net foundation load acting on the representative 15 feet strip of the continuous wall footing under a column at point A (see Figure 5.1), was found to be 373 kips, (Table 5.1). The point of application of the resultant of foundation loads was found to be approximately at the

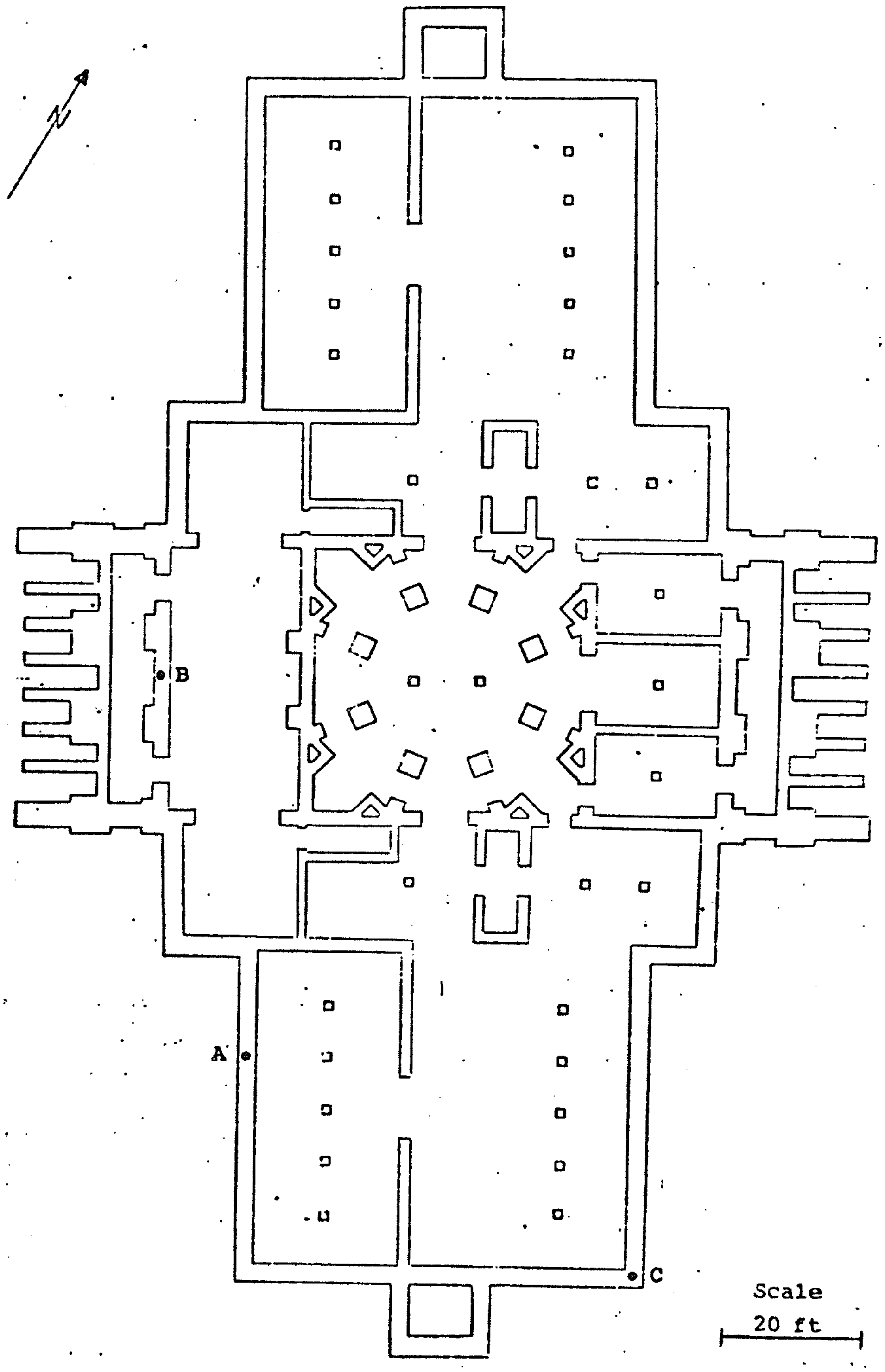


Figure 5.1 Administration Building: Foundation Plan

Table 5.1 Calculation of Typical Load for Administration Building

	Unit Weight	Tributary Area or Length	D. L. (Kips)	L. L. (Kips)
Roof: Dead Loads				
Clay tile	14.0 psf	180.0 sq.ft.	2.52	
1" sheathing	2.5 psf	180.0 sq.ft.	0.45	
2"x 6" Purlins @ 16" O.C.	1.7 psf	180.0 sq.ft.	0.31	
Longitudinal Truss	100.0 plf	15.0 ft.	1.50	
16" Brick wall	148.0 psf	45.0 sq.ft.	6.66	
8" Concrete slab	96.0 psf	195.0 sq.ft.	18.70	
Steel Beams Y	160.0 plf	11.0 ft.	1.76	
Steel Beam X	31.5 plf	7.5 ft.	0.24	
Roof: Live Load (snow load)	36.0 psf	165.0 sq.ft.		5.74
Concrete Slab:				
(1st, 2nd, 3rd Floor)	96.0 psf	339.0 sq.ft.	32.50	
Basement	10.3 psf	75.0 sq.ft.	0.77	
Steel Beams: (total)				
(1st, 2nd, 3rd floor)	-	-	6.00	
Basement	-	-	1.45	
Ceiling:				
(1st, 2nd 3rd floor, basement	8.0 psf	452.0 sq.ft.	3.61	
Flooring: (linoleum)				
(1st, 2nd, 3rd floor)	1.1 psf	339.0 sq.ft.	0.37	
Wall:				
(2nd, 3rd floor)	148.0 psf	420.0 sq.ft.	62.00	
(1st floor)	259.0 psf	210.0 sq.ft.	54.50	
Basement	296.0 psf	195.0 sq.ft.	57.60	
Sub-basement	296.0 psf	105.0 sq.ft.	31.00	
Floor Live Loads:				
(2nd, 3rd, floor)	50.0 psf	226.0 sq.ft.		11.30
(1st. floor)	60.0 psf	113.0 sq.ft.		6.78
Basement	100.0 psf	75.0 sq.ft.		7.50
Footing:	42.0 pcf	225.0 cu.ft.	10.7	
Soil:	108.0 pcf	450.0 cu.ft.	48.6	
		Total D. L.	341.24	
		L. L.		31.52

Note: Unit Weight of Materials
from: Timber Construction Manual, 1963.

centroid of the footing.

Considering the foundation as a 7 feet 8 inches by 15 feet strip footing, under the column at point A, the net soil pressure was found to be 3,230 pounds per square foot. This value is higher than the allowable value of the design pressure for stiff clay given by the 1965 Winnipeg Building Code, which is 3,000 pounds per square foot. From Equation (4.1.a) the ultimate bearing capacity was computed to be 4,520 pounds per square foot using a soil cohesion value of 800 pounds per square foot, for the medium stiff clay at 10 to 20 foot depth from nearby Test Holes 12, 13, and 32. Thus, the safety factor against soil shear failure was found to be 1.40 which is rather low.

From the results of the bearing capacity analysis, one should expect the Administration Building to settle heavily as the design pressure used is higher than 3,000 pounds per square foot allowed by the Code. However, the structure has survived without serious settlement problems. One probable reason for this may be due to its design as a rigid structure which tends to minimize the detrimental settlement by redistributing high load concentrations. Under-design in those days, however, was not unique to the Administration Building alone. Peck[30] reported that the average soil pressure used in designing the old Board of Trade Building in Chicago was 3.34 tons per square foot, and in 5 years after construction the structure had settled differentially more than 5 inches. The structure, however, continued to perform well for 45 years until it was demolished in 1928.

In order to learn about the settlement pattern of the Administration Building, settlement calculations were made for points A, B, and C (see Figure 5.1), selected to represent locations of the approximately maximum and minimum settlements.

The complicated footing plan of the Administration Building made necessary a simplified method of determining stresses under the footings. Peck and Uyanik[30] suggest a method where bearing wall and individual footings are replaced by circular footings of equivalent area, and the same centroidal position. Stress increases at various depths under the center of the circular footing may be found using the Bousinesq Theory. Tabulated influence number values, Table (5.2), as given by Newmark simplify the calculation. Peck and Uyanik claim that the error arising from this simplification is small and can be considered negligible, and for these reasons the method was adopted.

Once the net vertical stress increases under each footing at various depth had been determined, the settlement of each soil layer under each point considered was readily determined using Equation (4.6). For accurate computations, the true soil properties under each footing at each depth must be used. Unfortunately such data are not at present obtainable for every building site on the Campus. Therefore, for approximate settlement computations, the soil property from similar sites and soil conditions was employed. In this particular case test values reported by Baracos (see Table 3.7) were used assuming similarity in property of the Lake Agassiz clay in the Winnipeg Area as previously discussed in Chapter III. The settlement calculations are given

Table 5.2 Influence numbers for stress 'q' under the centre of a circular footing due to uniform vertical load intensity of 'q_o'

I	r/z
0.0000	0.0000
0.0200	0.1164
0.0400	0.1661
0.0600	0.2052
0.0800	0.2391
0.1000	0.2698
0.1200	0.2983
0.1400	0.3252
0.1600	0.3511
0.1800	0.3761
0.2000	0.4005
0.2200	0.4245
0.2400	0.4481
0.2600	0.4715
0.2800	0.4948
0.3000	0.5181
0.3200	0.5415
0.3400	0.5650
0.3600	0.5887
0.3800	0.6127
0.4000	0.6370
0.4200	0.6617
0.4400	0.6870
0.4600	0.7128
0.4800	0.7392
0.5000	0.7664
0.5200	0.7945
0.5400	0.8235
0.5600	0.8536
0.5800	0.8849
0.6000	0.9176
0.6200	0.9519
0.6400	0.9880
0.6600	1.0261
0.6800	1.0666
0.7000	1.1097
0.7200	1.1561
0.7400	1.2062
0.7600	1.2607
0.7800	1.3207
0.8000	1.3871
0.8200	1.4618
0.8400	1.5470
0.8600	1.6460
0.8800	1.7637
0.9000	1.9084
0.9200	2.0944
0.9400	2.3506
0.9600	2.7479
0.9800	3.5460

$$q = I\, q_o$$

where: I = influence number,
r = radius of circular footing,
z = depth under centre of footing where q is to be evaluated.

in Appendix B. The summary of results of settlement calculations for the Administration Building is shown in Table (5.3). In the settlement column, method 1 and method 2 represent settlement values computed by Equation (4.6) and Equation (4.7) respectively.

Table 5.3 Calculated Settlements for the Administration Building

Point	Pressure (psf) at depth indicated (ft)					Settlement (in)	
	12	16	20	24	28	Method 1	Method 2
A	2560	1590	690	160	-98	11.2	2.6
B	2610	2040	1202	626	250	14.2	3.2
C	2770	1407	570	246	54	11.2	2.5

5.3 Spread Footing Bearing Capacity and Settlement Analyses (Elizabeth Dafoe Library)

The main part of the Library is a two-storey building with basement, 118 feet by 184 feet in plan. The remainder of the structure is one-storey comprising of the main entrance, browsing area, and offices. The whole structure was founded on reinforced concrete spread footings, at elevation 14 feet below grade. The footing plan is shown in Figure (5.2). A square footing (L-5) in the browsing area was analyzed to establish the value of the design soil pressure. This is a square footing of 7 by 7 feet, designed to support the interior loads in the browsing area. This part of the Library under analysis consists of only a first floor and basement. The loads on the basement floor are transmitted directly to the ground and for this reason do not appear in the calculation for foundation loads.

The tributary area was found to be 324 square feet, from which a careful estimate was made of the net load supported by this particular footing, the magnitude of which was 118.7 kips, (see Table 5.4). The design soil pressure calculated from the above mentioned value of net load and the footing area of 49 square feet was 2420 pounds per square foot.

For saturated clay under undrained condition, the net ultimate bearing capacity is given by:

$$q_{ult.net} = cN_c s_c d_c i_c \qquad (4.1.a)$$

where the various symbols are as previously defined.

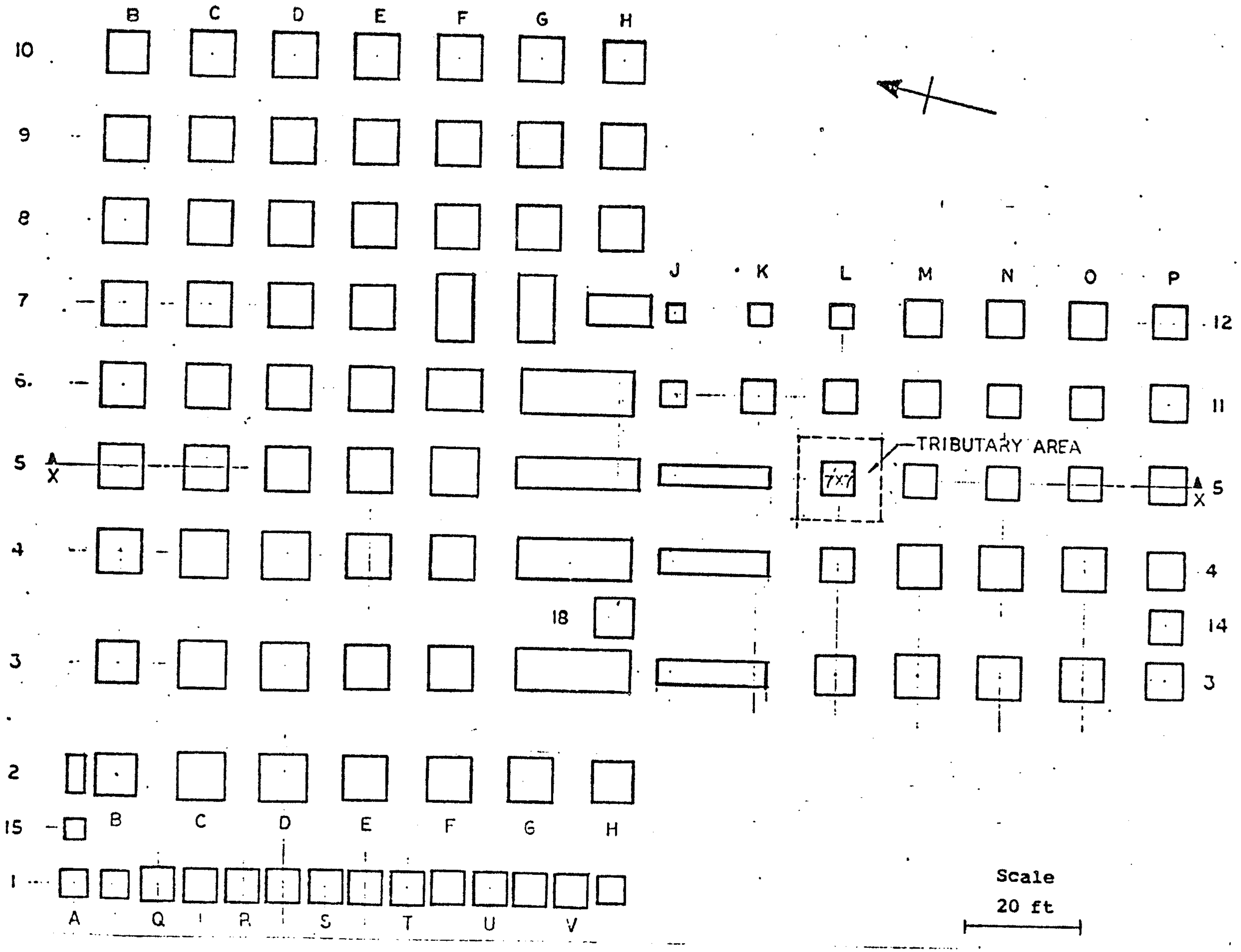


Figure 5.2 Elizabeth Dafoe Library: Foundation Plan

Table 5.4 Calculation of Typical Foundation Load for Elizabeth Dafoe Library

	Unit Weight	Tributary Area or Length	D. L. (Kips)	L. L. (Kips)
Concrete Slab:				
Roof	63.6 psf	324.0 sq.ft.	22.20	
Main Floor	106.0 psf	324.0 sq.ft.	34.40	
Column:				
Main Floor	150.0 pcf	31.0 cu.ft.	4.70	
Basement	150.0 pcf	20.3 cu.ft.	3.10	
Footing:	42.0 pcf	93.7 cu.ft.	3.96	
Roofing:				
¼" asphalt	2.5 psf	324.0 sq.ft.	0.80	
Roof Insulation:				
2" cork board	1.0 psf	324.0 sq.ft.	0.32	
Flooring: -				
(3/16" asphalt floor tile)				
Main Floor	2.0 psf	324.0 sq.ft.	0.65	
Partition:				
¼" plywood	0.75 psf	240.0 sq.ft.	0.18	
4" hollow clay tile	18.0 psf	240.0 sq.ft.	4.32	
Live Loads:				
Snow load	36.0 psf	324.0 sq.ft.		11.66
Main Floor	100.0 psf	324.0 sq.ft.		32.40
		Total D. L.	74.63	
		L. L.		44.06

Note: Unit Weight of Materials

from: Timber Construction Manual, 1963.

The ultimate net bearing capacity computed by Equation (4.1.a), incorporating the shape and depth factors, was found to be 7,500 pounds per square foot. The cohesion of brown clay at 14 foot depth was taken to be 750 pounds per square foot, at one-half of the unconfined compressive strength of the clay shown in the nearby Test Hole 23. Details of calculations are compiled in Appendix C.

For the purpose of settlement estimate, footings L-5, P-12, and E-5 were analyzed. These were selected to represent the points of approximately minimum, average, and maximum settlements respectively. Soil pressure under each of these footings was assumed to be the same as the soil pressure previously found under footing L-5, which is 2420 pounds per square foot. The stress increases due to foundation load were then computed, assuming the seat of settlement to extend below the base of the footing equal to the "significant depth", which is the depth within which the load on the footing alters the state of stress in the soil enough to produce a perceptible contribution to the settlement. The compressible clay deposit was subdivided into three layers of 4 feet thick.

Again, for approximate settlement computations, soil properties given in Table (3.7) were employed. Then the vertical stress increase at mid-depth of each of the soil layers under the footing was computed using Newmark's influence chart.

To establish the maximum and the minimum values of settlements, the consolidation settlement was estimated both on the basis of Equation (4.6) and Equation (4.7). The computed stresses and settlements are given in Table (5.5). In the settlement column, method 1 and method 2 represent settlement values computed by Equation (4.6) and Equation (4.7) respectively.

Table 5.5

Calculated Settlements for the Elizabeth Dafoe Library

Point	Pressure (psf) at depth indicated (ft)			Settlement (in)	
	16	20	24	Method 1	Method 2
L-5	1764	443	122	5.5	1.5
P-12	2102	656	332	7.0	1.9
E-5	1944	1036	583	7.7	2.1

Note: Depth in feet below surface grade

CHAPTER VI

THEORY AND DESIGN OF DEEP FOUNDATIONS

6.1 Introduction

Caissons and piles form the two main categories under the general heading of deep foundations. A caisson is a slender cylindrical body of masonary that transfers a load through a poor stratum onto a better one. A pile is essentially a very slender caisson that transfers a load either through its lower end onto a firm stratum or else through side friction onto the surrounding soil. The first case is generally referred to as end-bearing piles and the second case friction piles. The pile system may, however, utilize both end-bearing and friction components to carry the imposed load. As both caissons and piles have been utilized at the University of Manitoba, both foundation types are considered.

Since piles and caissons serve the same purposes, to transfer the weight of a structure onto a firm stratum covered by soft and compressible soil, no sharp distinction can be made between the two. Cast-in-place piles installed in drill holes might preferably be considered caissons of small diameter. On the other hand drilled in caissons made by driving a heavy steel pipe with a cutting shoe down to bedrock might be called end-bearing piles. The principal difference between caissons and piles lies in the method of installation and size. Caissons are usually large enough to permit a man to work inside during construction.

The relative merits of caissons in comparison with piles depend not only on economic but also on several technical factors. These include the influence of the method of construction on the load that can be assigned to the foundation, and the influence of the soil and water conditions on the ease or difficulty of construction and on the integrity of the completed foundation.

6.2 The Modified Engineering News Formula

Despite the misleading results dynamic pile formulas can give, especially when piles are driven in cohesive soils; they still are considered of qualified value for estimating the capacity of piles driven in cohesionless soils. Although these pile formulas are numerous, their general approaches are the same. They equate the work performed by the driving hammer to the work required to increase the penetration of the pile against the resistance of the soil plus energy lost. If energy losses can be accounted for, there is at least a theoretical possibility of estimating the dynamic resistance, R_d, from the average penetration, S, of the pile under the last few blows of the hammer, provided the weight, W_r, of the ram and the height of fall, H, are known.

The modified Engineering News formula derived from the same principle discussed above except that a constant, C, is introduced. The quantity C is regarded as an additional penetration of the point of the pile that would have occurred if there were no energy losses, and is approximately equal to 1.0 inch for piles driven by a drop hammer and 0.1 inch for pile driven by a steam hammer. Due to the realization of many theoretical shortcomings of this approach, the reduction factor designed to account for the over estimated power rating is also incorporated. The modified Engineering News formula, incorporating a safety factor of 6.0, takes the form:

$$R_d = \frac{2\ En}{S + 0.1} \cdot \frac{W_r + e^2 W_p}{W_r + W_p} \qquad (6.1)$$

where: R_d = computed design capacity (lb),

En = manufacturer's maximum rated energy (ft-lb),

S = set or final penetration per blow (inch),

e = coefficient of restitution,

W_r = weight of hammer ram (lb),

W_p = weight of pile (lb).

Note: The coefficient of restitution value ranges from 0.6 for steel on steel to 0.25 for hammer striking on the head of a wood pile or a wood cushion block.

6.3 Estimating Pile Capacity by Static Methods

Formulas for determing the load capacity of piles using static methods may be expressed by the following basic equation:

$$Q_{ult} = Q_p + Q_s \qquad (6.2)$$

where: Q_{ult} = ultimate pile capacity,

Q_p = load carried in point bearing,

Q_s = load carried by friction along, perimeter of pile.

The load Q_s carried along the perimeter of the pile can be obtained from:

$$Q_s = (\sigma_{h_{av}} \tan \phi_a + c_a) A_s \qquad (6.3)$$

and:

$$\sigma_h = K_o \gamma z \qquad (6.4)$$

where: ϕ_a = angle of friction between pile and soil,

c_a = average soil adhesion

A_s = circumferential area of pile shaft

$\sigma_{h_{av}}$ = average lateral pressure along the pile shaft,

K_o = coefficient of lateral to vertical earth pressure,

z = depth,

γ = effective unit weight of soil.

For the special case of saturated cohesionless soils in undrained loading where the angle of internal friction can be considered to be zero, Equation (6.3) simplifies to;

$$Q_s = c_a A_s \qquad (6.3.a)$$

Equation (6.3.a) applies to support obtained from the brown and grey clays on the Campus. Several considerations govern the choice of the value of c_a to use. Broms[7] suggests the following relationship:

$$c_a = \alpha c \qquad (6.5)$$

where: α = an empirical coefficient,

c = cohesion of soil, and equals ½ of the unconfined compression of the soil.

Meyerhof and Murdock[26] found that the adhesion increases with increasing shear strength, and is about 80% of the shear strength adjacent to the concrete, and may be weakened by the effect of softening due to increase in water content in the clay adjacent to the pile. Their findings for London Clay are shown in Table (6.1). Meyerhof and Murdock also state that the lowest possible strength of the clay is that measured after it has been allowed to soften fully under zero load. With short bored piles where the clay likely to be heavily fissured, the adhesion may be taken as 0.3 c, and should be neglected in the zone of seasonal shrinkage. According to the 1965 Winnipeg Building Code, the

Table 6.1 Effect of Softening on Adhesion
(After Meyerhof & Murdock)

Water Content	Increase in Water Content	Shear Strength (psf)	Relative Strength	Adhesion Original Strength
28	0	3,300	1.00	0.80
28.5	0.5	2,900	0.88	0.70
29	1	2,550	0.77	0.62
30	2	2,000	0.61	0.48
31	3	1,640	0.50	0.40
32	4	1,320	0.40	0.32
33	5	1,050	0.32	0.26
34	6	850	0.26	0.21

frictional support of cohesive soils must be neglected for a depth of at least 5 feet below basement, or 10 feet below adjacent ground level whichever is greater. According to Skepmton[34] with deeper piles for heavy foundations the shaft adhesion may attain values in the region of 0.6 c under favourable condition. But an adhesion of this magnitude should be adopted only after checking by pile loading tests and, in any case, it is unwise to use values exceeding 2,000 pounds per square foot. Values of pile adhesion or skin friction in lieu of other data, based on studies on pile loading tests by Tomlinson,[38] are shown in Table (6.2).

However, investigations by Yaipukdee,[41] on friction value for cast-in-place concrete piles in Winnipeg clay using direct shear tests, indicate that the friction value between the clay and the pile is increased due to the effect of cementing action of the concrete, consequently the strength of the soil surrounding the pile is also increased. The strength, however, decreased with increasing distance from the pile. A a distance of about 0.25 inches

Table 6.2 Ultimate Values of Skin Friction on Piles Embedded in Cohesive Soils (After Tomlinson)[38]

Material of Pile	Unconfined Compressive Strength of Clay (tons/ft^2)	Ultimate Skin Friction between Pile and Clay (lbs/ft^2)
Concrete and Timber	0 - 0.75	0 - 700
	0.75 - 1.5	700 - 1000
	1.5 - 3.0	1000 - 1300
	over 3.0	1300
Steel	0 - 0.75	0.- 700
	0.75 - 1.5	700 - 1000
	1.5 - 3.0	1000 - 1200
	over 3.0	1200

from the pile, its value is approximately equal to the natural shearing strength of the clay for the same moisture content. The values of skin friction between the medium stiff clays and concrete piles in relation to the moisture content and the distance of shear plane from the pile are given in Table (6.3). The value of α to use for Campus soils requires engineering judgment. A value of 0.8 was considered reasonable.

For the special case of cohesionless soils, Equation (6.3) simplifies to:

$$Q_s = \sigma_{h_{av}} (\tan \phi_a) A_s \qquad (6.3.b)$$

The value of K_o to find $\sigma_{h_{av}}$ ranges from 0.5 to 4.0 depending on the kind of pile, steel, concrete, or wood; and the relative density of the soil. For driven piles the value of K_o depends on the volume per unit length of pile, and for small displacements the value approaches the lateral earth pressure in the at-rest case. The basic problem for estimating the load carried by friction is to arrive at the correct lateral pressure coefficient for cohesionless soils, or a resonable skin friction value for cohesive soils. These

Table 6.3 Values of c_a, c, δ, and ϕ for Clay-mortar Interface (after Yaipukdee)

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

Note: T is the distance between pile and shear plane.

values, however, can be derived at only through a considerable amount of engineering judgment as well as access to sufficient soil property data.

Because of the limited occurrence of cohesionless soils on the Campus, to the relatively thin thickness of glacial till that would give frictional support, Equation (6.3.b) is not required.

With regard to the end-bearing support, Q_p of the piles, these may be found from modified forms of the general bearing capacity equations. Broms[7] gives:

$$Q_p = \left[K_c c N_c + K_\gamma \gamma D N_\gamma + K_q\, D_f (N_q - 1)\right] A_p \qquad (6.6)$$

where: K_c = shape factor for cohesion,

K_γ = shape factor for depth,

K_q = shape factor surcharge,

D = diameter,

and the other terms as previously defined.

For circular piles K_c, K_γ, and K_q may be taken as 1.3, 0.6, and 1.0 respectively.

For the case of the saturated clay in undrained loading, Equation (6.6) simplifies to:

$$Q_p = K_c \; c \; N_c \, . \qquad (6.6.a)$$

Skempton[33] suggest for this case a further simplification:

$$Q_p = 9 \, c \, . \qquad (6.6.b)$$

For the case of cohesionless soils, Equation (6.6) reduces to:

$$Q_p = \left[K_\gamma \gamma D \, N_\gamma + K \gamma D_f \; (N_q - 1) \right] \; A_p \, , \qquad (6.6.c)$$

which for round pile section:

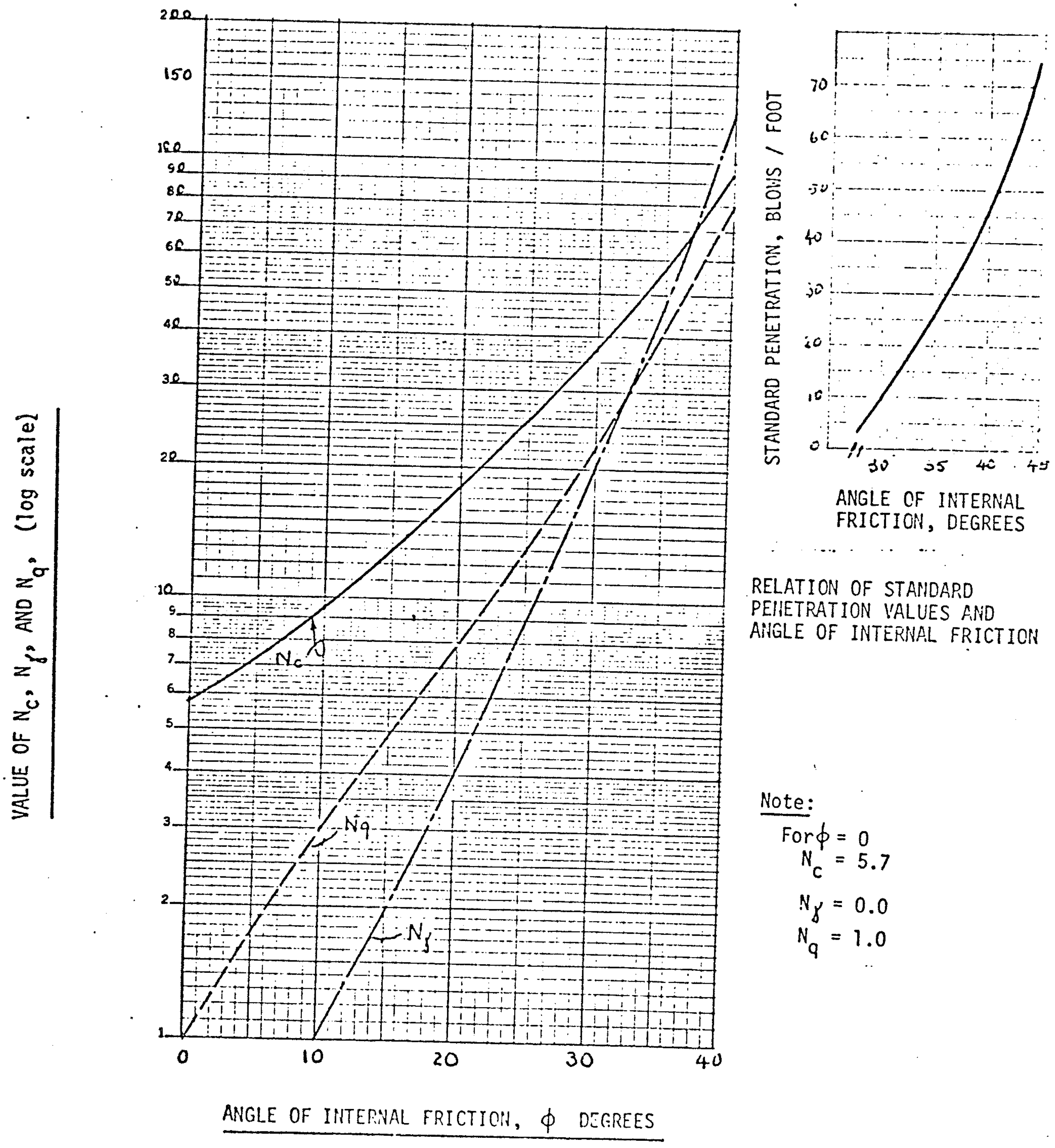
$$Q_p = \left[0.6 \gamma \frac{D}{2} N_\gamma + \gamma D_f \; (N_q - 1) \right] \; A_p \qquad (6.6.d)$$

and for square pile section:

$$Q_p = \left[0.4 \gamma B \, N_\gamma + \gamma D_f \; (N_q - 1) \right] \; A_p \qquad (6.6.e)$$

The value 0.4 for K_γ is suggested by Broms[7] and Terzaghi.[36]

The values of the bearing capacity factors N_c, N_q, and N_γ, for different angles of internal friction ϕ after Terzaghi[36] are given in Figure (6.1). For cohesionless soils where only the N values of the standard penetration



RELATION OF STANDARD PENETRATION VALUES AND ANGLE OF INTERNAL FRICTION

Note:

For $\phi = 0$

$N_c = 5.7$

$N_\gamma = 0.0$

$N_q = 1.0$

Figure 6.1 Relation between Angle of Internal Friction and the Terzaghi Bearing Capacity Factors, Based on Data by Vogel[28] test may be known, the angle of internal friction can be obtained from the curves in Figure (6.2) which shows the approximate relationship between N an Ø as advocated by Peck, Hanson, and Thornburn.[29]

Investigations by Meyerhof[25] have indicated that the bearing capacity of footings, the point reistance, and skin friction of piles in cohesionless soils can frequently be determined most conveniently from the results of standard penetration tests made on the site. These empirical relationships between the ultimate values of point resistance, skin friction, and the N values of penetration tests are given below:

$$Q_p = [2.5 \text{ to } 4.0\ N]\ A_p \quad \text{ton per sq.ft.} \tag{6.7}$$

where: N = number of blows, standard penetration at and below the pile tip.

$$Q_s = [0.02\ N]\ A_s \quad \text{ton per sq.ft.} \tag{6.8}$$

where: N = average number of blows, standard penetration along length of pile.

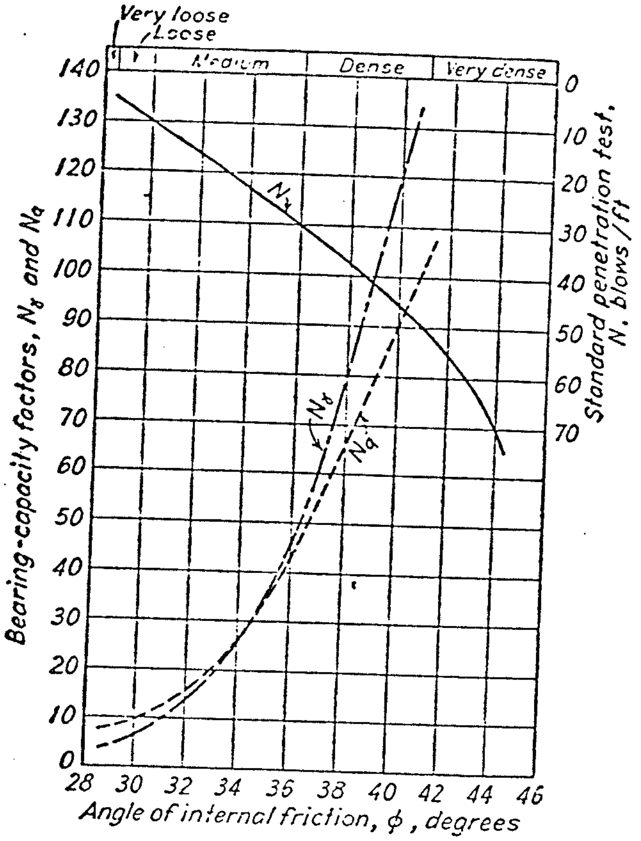


Figure 6.2 Relationship Between Ø and N-Value

(After Peck, Hanson, Thornburn)

6.4 Net Bearing Capacity of Pile Groups

Terzaghi and Peck[37] state that a group of friction piles may fail as units by breaking into the ground before the load per pile becomes equal to "safe design load". Hence, the computation of the safe design load must be supplemented by a computation of the ultimate bearing capacity of the entire group. If the pile and the soil between the piles, sink as a unit, the ultimate bearing capacity of the rectangular pile group is given with sufficient accuracy by:

$$Q_g = q_d\, BL + L_p\, (2B + 2L)\, s \tag{6.9}$$

where: Q_g = ultimate capacity of pile group,

q_d = ultimate bearing capacity per unit area,

BL = base area of pile group,

L_p = effective length of piles, excluding depth of possible soil shrinkage,

s = average shearing resistance of soil,

$= \underset{av}{\sigma_h} \tan \phi + c$

According to Chellis,[23] computations based on Equation (6.9) has shown that a base failure can hardly occur unless the pile group consist of a large number of friction piles embedded in silt or soft clay. In this case the overlapping zones of pressure around each pile are developed hence the reduction in the bearing capacity value of the pile group is required. The reduction in value per pile depends on the size and shape of the pile group and the size, spacing, and length of the piles.

6.5 Settlement of Pile Groups

Under certain conditions, friction piles can settle excessively and an estimate of settlement should be made.

The settlement of a pile group is larger than the settlement of a single pile. Investigations have indicated that the larger the group, the deeper the stress penetrates the strata. For routine settlement calculations of pile groups the method presented by Peck, Hanson, and Thornburn[29] may be employed. Figure (6.3) shows how the stress distribution below the pile is obtained.

The stresses in the soil underlying a group of piles are not readily evaluated for several reasons such as; the unknown distribution of friction effects along the pile, the overlap of stress from adjacent piles, and the influence of pile driving. Therefore, it has been the usual practice to simplify the stress computations and to assume that the pile cap is sufficiently rigid so that settlement is uniform.

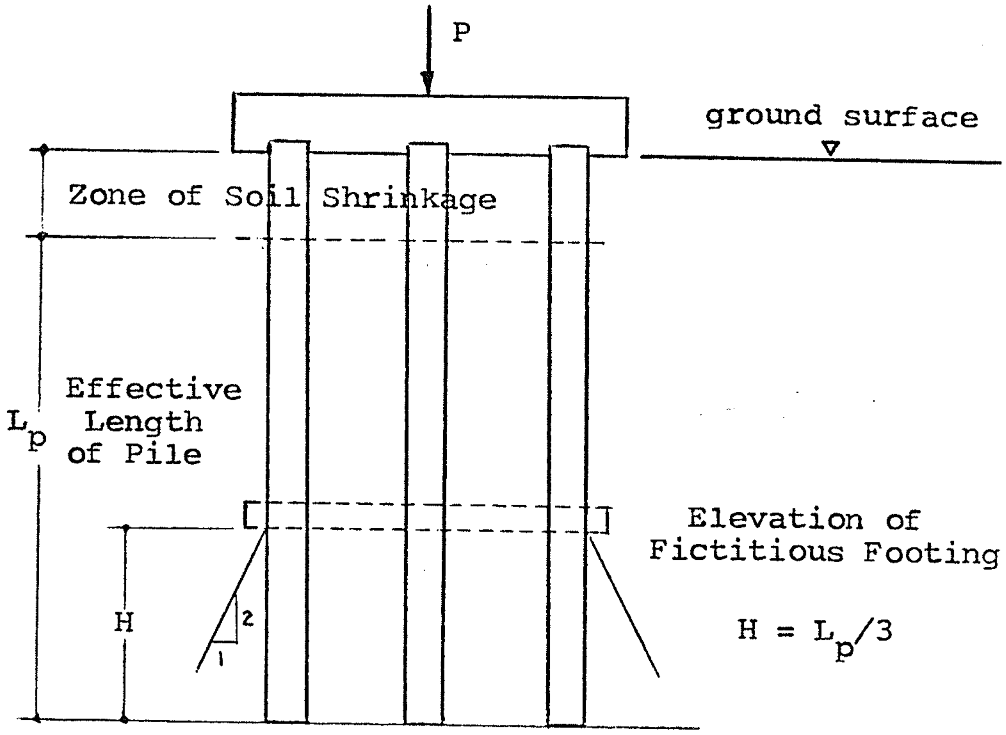


Figure 6.3 Simplified Computation of Soil Stresses Beneath a Pile Group

For friction piles, the load is placed on a fictitious rigid footing located a $L_p/3$ from the bottom of the piles, with L_p as in Figure (6.3). The spread-out of load is taken as either 2:1 or 30 degree measured from the vertical. The analysis in Appendix D uses the spread-out of 30 degree for the ease of numerical calculations. Except for the simplified load spread assumption, the settlement computations for the fictitious footing are made in the same way as for the spread footings discussed in Chapter IV. The soil is divided into convenient strata below the footing, and the compression of each stratum computed using Equations (4.6) or (4.7). The settlement of the pile cluster is taken as the same of the compressing of the stratum below the fictitious footing.

CHAPTER VII

ANALYSES OF DEEP FOUNDATION TYPES

This chapter deals with analyses of the foundation types which have been used to support the newer and larger buildings built recently on the Campus, and which include friction piles, end-bearing piles, and caissons. The foundation of the Education Building Addition 1968 was selected to represent the cast-in-place friction piles and caissons, whereas the foundation of the University Centre was selected to represent the precast, driven, end-bearing piles.

7.1 Analyses of Selected Friction Piles

The foundations of the Education Building Addition, consist of both caissons, supported on limestone bedrock, and cast-in-place concrete friction piles in stiff to moderately stiff clay. In this section Pile Foundations 1, and 2, (see Figure 7.1), consisting of cast-in-place friction piles will be analysed.

Column Load B-3 is supported on a cluster of three 16 inch diameter by 30 feet cast-in-place, concrete friction

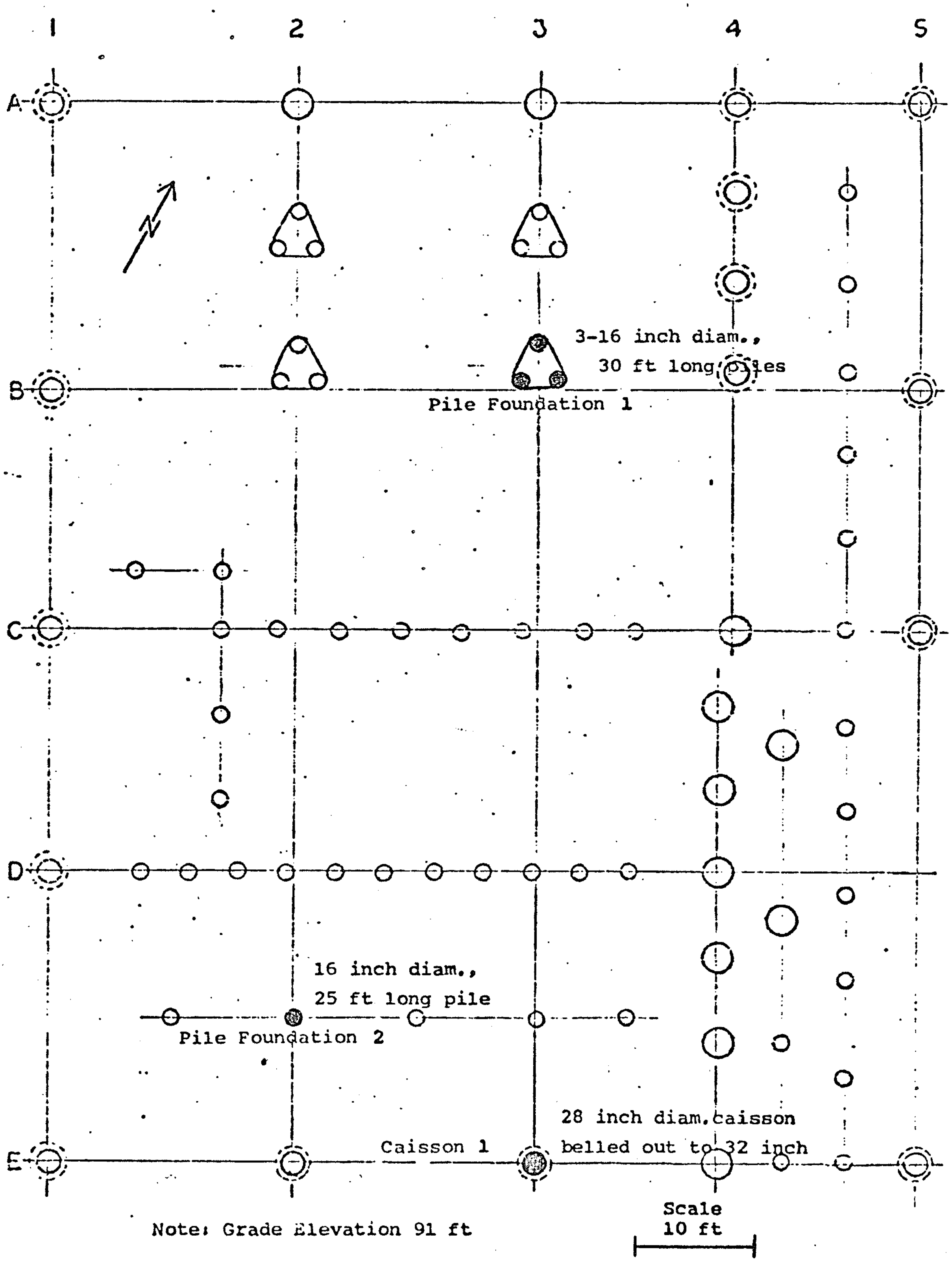


Figure 7.1 Education Building Addition: Foundation Plan

piles. The analysis was made considering first individual action of the piles, and then the group action. For settlement calculations only settlement of the pile group was computed because the group settlement is always larger than the settlement of a single pile.

The ultimate bearing capacity per pile was calculated as shown in Appendix D using Equations (6.2), (6.3.a), and (6.6.b)with appropriate substitution of coefficients.

The accuracy of computation of the bearing capacity of friction piles in clay depends on the validity of the c values used. In this case a cohesion of 1110 pounds per square foot was used, taken to be one-half of the average unconfined compressive strength of the brown clay given in Test Hole 15, the one previously done and located closest to the Education Building. A safe skin friction value of 300 pounds per square foot was obtained. This same skin friction value is also given by the Winnipeg Building Code for the maximum design capacity of a concrete friction pile, (Code - Section 4.2.2.10).

The ultimate bearing capacity of the individual friction pile in clay can easily be calculated using Equation (6.2). The skin friction resistance calculated from Equation (6.3.a) is 88.8 kips. The end-bearing capacity calculated from Equation (6.6.b) is 14 kips. Hence the resulting total ultimate load is 102.8 kips per pile. With a total of three piles in the group, the ultimate bearing capacity of the group, taking pile effective length to be 24 feet, is 308 kips. The actual column load carried by the footing is 40 kips, therefore, a safety factor of 7.7 was attained.

The ultimate bearing capacity of the piles for group action was also computed, using the perimeter and end-bearing areas of the pile group. The ultimate bearing capacity of the pile group was found to be 377 kips. With the actual column load of 40 kips, the factor of safety is approximately 9.4. Therefore, for Pile Foundation 1, the piles acting individually govern and the safety factor is the lower 7.7 value.

Using the method outlined in Section (6.5), the settlement of Pile Foundation 1 was calculated. Some question arises as to what unit weight of the soil to use in Equation (4.6) applied to the settlement calculation. The submerged unit weight is normally used when the soil is found below the water table. However adjacent Test Hole 15 did not establish any definite water table. There is also a distinct possibility as suggested by the preconsolidation pressures that the water table may have been at considerable depth during past times. For these reasons p_o was taken as the existing overburden pressure based on the total unit weight of the soil.

Referring to Figure (7.2) for settlement computations, the soil under the "fictitious footing" was subdivided into two layers of 11 feet each. The stress increases due to the column load at the mid points of these strata, point I and II, were then computed and were found to be 0.32 and 0.07 tons per square foot respectively. The total consolidation settlement at each point was then computed using Equation (4.7) with the change in void ratio

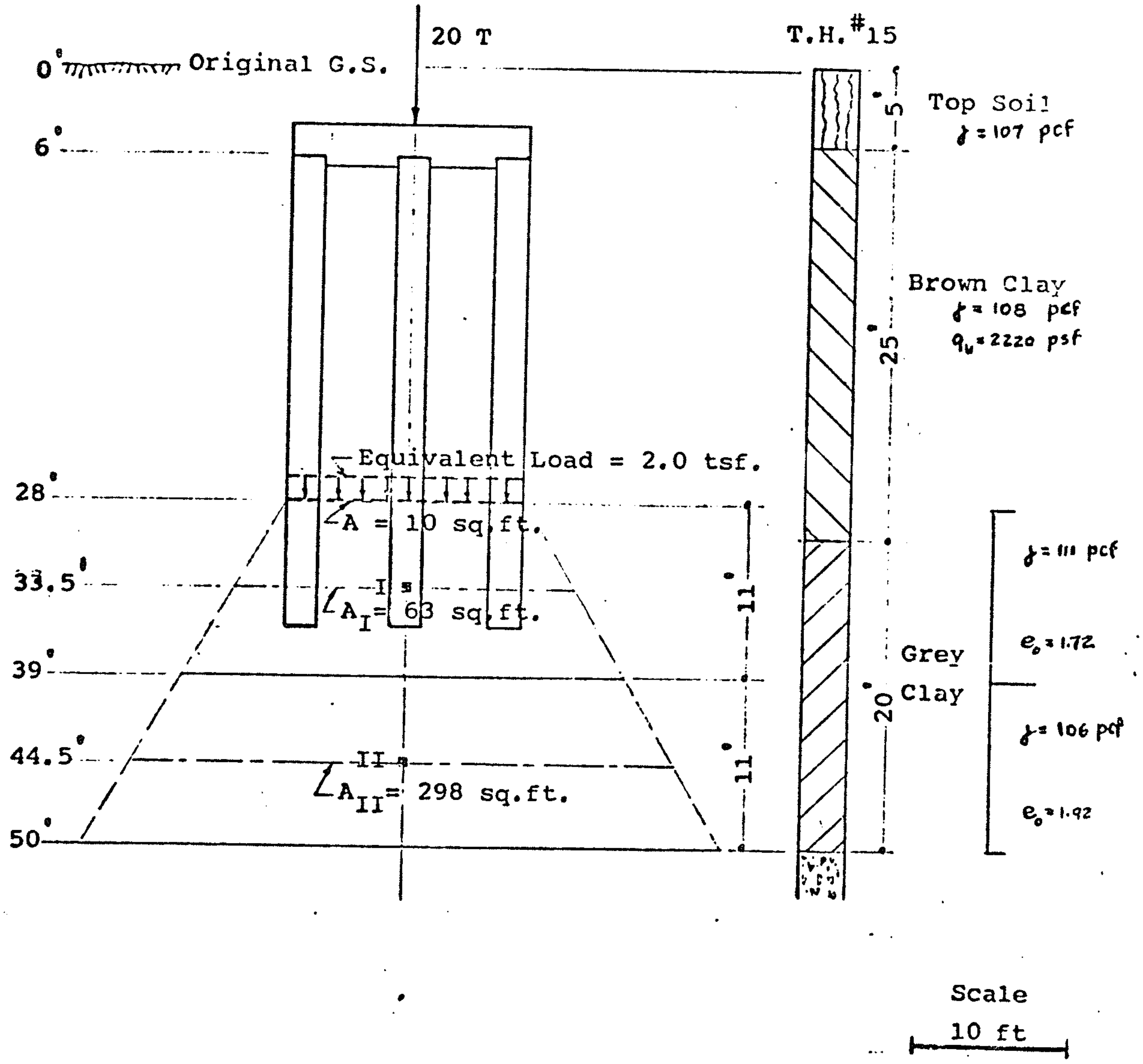


Figure 7.2 Soil Sub-division Under Pile Foundation 1

between initial and final pressures read off directly from the appropriate consolidation curves in Figure (3.5). The settlement values of points I and II were then added up to give the resulting total consolidation settlement of 1.42 inches.

Pile Foundation 2, in Figure (7.1) is supported by a cast-in-place concrete friction pile of 16 inches in diameter by 25 feet long. Assuming the zone of soil shrinkage to extend 6 feet below the basement level, the effective pile length is 19 feet. The pile carries a total column load of 14 kips, and supports a 5 inches thick, concrete basement slab.

The soil profile from Test Hole 15, (Appendix A) was again assumed for this footing. The soil subdivisions and properties are shown in Figure (7.3). The ultimate bearing capacity of the pile was computed to be 85 kips. With the actual column load of 14 kips, the safety factor against a bearing capacity failure is 6.1. The total settlement based on the consolidation of the 21 feet thickness of clay between the fictitious footing and the top of the till, was found to be 0.92 inches.

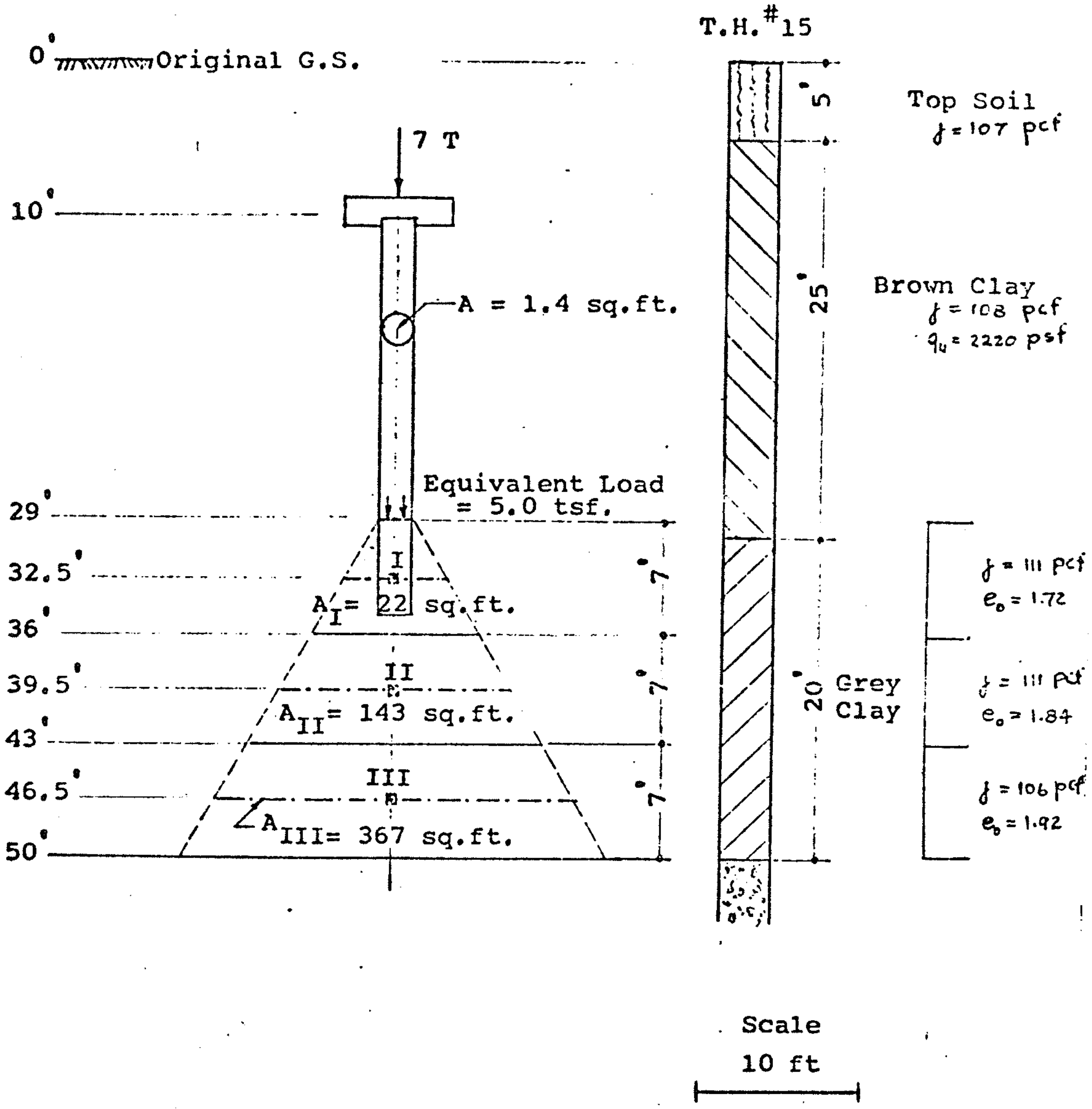


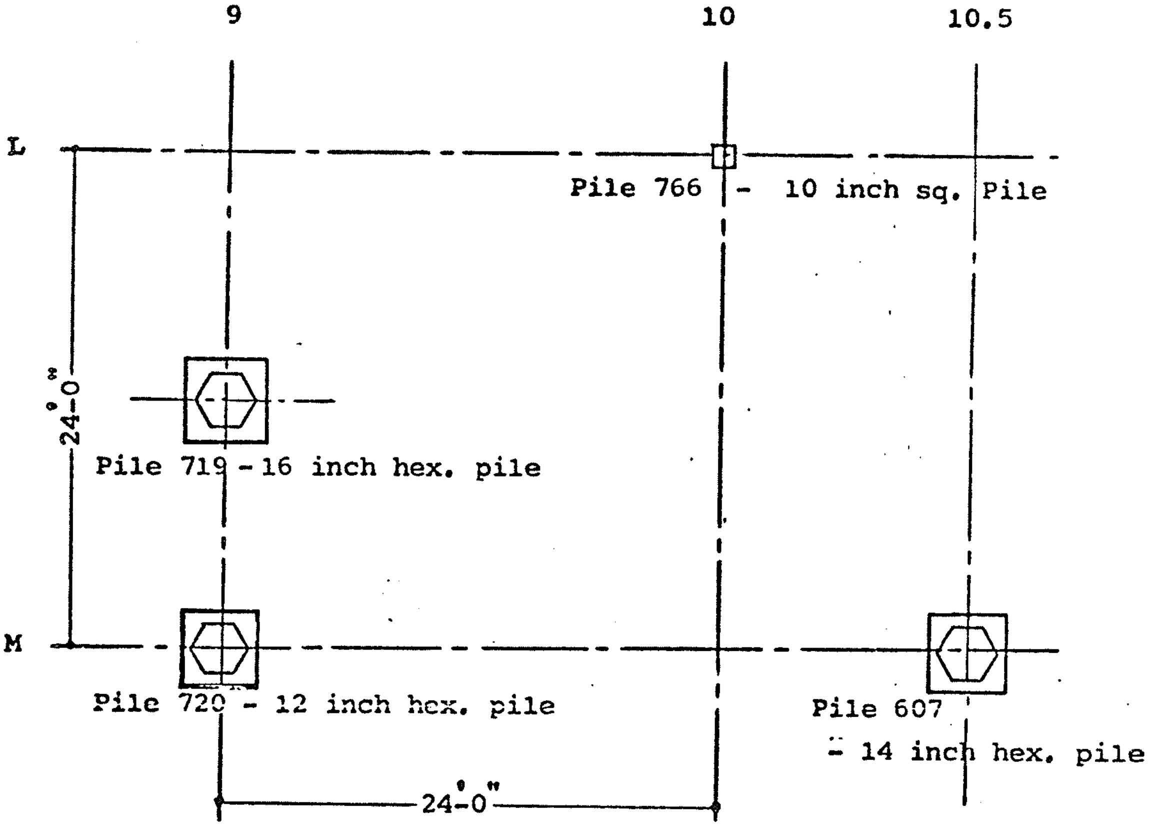
Figure 7.3 Soil Sub-division Under Pile Foundation 2

Detailed numerical analyses for Pile Foundations 1 and 2 are given in Appendix D.

7.2 Analyses of Selected End-Bearing Piles

The foundations of the University Centre which rested on end-bearing concrete piles driven to refusal in hardpan were examined. To analyse the capacity of these piles, four typical piles in the south-east section of the building were selected. The location and other data pertaining to these footings are shown in Figure (7.4). The foundation plan of the University Centre is on a 24 feet by 24 feet grid with intermediate piles or footings for light column loads on a 12 feet by 12 feet grid. The actual column loads carried by these piles, range from the maximum load per pile of 160 kips supported by Pile 719, to the minimum of 30 kips on Pile 766.

For each pile size, the load-carrying capacity was analysed using the dynamic pile formula to compute the safe design load, and the bearing-capacity theory to compute the ultimate capacity. For the dynamic method the modified Engineering News formula was used, and the design load per pile



NOTE: Surface grade elevation = 108' - 0" (Architectural)
= 765' (Geodetic)

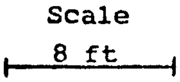


Figure 7.4 University Centre: Locations of Piles
Nos. 607, 719, 720, 766

was computed by:

$$R_d = \frac{2\ En}{S + 0.1} \cdot \frac{W_r + e^2 W_p}{W_r + W_p} \qquad (6.1)$$

To compute the safe design load, R_d, the various terms used in Equation (6.1) above is obtained from the contractor's pile driving data (Table 7.1), and the weight of pile, W_p, can be easily calculated from the pile cross-sectional area and length assuming the bulk weight of concrete to be 150 pounds per cubic foot. The coefficient of restitution, e, for hammer striking on a wood cushion block, was taken as 0.25. It should be noted, however, that the safe design load computed by Equation (6.1) has already incorporated a safety factor of 6.0 according to the theory.

Alternatively, the safe design loads for end-bearing concrete piles may be obtained from the values given in the 1965 Winnipeg Building Code, Section (4.2.2.11). Under ideal conditions, where piles are driven to refusal in hardpan with hammers having sufficient energy to produce the desired results, the maximum design loadings for 12, 14 and 16 inch hexagonal, precast, prestressed, concrete piles may be taken as 100, 140, and 180 kips respectively.

Table 7.1 Pile Driving Data for Piles 607, 719, 720, and 766 (University Centre)

Footing	Pile No.	Size of Pile			Elev. of Tip of Pile	Length Cut-off Pile	Net Length of Pile	Tip Elev.	Cut-off Evlev.	Penetr. last Blows*
		Length	Butt	Tip						
M- 10.5	607	45'	14"	14"	751.7	7.3	42.3	706.7	749.0	$8 / \frac{7''}{8}$
L.5-9	719	45'	16"	16"	753.7	3.1	41.7	708.7	750.6	$12 / \frac{7''}{8}$
M-9	720	45'	12"	12"	753.5	4.9	40.1	708.5	748.6	$5 / \frac{7''}{8}$
L-10	766	50'	10" Sq	10" Sq	759.0	16.1	43.9	709.0	753.0	$3 / \frac{7''}{8}$

* Blows of hammer/pile penetration inches

As it is generally believed that pile driving formulas do not usually yield satisfactory results, the ultimate carrying capacity of the piles was, therefore, checked using the static method of analysis. In this method the soil ultimate net bearing capacity was computed using Equations (6.6.d)and(6.6.e)for hexagonal and square piles respectively. Equation (6.6.d) however, applies only in the case of circular piles, but for approximate procedure, however, it was used for computing the ultimate capacity of hexagonal piles.

Since the pile was driven to refusal in the glacial till, a knowledge of the properties of the till is essential to the accurate assessment of the ultimate bearing capacity. But, unfortunately, such data are scarce on the Campus at present. On examining the log of test-holes made on the site by the contractor, the till was described as light grey putty of medium dense consistancy. Heavy seepage was also encountered in many of the test holes when the boring had reached the till stratum. The typical composition of the till is a mixture of clay, silt, sand, gravel and occasional boulders. This infers a soil having both "cohesion" and "angle of internal friction". However, adequate cohesion "c" values were not available for the till, and very conservatively the value was neglected. Because of the great percentage of granular soils, and the fact that standard penetration test values of the dense till are very high, with a full one foot of penetration often being impossible. The angle of internal friction of the till was therefore assumed to be 44° (corresponding to N = 70) from Figure (6.2). The unit weight of till was taken to be 143 pounds per cubic foot and the average unit weight of the clays overlying the till stratum was calculated to be 118 pounds per cubic foot, based on one test from Test Hole 9.

The bearing capacity factors N_c, N_γ, and N_q for $\phi = 44^{\circ}$ were taken from Table I in Vogel's Thesis [28] to be 151, 252, and 147 respectively.

From the above values the net ultimate bearing capacity was computed. The value was then multiplied by the cross-sectional area of the pile point to get the ultimate carrying capacity per pile. A safety factor of 2.5 was then applied to get the safe design loading for each pile size. A summary of the computed safe design loadings for Piles 719, 607, 720 and 766 are shown on Table (7.2).

7.3 Analyses of Selected Drilled Caissons

Caisson 1 of the Education Building 1968 addition (Figure 7.1) was selected to represent caisson or pier foundations. This caisson has a shaft of 28 inches in diameter and is belled out to 32 inches in diameter at the base, resting on sound limestone bedrock. The caisson carries a column load of 313 kips. The detailed numerical calculations for this caisson are shown in Appendix F.

Caissons commonly serve to transfer the weight of a structure onto a firm stratum or sound bedrock covered by soft and compressible soil. Practically the entire load on a caisson is ultimately carried only by its base. Hence, the allowable load on caissons surrounded by relative compressible soil should not include any allowance for skin friction. Once the bearing capacity per unit of area of

Table 7.2 Summary of Computed Safe Design Loads and Actual Loads for Selected End-Bearing Piles, (University Centre)

Pile No.	Pile Size	Perimeter (ft)	Cross-Sectional Area (Sq.ft)	Net Pile Length (ft)	Set in/blow	Actual Column Load (Kips)	Theoretical Safe Design Loading				Allowable Max.Design Load*
							From Pile Driving Formula		From Bearing Capacity Theory		
							Load	S. F.	Load	S. F.	
#719	16" HEX	4.0	1.13	41.9	0.07	160	153	6.0	157	2.5	180
#607	14" HEX	3.5	0.88	42.3	0.11	109	94	6.0	123	2.5	140
#720	12" HEX	3.0	0.64	40.1	0.18	35	81	6.0	85	2.5	100
#766	10" x 10"	3.3	0.70	43.9	0.21	30	67	6.0	102	2.5	75

* From Winnipeg Building Code

the stratum supporting the base is known, the ultimate or allowable carrying capacity of a caisson may be expressed by Equation (6.6.d).

The safe bearing capacity of sound limestone bedrock in the Winnipeg Area is given as 60 kips per square foot in the 1965 Winnipeg Building Code, Section 4.2.2.1.(2).

Using the above bearing value for sound limestone bedrock with a bearing area at the base of the caisson of 5.6 square feet, the safe carrying load for Caisson 1 was computed to be 334 kips. Since the actual column load carried is 313 kips, the design is considered safe according to the Building Code.

CHAPTER VIII

DISCUSSION AND CONCLUSION

The design of foundations cannot be made in an intelligent and satisfactory manner unless the designer has at least a reasonably accurate conception of the physical properties of the soils involved.

Soil conditions over the Campus were found to be quite uniform. The basic soil strata encountered subject to minor local variation are: organic top soil, brown silty clay, tan silt, brown clay, grey clay, pale brown glacial till or "hardpan", and limestone bedrock. Physical properties of these has been discussed in detail in Chapter III.

A difficult problem encountered in the design of foundations on the Campus arises from the possible severe shrinking and swelling that accompany drying and wetting of the clays. It is evident from experience on the Campus that swelling and shrinking clays introduce foundation movements that so far can only be predicted on basis of experience. These clays can be indentified by their high plasticity index and liquid limit in the neighbourhood of 50 and 80 percent respectively. From these studies it may be inferred that foundations constructed during dry period are subject to heaving in subsequent wet periods, and those constructed during wet periods will undergo settlement in subsequent dry weather. Swelling pressures of about one ton per square foot are commonly shown during consolidation tests on undisturbed samples. For this reason any ground supported floors should be reinforced concrete, and should be structurally independent of walls or columns. If floor movement cannot be tolerated, then a structural floor not in contact with the soil would be required. This make necessary the use of a crawl space under such floors. The crawl space must not extend to the depth where water bearing silt layers are encountered.

Winnipeg soils are subject to the rigours of low winter temperatures. Frozen ground is, therefore, experienced to some degree every winter. When soil and ground water conditions are right, frost heave occurs. It should be noted that the silt is frost-heave susceptible material. The heaving of roadway surfaces and of shallow foundations supported on the silt stratum is usual. Because of this, the removal of the silt to a depth of at least four feet below finished grades is recommended for roadways and parking lots. The protecting against heaving of foundations is generally being met by placing foundations below the depth of frost penetration. As a result of this foundation location, frost action has become a relative minor problem in foundation design on the Campus.

For the varved brown clay, sulphate tests usually show very high sulphate contents, sometimes exceeding 12000 parts per million which is in sufficient concentration to be detrimental to concrete made of ordinary Portland cement. For this reason it is recommended that all concrete in contact with the soil employ sulphate-resistant cement. On the Campus of the University of Manitoba, damages to water-mains has been caused by ground movements and sulphate attacks. Soluble sulphates in the soil caused rapid corrosion of cast-iron pipe which,once weakened, fails in flexure as a result of seasonal soil movement.

Some elementary but very essential factors are now considered. The following paragraphs discuss briefly recommendations concerning foundation design and construction of the various foundation types being used on the Campus based on examinations of the buildings.

(a) Spread Footings: This type of foundation may successfully be supported on clay below the depth of seasonal soil moisture changes. The University of Manitoba has several buildings, such as: the Administration Building, Agricultural Science Building, and Tache Hall; where massive masonary or reinforced concrete footings are supported at least twelve feet below surface grade. Although some of these buildings are at least fifty years old, they are noteworthy for the absence of differential settlements. They are particularly successful where the basement floor are of a structural design, and a crawl space has been provided below the floor to avoid contact between soil and floor. It should be noted that this crawl space should be at least 4 to 6 inches deep to avoid eventual heave affecting the floor.

Analyses of the Elizabeth Dafoe Library footings showed a bearing value of 2500 pounds per square foot on the brown to grey brown silty clay giving a safety factor of about 3. A comparison of the test hole logs on the Campus indicates that bearing values of 2000 pounds per square foot could generally be employed on the brown clay, and the higher values only if justified by actual test borings and laboratory tests.

Spread footings are not recommended in areas where several feet of new fill is required, because the recently placed fill will cause settlement of the entire area which could affect the structures. Also spread footings are not recommended for heavy buildings near the river bank, because of the obvious problem of long-term settlements of the structure, and because they impose additional loads on the river bank, reducing the factor of safety against sliding. The River and Streams Act also requires that structures be located no closer to the river than 150 feet measured from the summer water's edge.

Settlement analyses for selected footings of Administration Building and Elizabeth Dafoe Library indicated a wide range of values when computed by method 1 and 2, as shown in Table (5.3) and (5.5). This discrepancy in settlement values may be accounted for by the fact that values computed by method 2 consider the effect of preconsolidation while those computed using method 1 do not. This confirms the belief that for preconsolidated soils, the settlement which will occur is usually small that settlement analysis is rarely of practical interest.

Deep foundations generally do not suffer from effects of seasonal moisture changes. The result of being below this depth is that the foundations are not affected by shrinking and swelling of the soils. If foundations are founded on the "hardpan", high end-bearing value of 15 kips per square foot can be used and hence the foundations can support larger loads. If the foundations are founded in the brown clay or the grey clay, the question of adequate bearing capacity and tolerable settlement must be answered.

With foundations in these materials settlement can be appreciable but are generally smaller than the settlements that would be encountered if the foundation was at a shallower depth.

(b) Friction Piles: For medium to lightly loaded structures, cast-in-place augered friction piles may be used providing they do not extend into the deeper seepage zone. These piles may employ an allowable friction value of 300 pounds per square foot which will give a safety factor of at least 2 to 3. Also for friction piles, the support in the upper 6 to 8 feet of the clays should be neglected because of possible soil shrinkage which makes such support unreliable. It should be noted also that the Winnipeg Code requires that the frictional support of cohesive soils be ignored for a depth of at least 5 feet below the basement level or 10 feet below adjacent ground surface whichever is deeper.

(c) Driven, End-Bearing Piles: Because of heavy seepage which is encountered in many localities on the

Campus, the most satisfactory foundations for all but the lightest structures on the Campus are driven precast piles end-bearing on "hardpan" or bedrock. Driven piles are considered more practical than cast-in-place augered piles, as these avoid the seepage problem that would otherwise be encountered. On the other hand, cast-in-place augered piles are quicker and less costly to install if heavy seepage does not occur.

The bearing capacity of driven precast, concrete piles, depends almost entirely on the capacity of the material upon which the point finds its bearing, and on the degree to which the point of the pile has a satisfactory seat on the bearing material. Locally according to the Winnipeg Building Code, loads of 100, 140 and 180 kip may be used for 12, 14, and 16 inch precast, prestressed, hexagonal piles respectively.

The danger of damage to the pile because of the possibility of eccentric support between the tip of the pile and bedrock or hardpan is more critical to the Building in the case of a single pile. It is, therefore, recommended that large loads should be carried by clusters of piles. On the Campus, however, the danger from eccentric contact or partial contact is probably not too severe because of the relatively horizontal surface of the bedrock.

Experience showed that the driving of a pile in a cluster tends to lift previously driven, nearby piles. It is therefore recommended that, after each cluster of piles is driven, all piles in the cluster be re-driven to ensure that they are firmly seated on "hardpan" or bedrock.

(d) Caissons: For buildings which will support very heavy loads, it is necessary to design for minimum total and differential settlements. This can be best achieved by using caissons supported on or into the bedrock if seepage can be readily controlled. A bearing value of 60 kips per square foot is recommended for caissons drill into sound white limestone bedrock and through any fractured zone near the top of limestone. In cases of some but not too severe seepage, a combination of the use of steel liners plus pumping may overcome the seepage difficulties on the Campus so that caissons will be successfully installed.

In concluding, it is hoped that the findings of this thesis will provide insight into the investigations of soil conditions and foundation problems on the University of Manitoba Campus. The author would recommend that future work include more consolidation tests, especially on samples from the deeper clays for which very little consolidation data are available. The drilling of more test borings beyond the central core of the Campus is required to ensure that the soil profiles follow the same general trends as shown in this study. In much of the central core, sufficient data now exists for most design purposes, except that water conditions, because of their seasonal and other fluctuations, cannot be predicted without boring just prior to commencing construction.

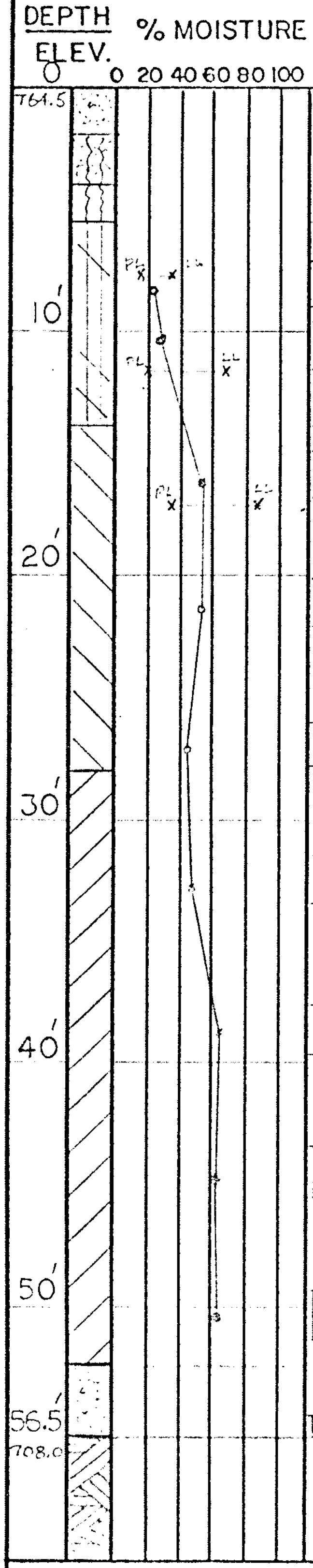
APPENDIX A

BORING RECORDS

LOG OF TEST HOLE NO. 1

COORDINATES E-5

LOCATION MARY SPEECHLY HALL



DESCRIPTION

SAND & GRAVEL FILL

OF SAND, GRAVEL & BLACK ORGANIC CLAYEY SILT

BLACK ORGANIC CLAYEY SILT

BROWN CLAYEY SILT

TAN SILT, SOFT; SOME CLAY

GREY BROWN CLAYEY SILT; SEEPAGE FROM STRATUM IN THIS LAYER

SOME CLAY LAYER

HORIZONTALLY STRATIFIED BROWN CLAY, GYPSUM POCKETS

GREY CLAY

SAND, SILT, FINE GRAVEL ROCK FLOUR FREE WATER

DISTURBED SAMPLE

HARD

LIMESTONE BEDROCK

DIAMOND DRILL

HOLE ADVANCED BY 4 IN. POWER AUGER & DIAMOND DRILL

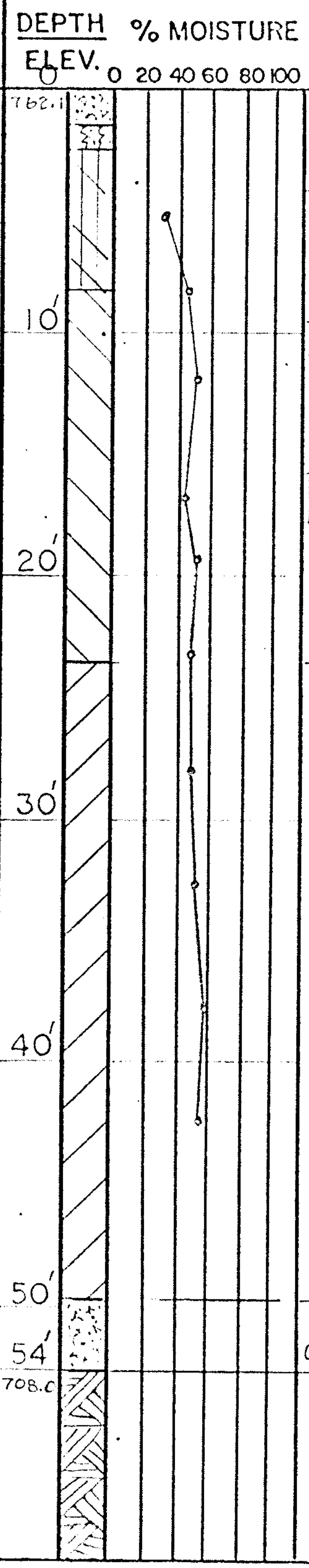
LOGGED BY H.E.D DATE JAN 28/61

REFERENCE FROM REPORT PREPARED FOR WAISMAN-ROSS ASSOCIATES BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 2

COORDINATES E-5

LOCATION MARY SPEECHLY HALL



DESCRIPTION

SAND & GRAVEL FILL

ORGANIC TOP SOIL, SAND, GRAVEL NUGGETY

BROWN CLAYEY SILT

TAN, SLIGHTLY CLAYEY SILT; LIGHT SEEPAGE

HORIZONTALLY STRATIFIED BROWN CLAY

GYPSUM POCKETS

GREY CLAY

SAND, SILT, FINE GRAVEL ROCKFLOUR FREE WATER (NEGLIGIBLE RECOVERY ON AUGER)

LIMESTONE BEDROCK

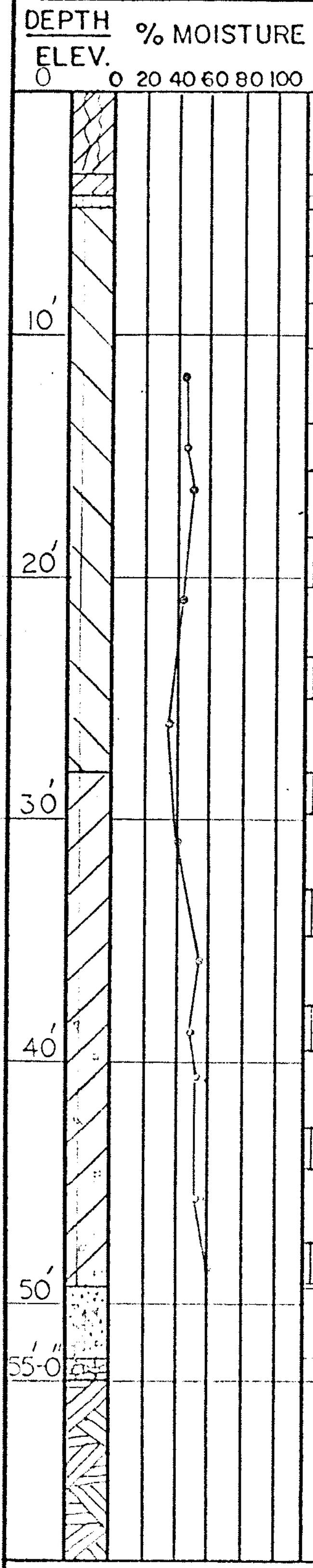
HOLE ADVANCED BY 4 IN. POWER AUGER

LOGGED BY H.F.D DATE JAN. 30/61

REFERENCE FROM REPORT PREPARED FOR WAISMAN-ROSS ASSOCIATES BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 3
COORDINATES E-5
LOCATION MARY SPEECHLY -HALL

DESCRIPTION



GREY-BLACK ORGANIC VERY SILTY CLAY

GREY-BROWN V. SILTY CLAY STIFF

6" LAYER TAN SILT

BROWN TO GREY BROWN

SILTY CLAY STIFF

SILT POCKETS

$q_u = 2230$
$\gamma_m = 109$
$S = 100$

MANY WHITE GYPSUM POCKETS

BROWNISH GREY TO GREY VARVED SILTY CLAY

$q_u = 2650$
$\gamma_m = 108$
$S = 100$

STIFF TO FIRM SMALL GYPSUM -POCKETS

SMALL LIMESTONE FRAGMENTS

$q_u = 2130$
$\gamma_m = 102$
$S = 100$

PALE GREY TILL SANDY SILT, ROCK FRAGS. DENSE

BADLY FRACTURED

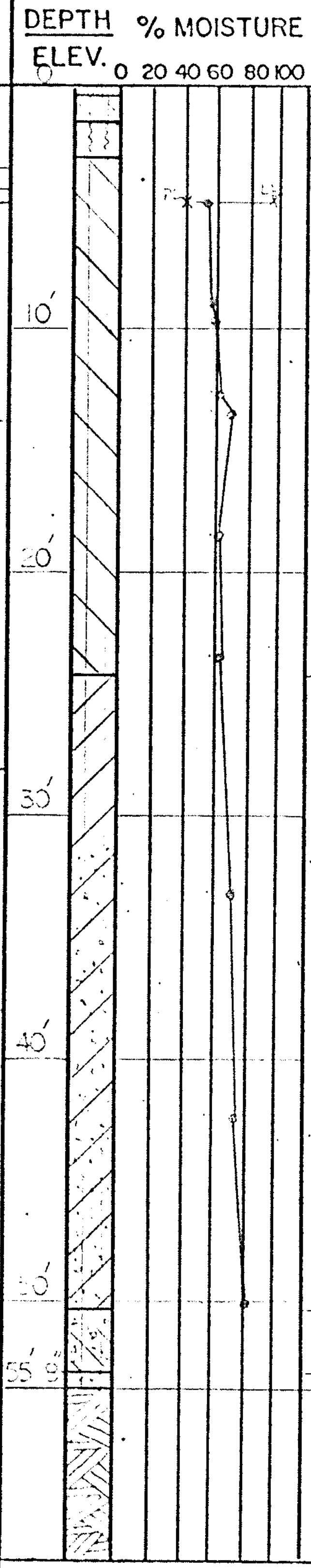
WHITE LIMESTONE BEDROCK

HOLE ADVANCED BY 5" FLIGHT AUGER
LOGGED BY H.P. DATE MAR. 2/62
REFERENCE FROM REPORT PREPARED FOR WAISMAN, ROSS & ASSOCIATES BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 4
COORDINATES F-3
LOCATION ELIZABETH DAFOE -LIBRARY

DESCRIPTION

4" TOP SOIL AND ROOTS



GREY-BROWN SILTY CLAY FILL

BLACK ORGANIC CLAYEY SILT

BROWN TO GREY-BROWN VARVED SILTY CLAY

SOFT VERY SILTY

SEEPAGE

SILT AND GYPSUM POCKETS STIFF

GREY SILTY CLAY

SEEPAGE

SMALL SILT POCKETS

MED. STIFF TO SOFT

SAND & LIMESTONE FRAGS.

SANDY AND SILTY CLAY ROCK FLOUR, LIMESTONE FRAGS.

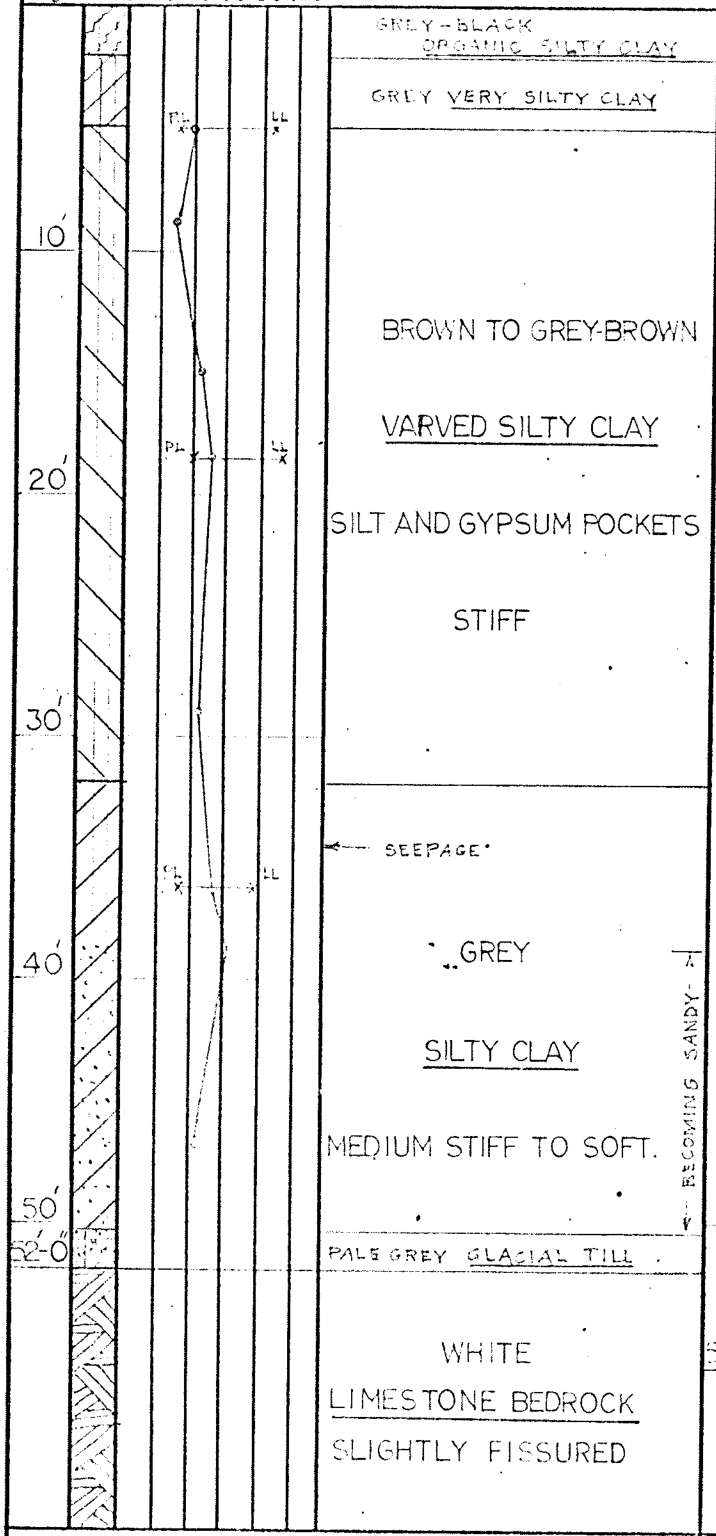
WELL CEMENTED GLACIAL TILL

WHITE LIMESTONE SLIGHTLY FISSURED

HOLE ADVANCED BY 5" FLIGHT AUGER
LOGGED BY [illegible] & S.M. DATE MAY 16/62
REFERENCE FROM REPORT PREPARED FOR GREEN, BLANKSTEIN, RUSSELL & ASSOCIATES BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 5
COORDINATES F-3
LOCATION ELIZABETH DAFOE
-LIBRARY

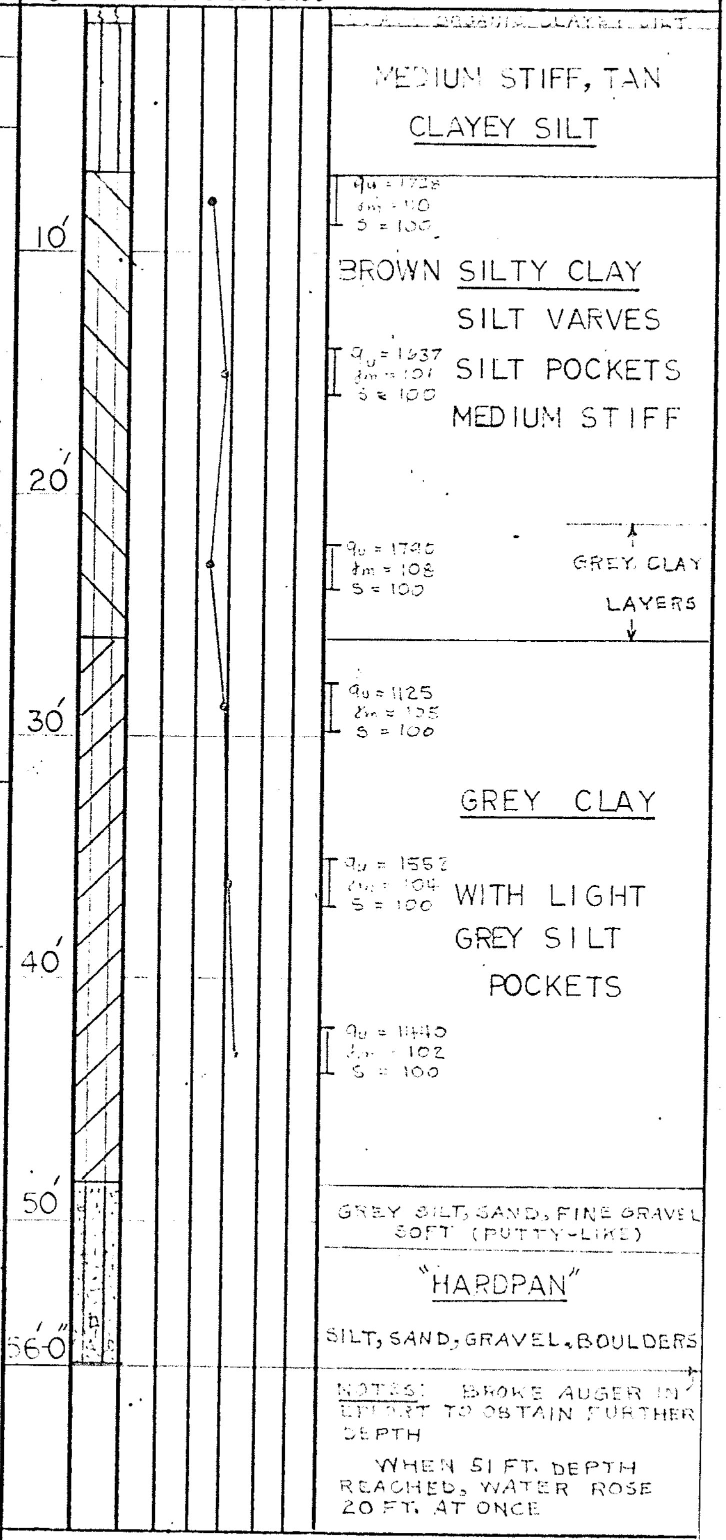
DEPTH ELEV. | % MOISTURE | DESCRIPTION



HOLE ADVANCED BY " FLIGHT AUGER
LOGGED BY M.M. & S.M. DATE MAY 15/6?
REFERENCE FROM REPORT PREPARED FOR GREEN, BLANKSTEIN, RUSSELL & ASSOCIATES BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 6
COORDINATES E-2
LOCATION NEW SCIENCE
-BUILDING

DEPTH ELEV. | % MOISTURE | DESCRIPTION



NOTES: BROKE AUGER IN EFFORT TO OBTAIN FURTHER DEPTH

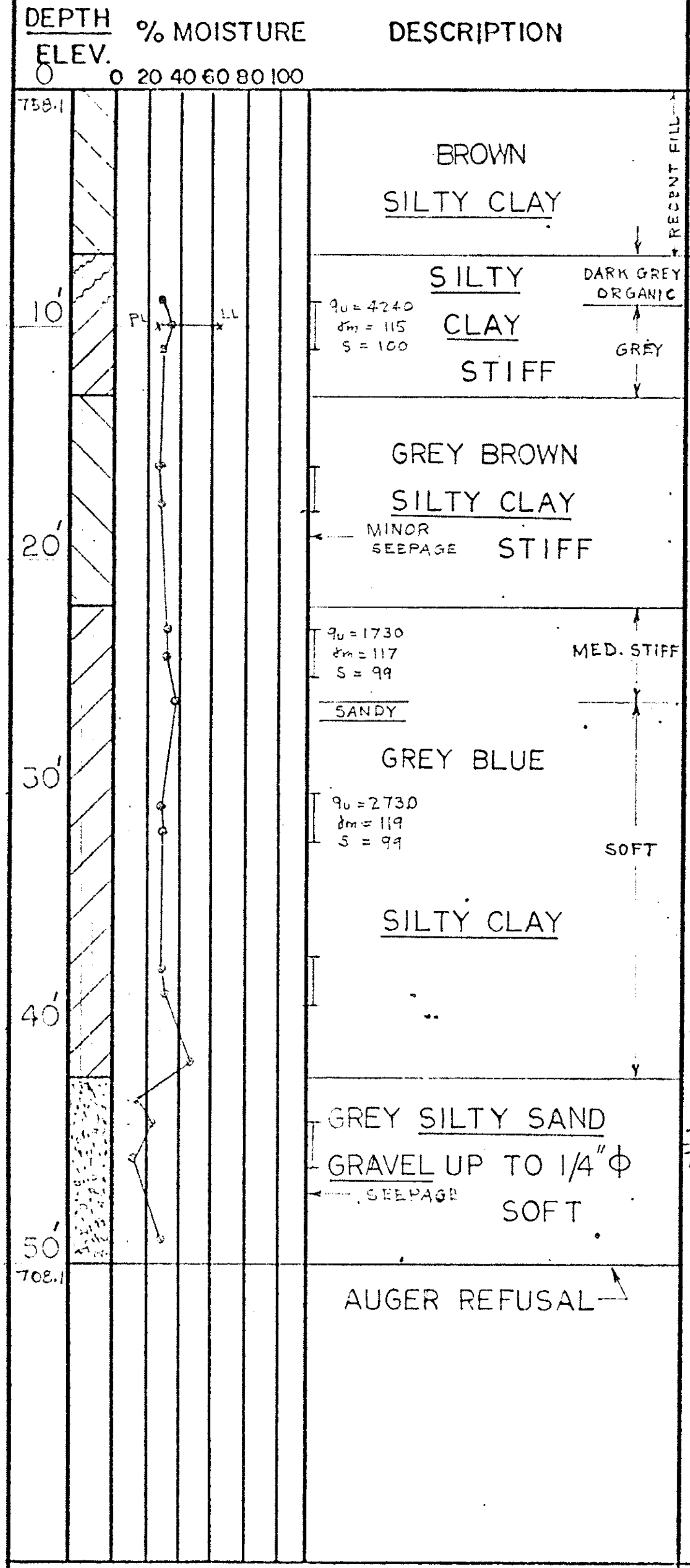
WHEN 51 FT. DEPTH REACHED, WATER ROSE 20 FT. AT ONCE

HOLE ADVANCED BY 4" DIAM. PENN-DRILL
LOGGED BY J.M. DATE FEB. 13/59
REFERENCE FROM REPORT PREPARED FOR GREEN, BLANKSTEIN, RUSSELL & ASSOCIATES BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 7

COORDINATES F-2

LOCATION PERIMETER ROAD NORTH OF LIBERAL ARTS COLLEGE



HOLE ADVANCED BY 5 IN. AUGER

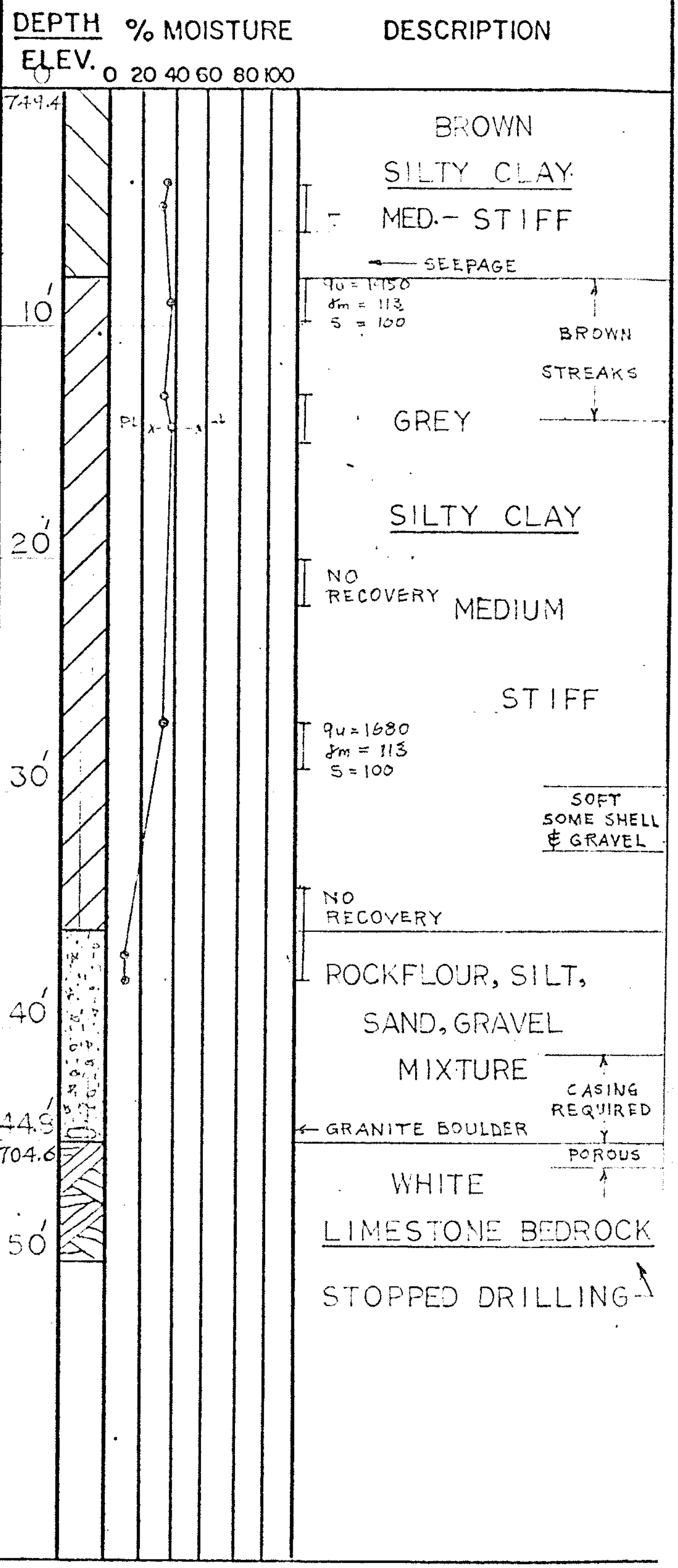
LOGGED BY H.P. DATE MAY 5/61

REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 8

COORDINATES G-2

LOCATION PERIMETER ROAD NORTH OF LIBERAL ARTS COLLEGE



HOLE ADVANCED BY 5" FLIGHT AUGER

LOGGED BY H.P & M.M DATE MAY 4/61

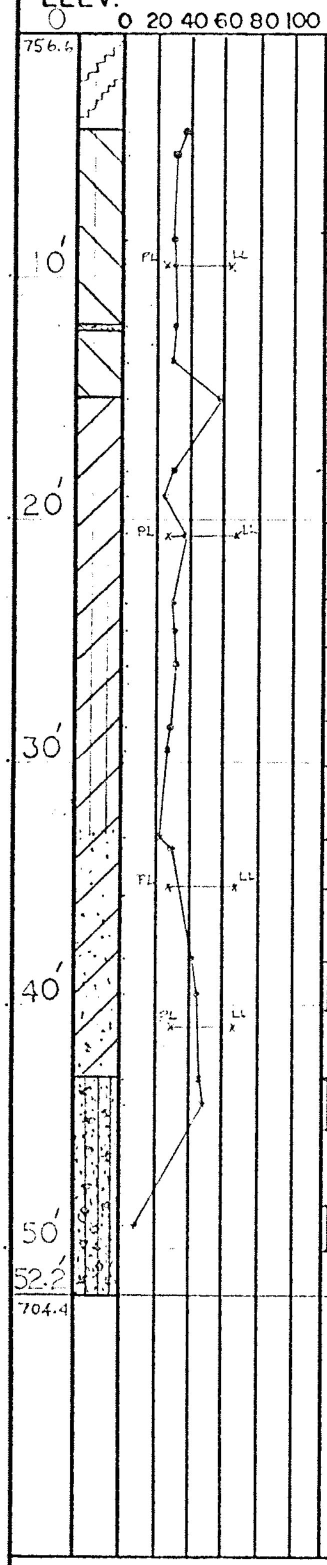
REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 9

COORDINATES G-4

LOCATION PARKING LOT-B. EAST OF ELIZABETH DAFOE LIBRARY

DEPTH / ELEV. — % MOISTURE — DESCRIPTION



ORGANIC SILTY CLAY — BLACK ↓ GREY

$q_u = 4630$, $\gamma_m = 115$, $S = 93$

BROWN SILTY CLAY STIFF

$q_u = 3920$, $\gamma_m = 117$, $S = 93$

WATER BEARING GREY SANDY SILT — SOFT

$q_u = 3800$, $\gamma_m = 118$, $S = 95$

$q_u = 4170$, $\gamma_m = 127$, $S = 99$

GREY SILTY CLAY MED. STIFF

$q_u = 2970$, $\gamma_m = 117$, $S = 96$

$q_u = 2970$, $\gamma_m = 121$, $S = 99$

BLUE CLAYEY SAND, SOFT, SMALL SHELLS

BLUE SANDY CLAY MED. STIFF

$q_u = 1830$, $\gamma_m = 110$, $S = 97$

$q_u = 1610$, $\gamma_m = 110$, $S = 100$

ROCKFLOUR, SILT, SAND, GRAVEL MIXTURE

$q_u = 1510$, $\gamma_m = 143$, $S = 100$

AUGER REFUSAL

HOLE ADVANCED BY 5" FLIGHT AUGER

LOGGED BY H.P. DATE MAY 3/61

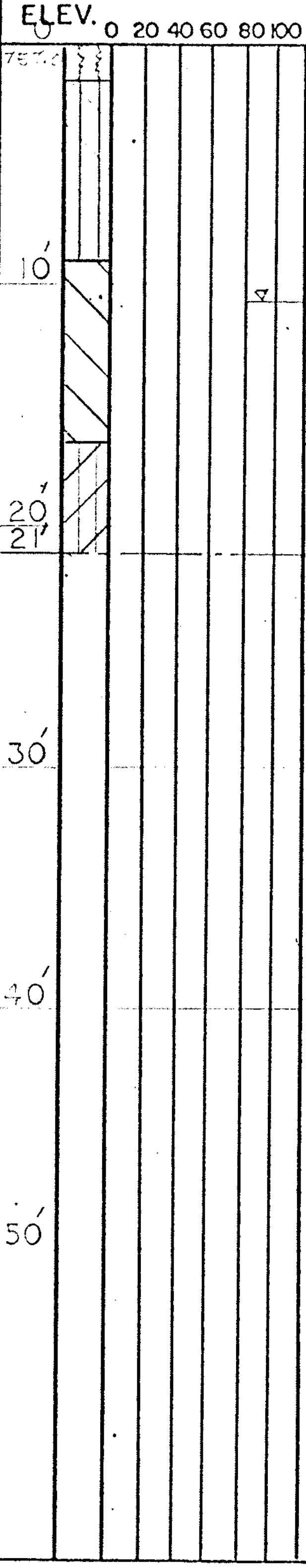
REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 10

COORDINATES G-4

LOCATION PARKING LOT-B. EAST OF ELIZABETH DAFOE LIBRARY

DEPTH / ELEV. — % MOISTURE — DESCRIPTION



BLACK ORGANIC CLAYEY SILT

BROWN CLAYEY SILT MED. STIFF

MAY 8/61

BROWN CLAY MED. STIFF

WATER-BEARING SEAM

GREY SILTY CLAY MED. STIFF

HOLE TERMINATED

HOLE ADVANCED BY 5" AUGER

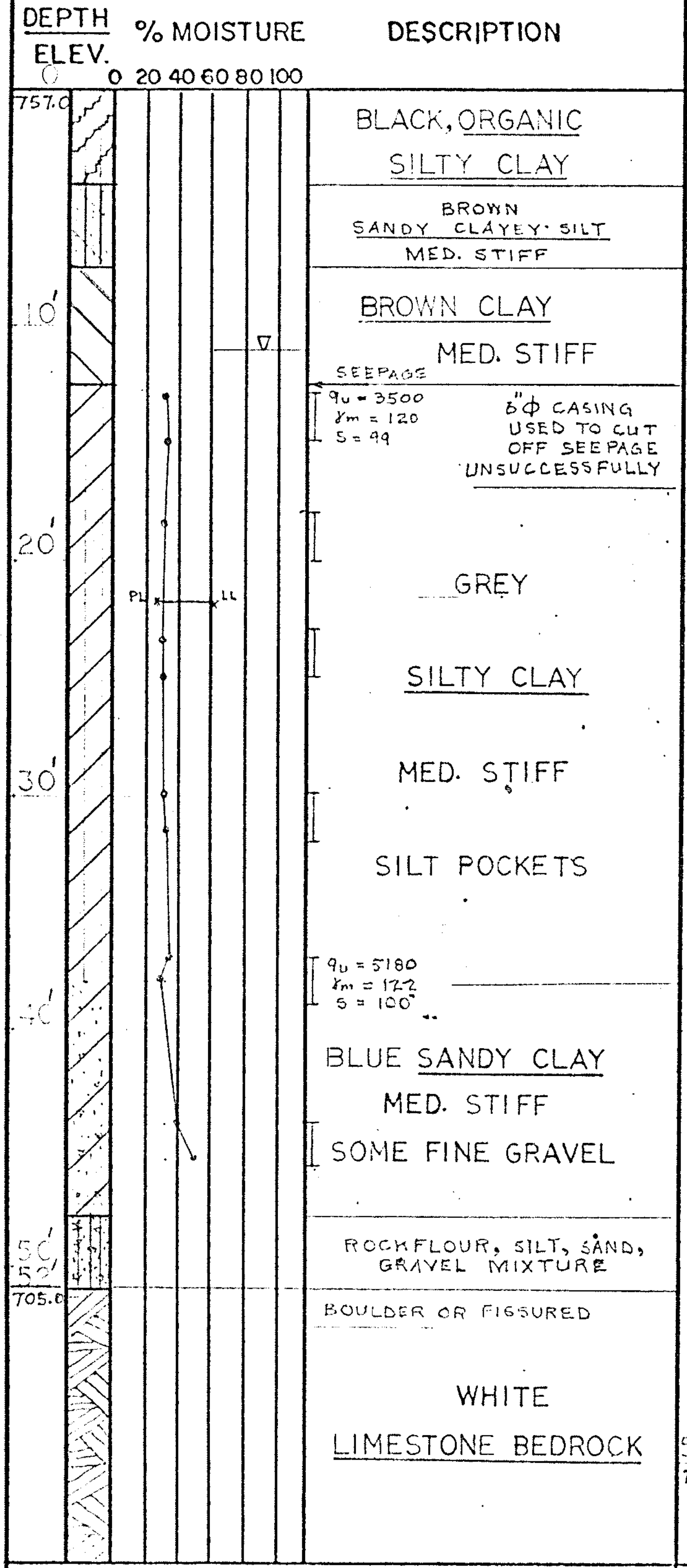
LOGGED BY H.P. DATE MAY 5/61

REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 11

COORDINATES F-4

LOCATION PARKING LOT-B EAST OF ELIZABETH DAFOE LIBRARY



BLACK, ORGANIC SILTY CLAY

BROWN SANDY CLAYEY SILT MED. STIFF

BROWN CLAY MED. STIFF

SEEPAGE

$q_u = 3500$
$\gamma_m = 120$
$S = 99$

6"Φ CASING USED TO CUT OFF SEEPAGE UNSUCCESSFULLY

GREY SILTY CLAY MED. STIFF SILT POCKETS

$q_u = 5180$
$\gamma_m = 122$
$S = 100$

BLUE SANDY CLAY MED. STIFF SOME FINE GRAVEL

ROCKFLOUR, SILT, SAND, GRAVEL MIXTURE

BOULDER OR FISSURED

WHITE LIMESTONE BEDROCK

HOLE ADVANCED BY 6" & 5" AUGER

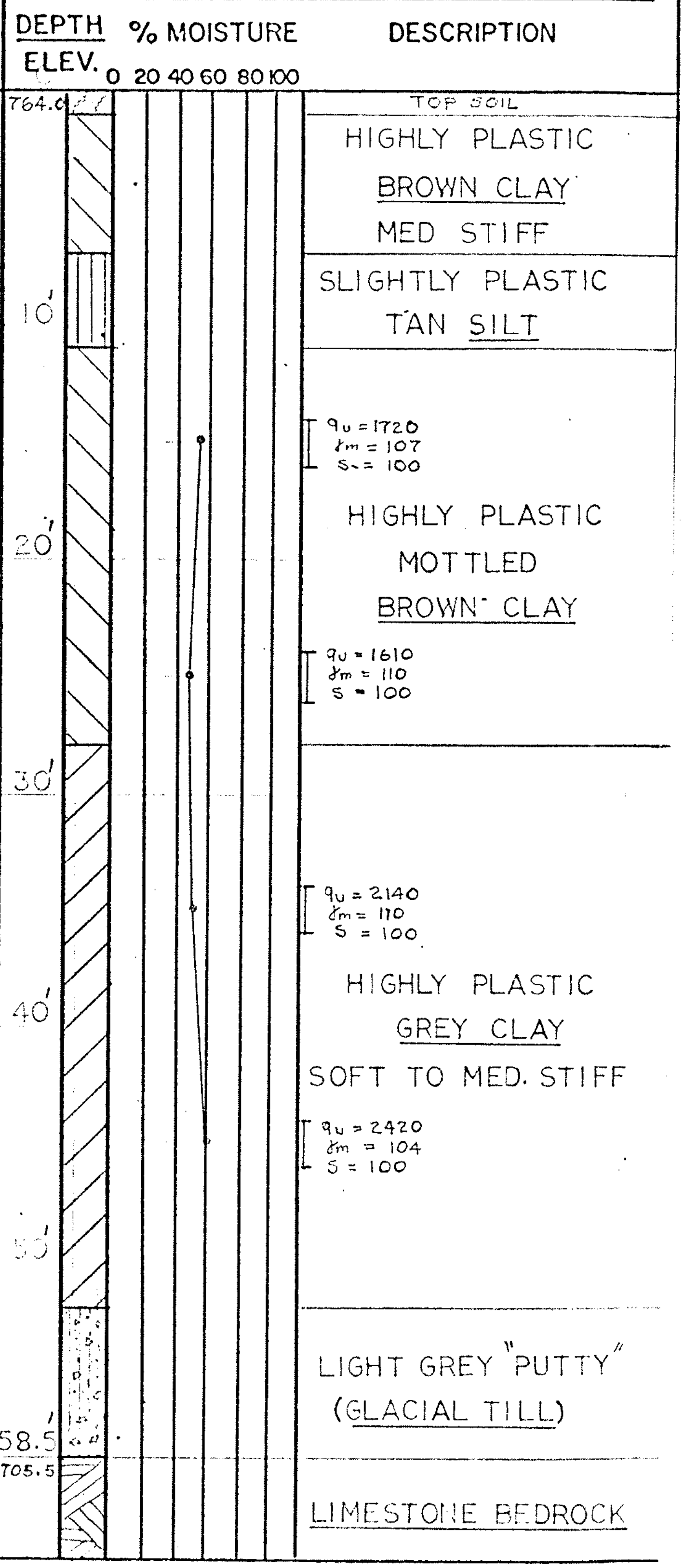
LOGGED BY H.P. DATE MAY 8/61

REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY BARACOS AND MARANTZ

LOG OF TEST HOLE NO. 12

COORDINATES E-4

LOCATION STUDENT UNION - BUILDING



TOP SOIL

HIGHLY PLASTIC BROWN CLAY MED STIFF

SLIGHTLY PLASTIC TAN SILT

$q_u = 1720$
$\gamma_m = 107$
$S = 100$

HIGHLY PLASTIC MOTTLED BROWN CLAY

$q_u = 1610$
$\gamma_m = 110$
$S = 100$

$q_u = 2140$
$\gamma_m = 110$
$S = 100$

HIGHLY PLASTIC GREY CLAY SOFT TO MED. STIFF

$q_u = 2420$
$\gamma_m = 104$
$S = 100$

LIGHT GREY "PUTTY" (GLACIAL TILL)

LIMESTONE BEDROCK

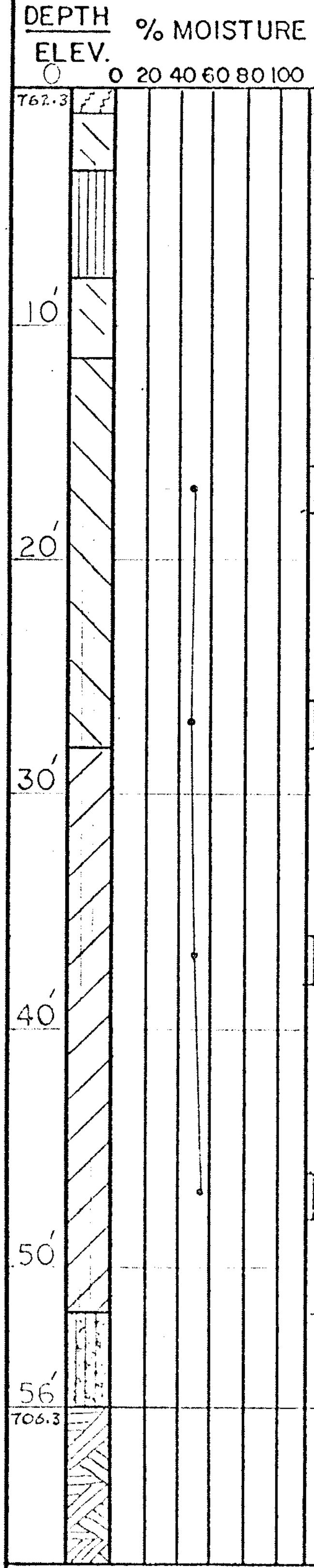
HOLE ADVANCED BY 16" POWERED AUGER

LOGGED BY D. J. PRICE DATE DEC. 7/67

REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY CROSIER GREENBERG & PARTNERS

LOG OF TEST HOLE NO. 13
COORDINATES E-3
LOCATION STUDENTS UNION - BUILDING

DEPTH ELEV. | % MOISTURE | DESCRIPTION



TOP SOIL

HIGHLY PLASTIC BROWN CLAY MED. STIFF

SLIGHTLY PLASTIC TAN SILT

HIGHLY PLASTIC BROWN CLAY MED. STIFF

HIGHLY PLASTIC, MOTTLE BROWN CLAY MED. STIFF

$q_u = 1830$
$\gamma_m = 104$
$S = 96$

$q_u = 2130$
$\gamma_m = 108$
$S = 98$

HIGHLY PLASTIC GREY CLAY SOFT TO MED. STIFF SILT POCKETS

$q_u = 1270$
$\gamma_m = 107$
$S = 99$

$q_u = 1640$
$\gamma_m = 105$
$S = 100$

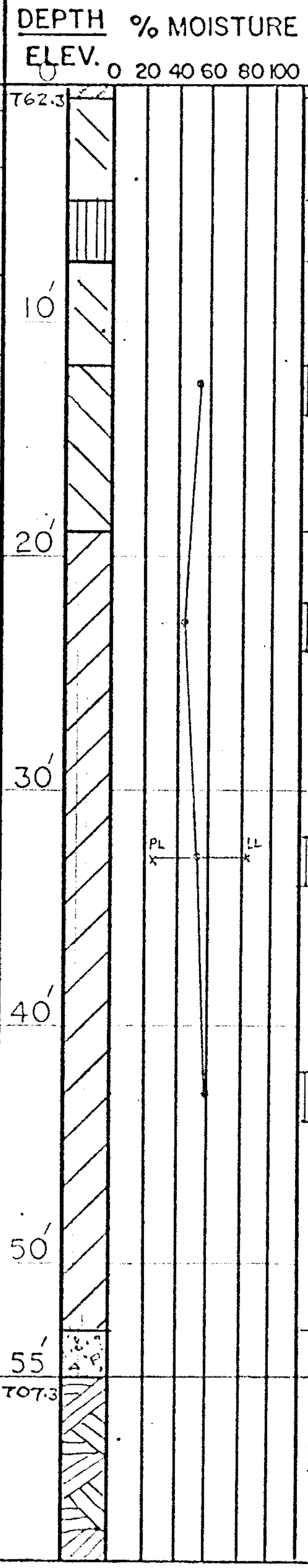
LIGHT GREY PUTTY

LIMESTONE BEDROCK

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY D.J. PRICE DATE DEC. 7/67
REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY CROSIER GREENBERG & PARTNERS

LOG OF TEST HOLE NO. 14
COORDINATES D-4
LOCATION STUDENTS UNION - BUILDING

DEPTH ELEV. | % MOISTURE | DESCRIPTION



TOP SOIL

HIGHLY PLASTIC BROWN CLAY MED. STIFF

SLIGHTLY PLASTIC TAN SILT

HIGHLY PLASTIC BROWN CLAY MED. STIFF

$q_u = 1750$
$\gamma_m = 107$
$S = 100$

PLASTIC MOTTLED GREY-BROWN CLAY

$q_u = 2220$
$\gamma_m = 111$
$S = 98$

HIGHLY PLASTIC GREY CLAY SOFT TO MED STIFF

$q_u = 1840$
$\gamma_m = 112$
$S = 96$

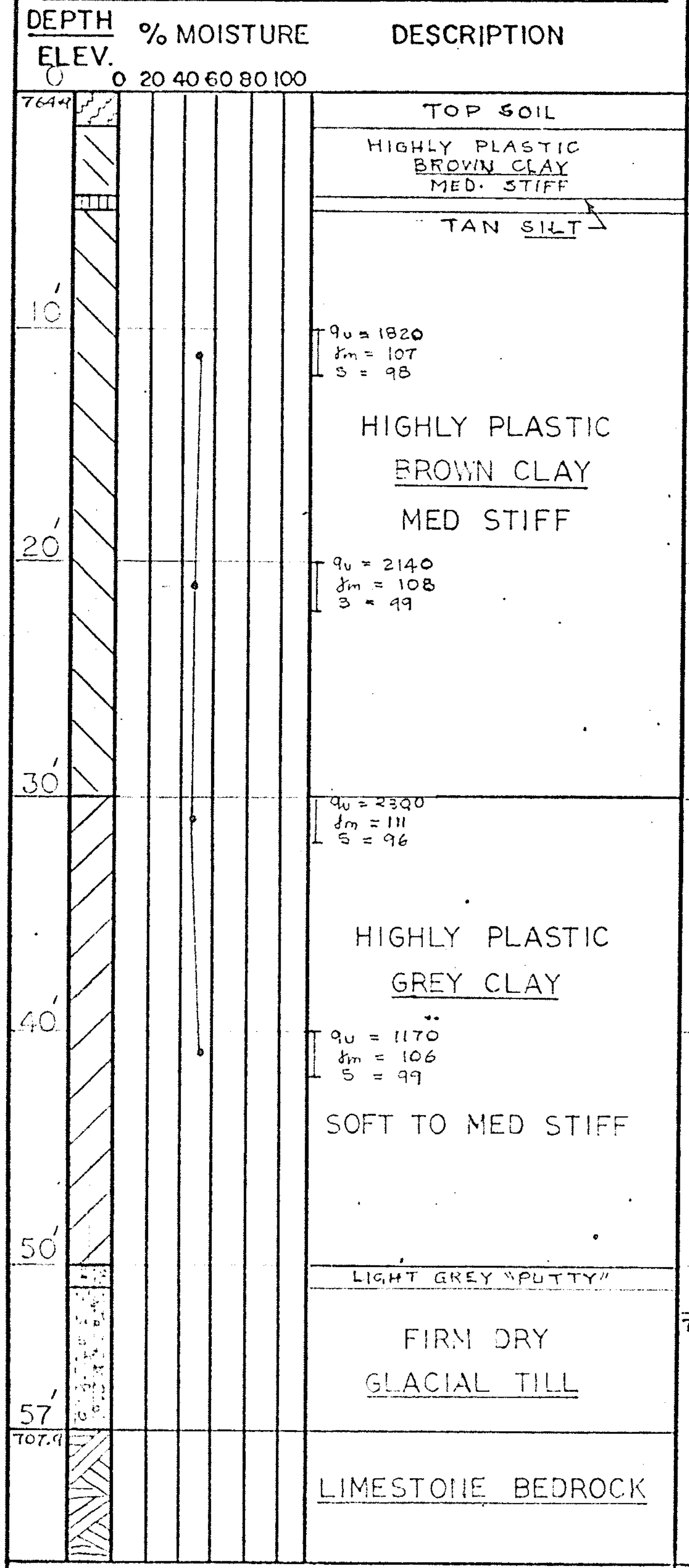
$q_u = 1790$
$\gamma_m = 110$
$S = 99$

FIRM, DRY GLACIAL TILL

LIMESTONE BEDROCK

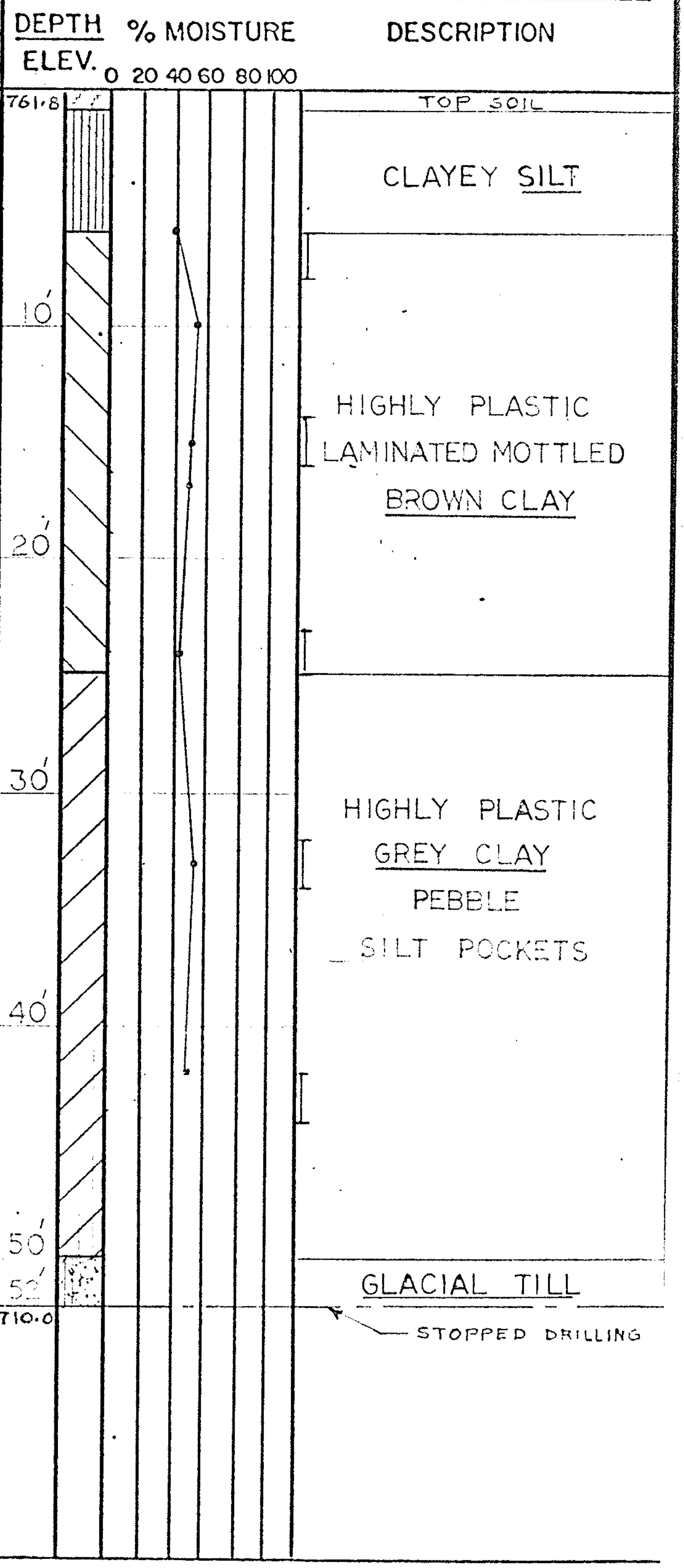
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY D.J. PRICE DATE DEC. 6/67
REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY CROSIER GREENBERG & PARTNERS

LOG OF TEST HOLE NO. 15
COORDINATES D-3
LOCATION STUDENTS UNION - BUILDING



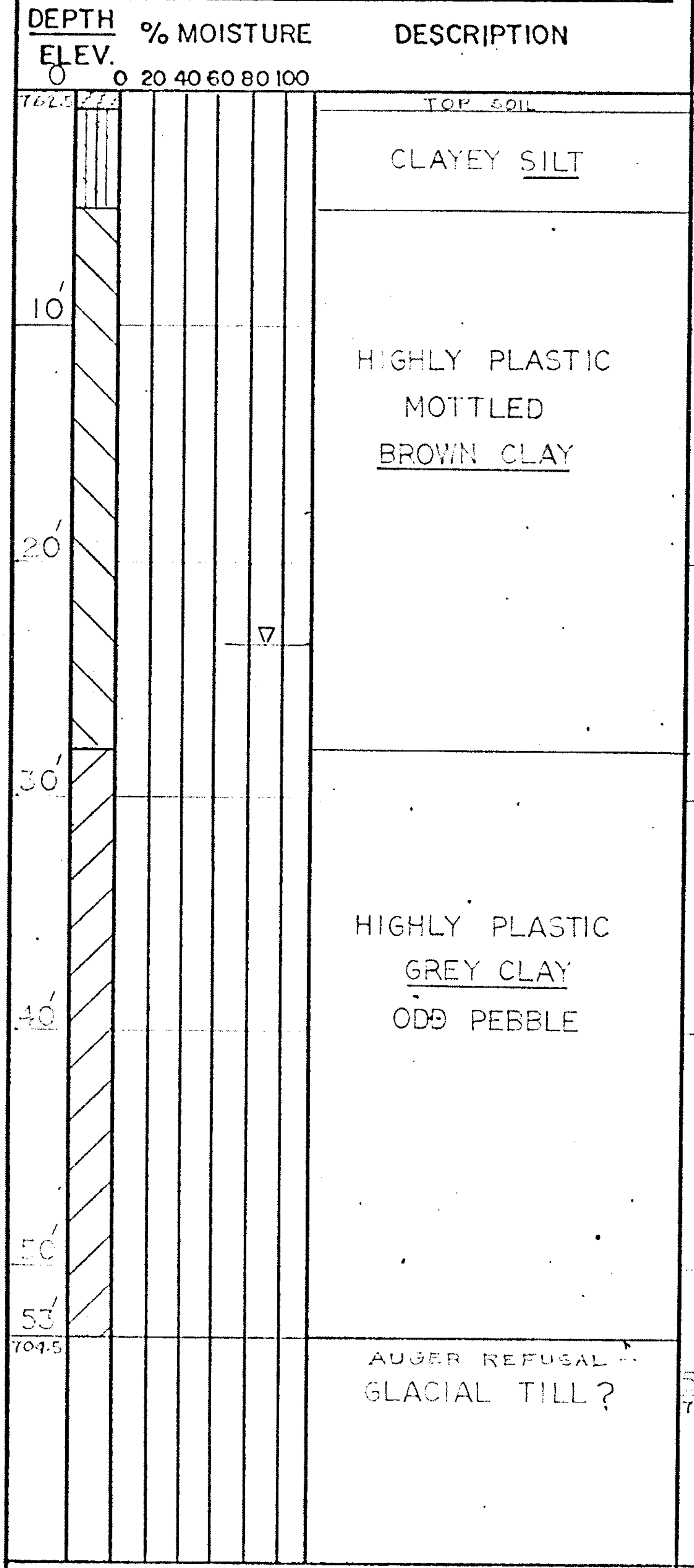
HOLE ADVANCED BY 16" POWERED AUGER
LOGGED BY D. J. PRICE DATE DEC. 6/67
REFERENCE FROM REPORT PREPARED FOR THE UNIVERSITY OF MANITOBA BY CROSIER GREENBERG & PARTNERS

LOG OF TEST HOLE NO. 16
COORDINATES D-4
LOCATION SCHOOL OF MUSIC



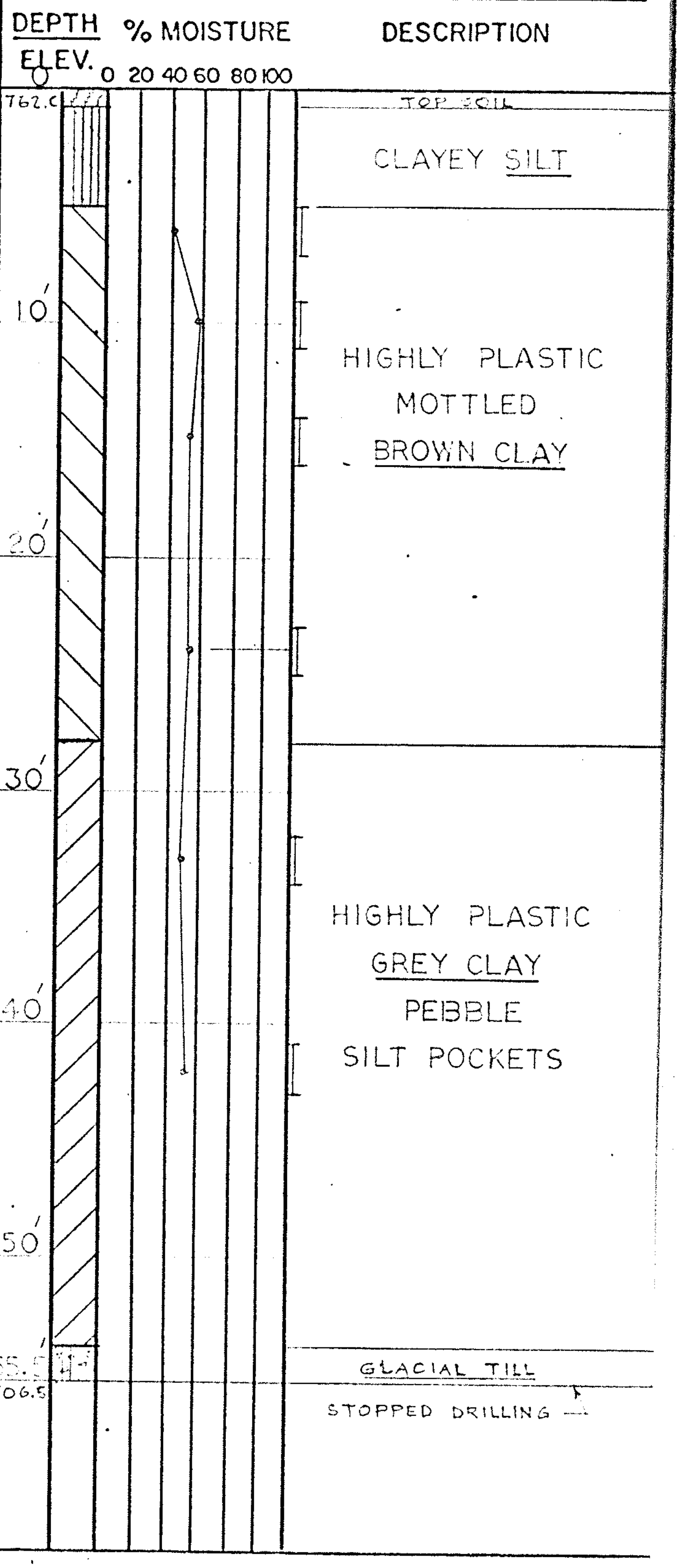
HOLE ADVANCED BY ROTARY DRILL
LOGGED BY M. H. G. DATE APR.30/64
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER SEARLE BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 17
COORDINATES D-4
LOCATION SCHOOL OF MUSIC



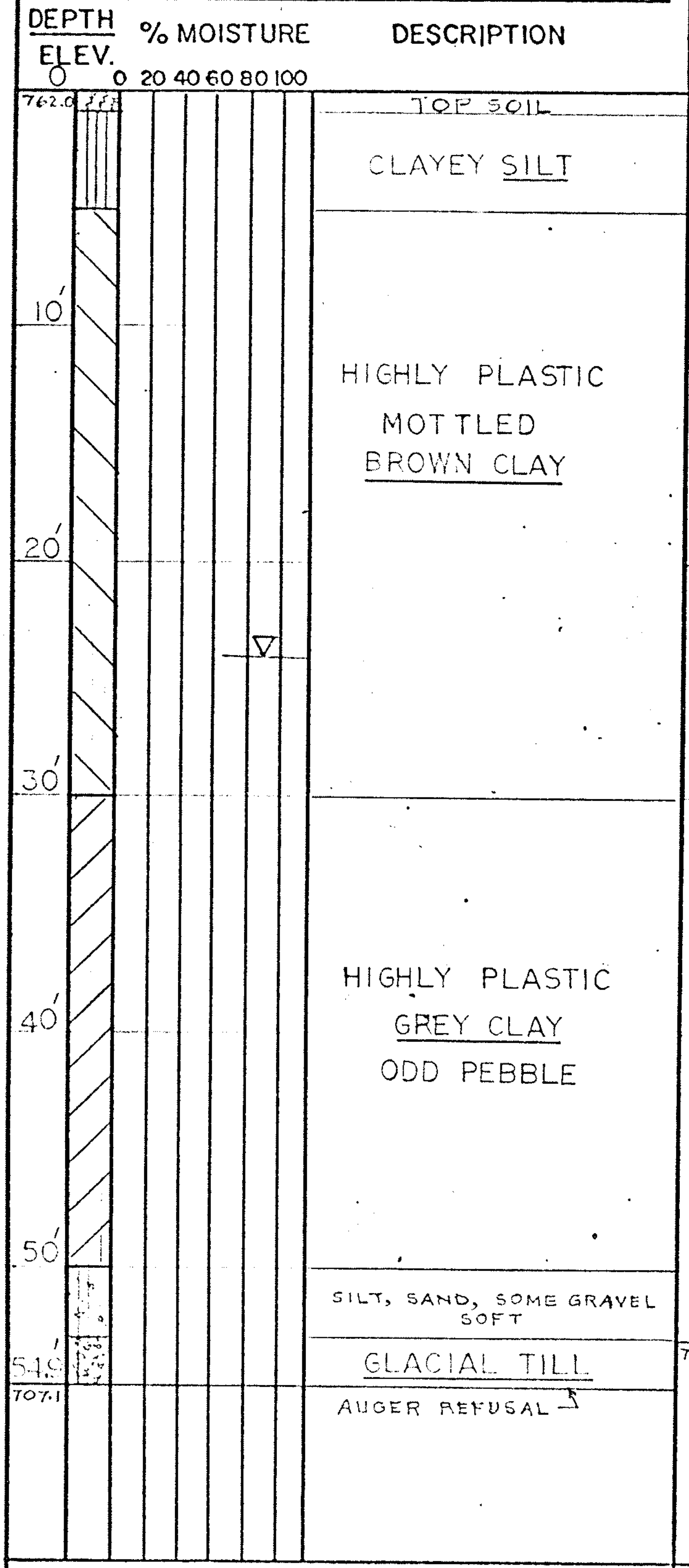
HOLE ADVANCED BY ROTARY DRILL
LOGGED BY M.H.G. DATE APR. 30/64
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER SEARLE BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 18
COORDINATES D-4
LOCATION SCHOOL OF FINE ARTS



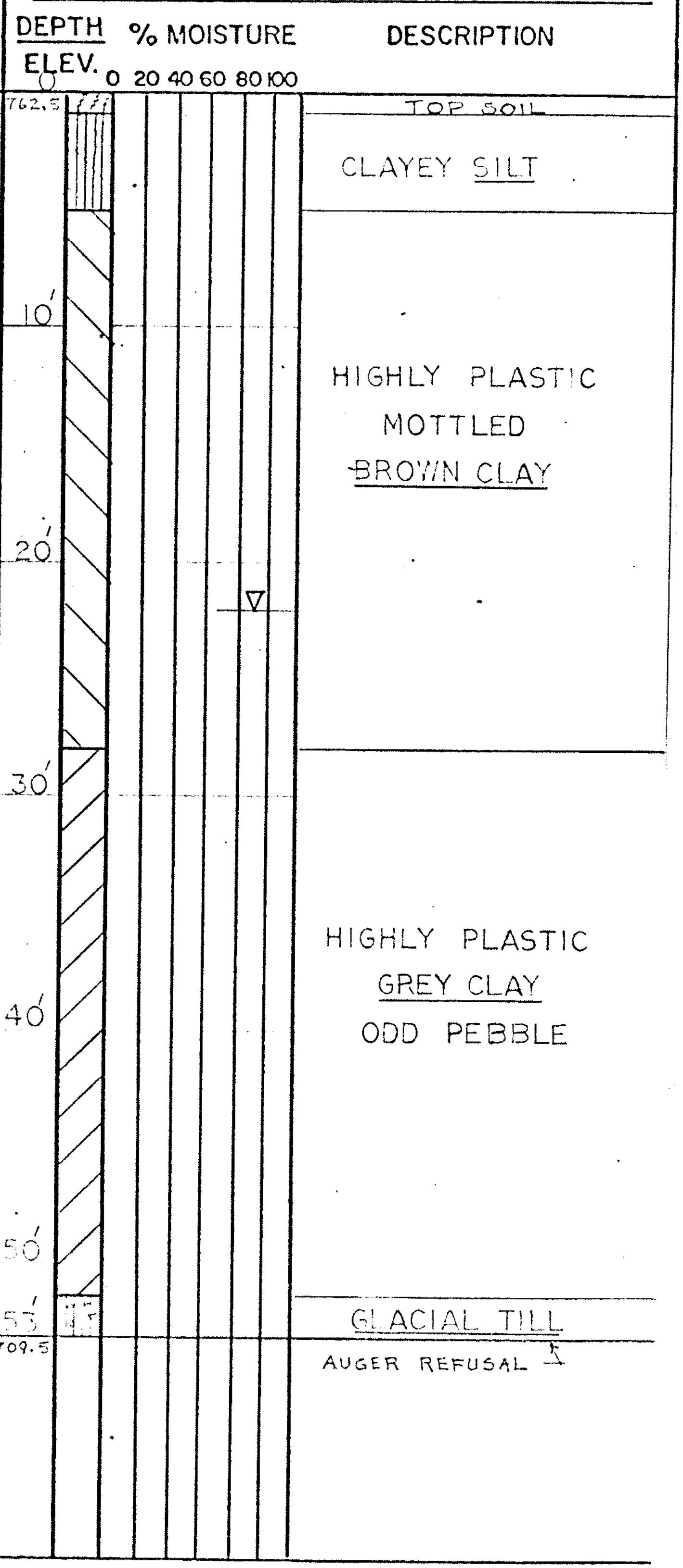
HOLE ADVANCED BY ROTARY DRILL
LOGGED BY M.H.G. DATE
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER SEARLE BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 19
COORDINATES C-4
LOCATION SCHOOL OF FINE ARTS



HOLE ADVANCED BY ROTARY DRILL
LOGGED BY M.H.G. DATE APR.30/64
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER SEARLE BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 20
COORDINATES D-4
LOCATION SCHOOL OF FINE ARTS

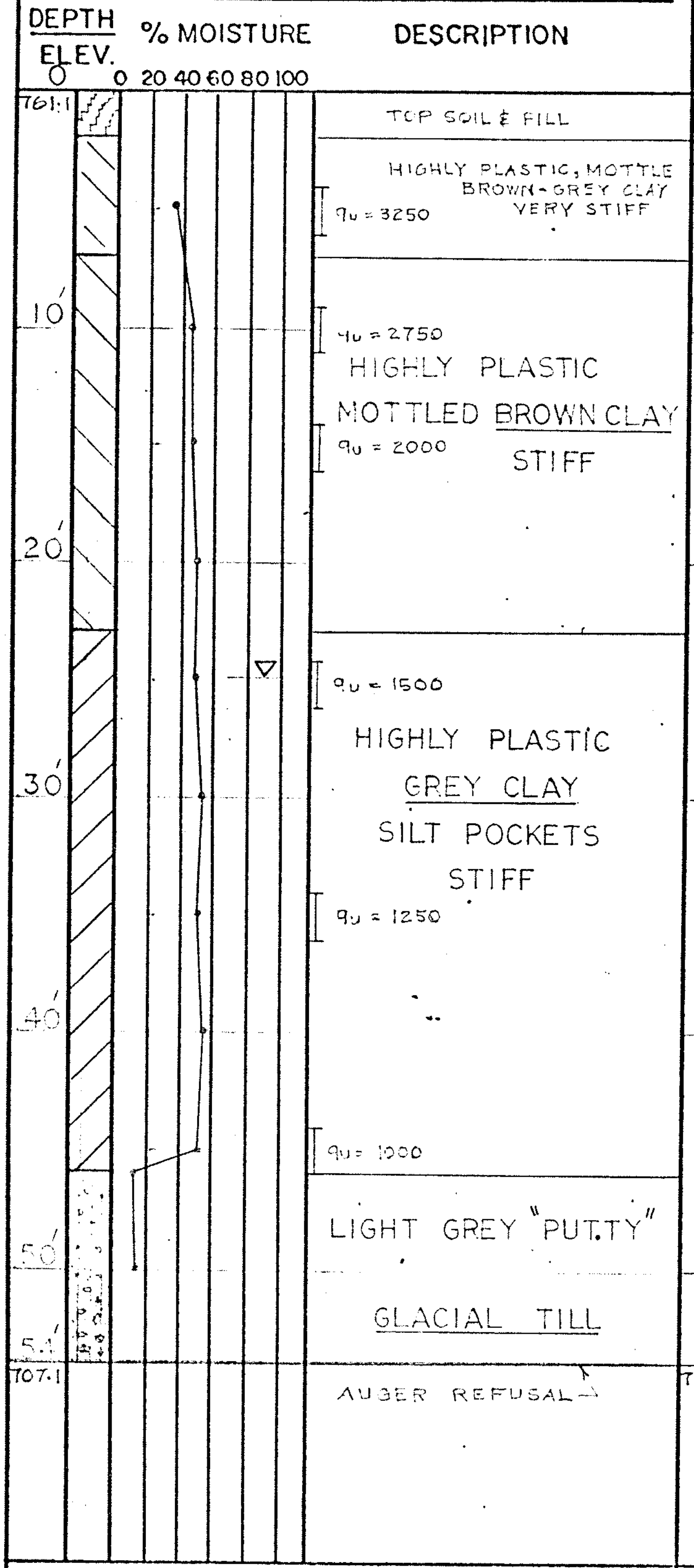


HOLE ADVANCED BY ROTARY DRILL
LOGGED BY M.H.G. DATE APR.30/64
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER SEARLE BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 21

COORDINATES F-4

LOCATION ARTS-ISBISTER -BUILDING



TOP SOIL & FILL

HIGHLY PLASTIC, MOTTLE BROWN-GREY CLAY VERY STIFF

HIGHLY PLASTIC MOTTLED BROWN CLAY STIFF

HIGHLY PLASTIC GREY CLAY SILT POCKETS STIFF

LIGHT GREY "PUTTY" GLACIAL TILL

AUGER REFUSAL

HOLE ADVANCED BY 16" POWER AUGER

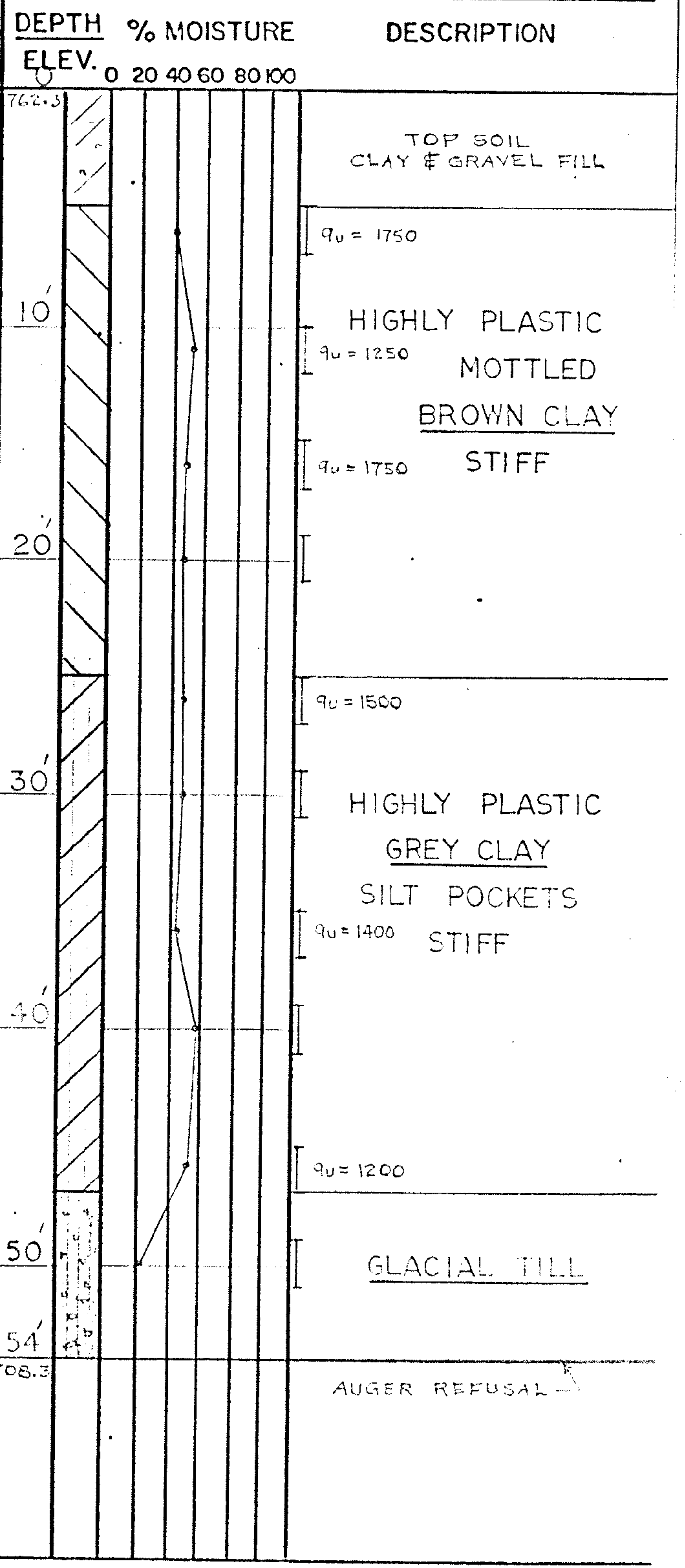
LOGGED BY — DATE SEPT. 24/65

REFERENCE FROM REPORT PREPARED FOR WAISMAN ROSS BLANKSTEIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 22

COORDINATES F-4

LOCATION ARTS-ISBISTER -BUILDING



TOP SOIL CLAY & GRAVEL FILL

HIGHLY PLASTIC MOTTLED BROWN CLAY STIFF

HIGHLY PLASTIC GREY CLAY SILT POCKETS STIFF

GLACIAL TILL

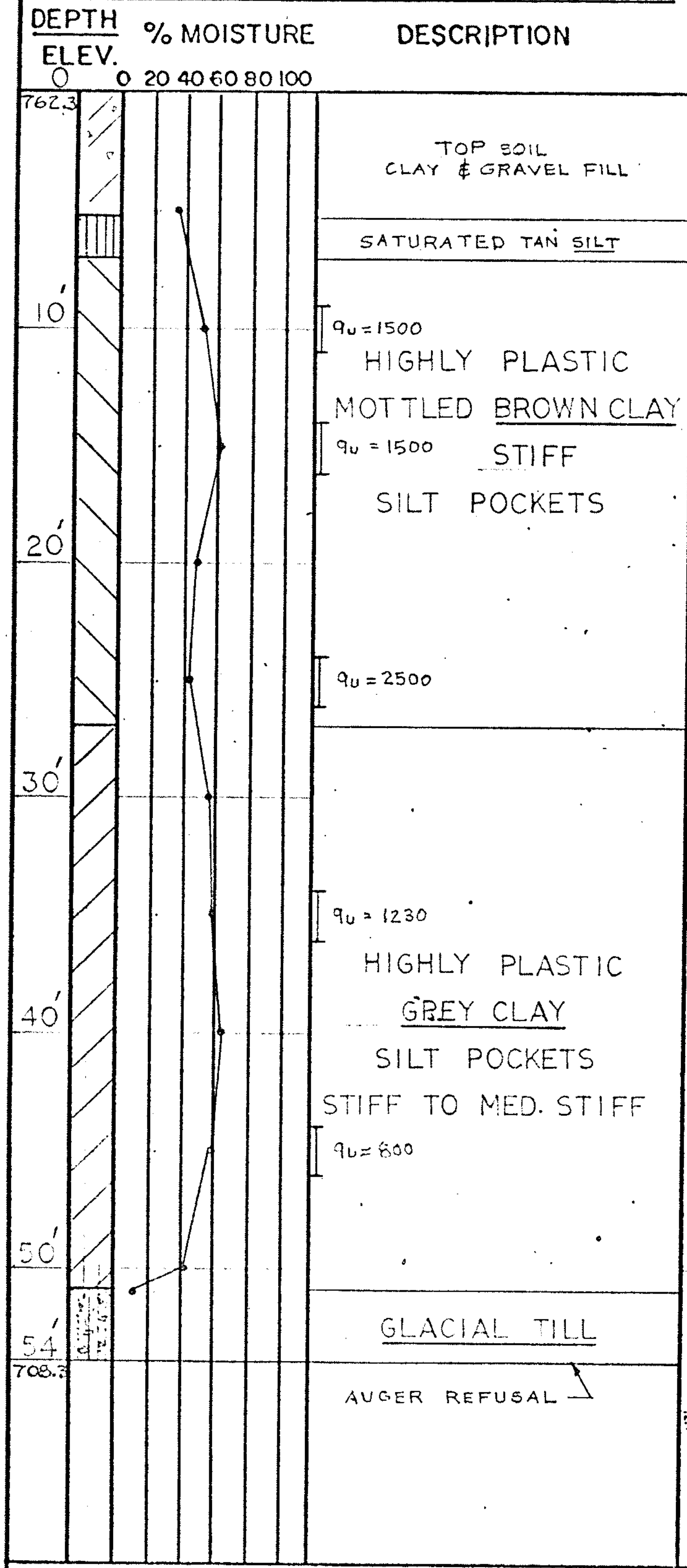
AUGER REFUSAL

HOLE ADVANCED BY 16" POWER AUGER

LOGGED BY — DATE SEPT. 24/65

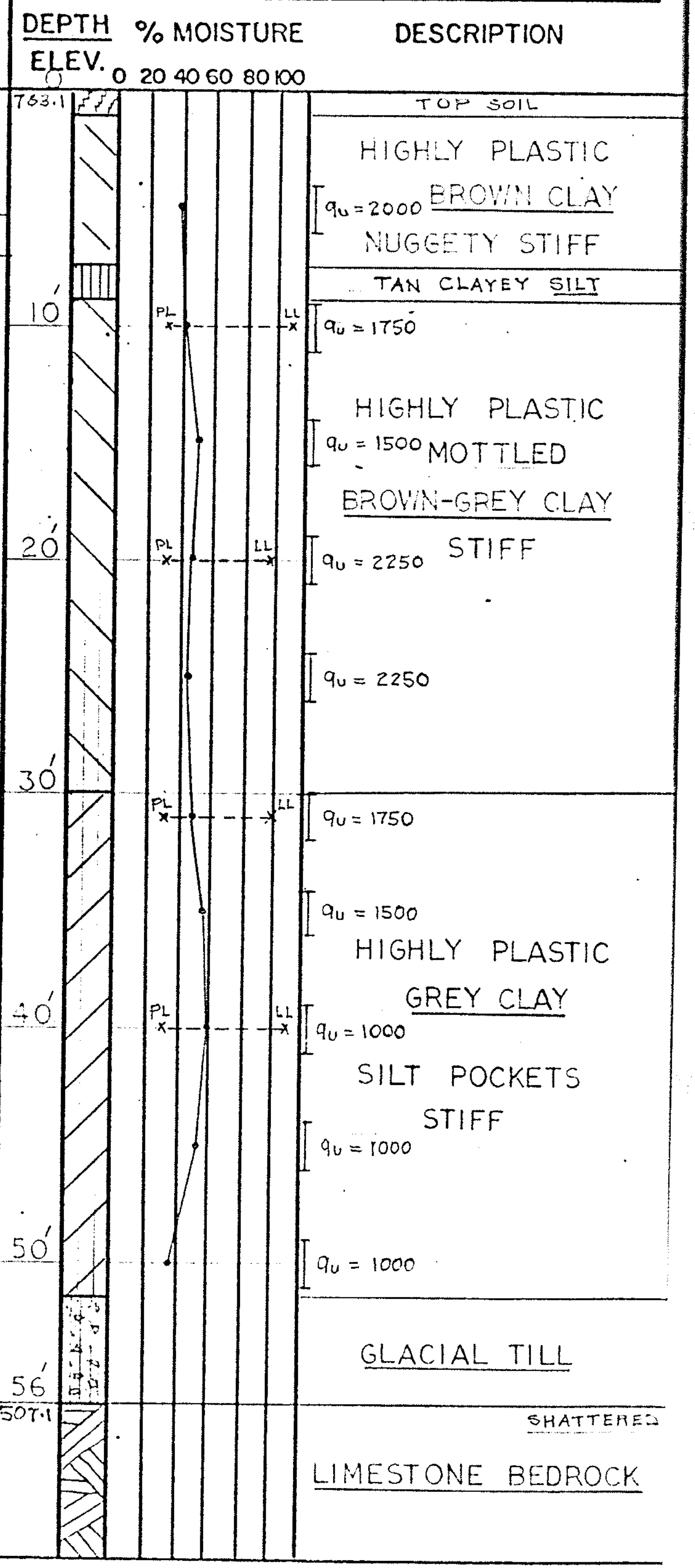
REFERENCE FROM REPORT PREPARED FOR WAISMAN ROSS BLANKSTEIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 23
COORDINATES F-4
LOCATION ARTS-ISBISTER -BUILDING



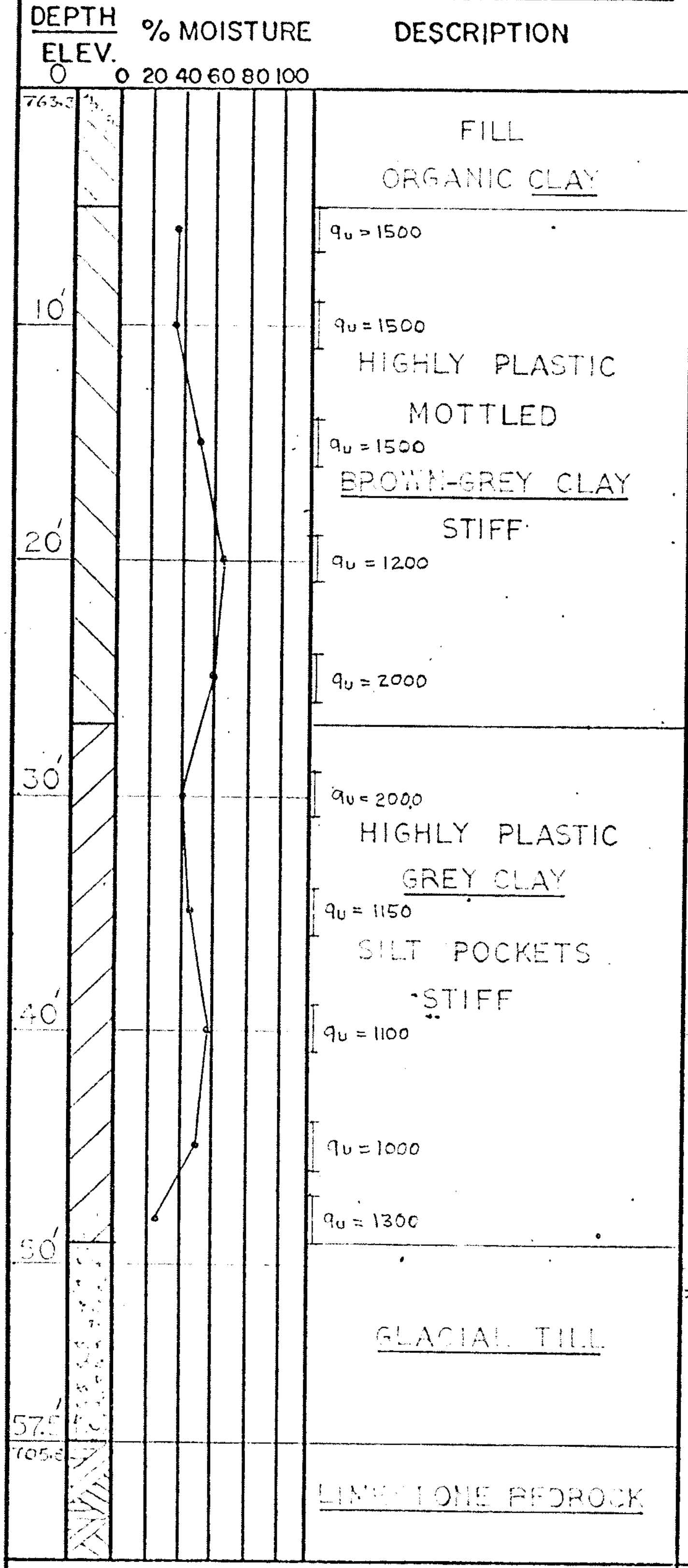
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY —— DATE SEPT. 24/65
REFERENCE FROM REPORT PREPARED FOR WAISMAN ROSS BLANKSTEIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 24
COORDINATES F-4
LOCATION ARTS-ISBISTER -BUILDING



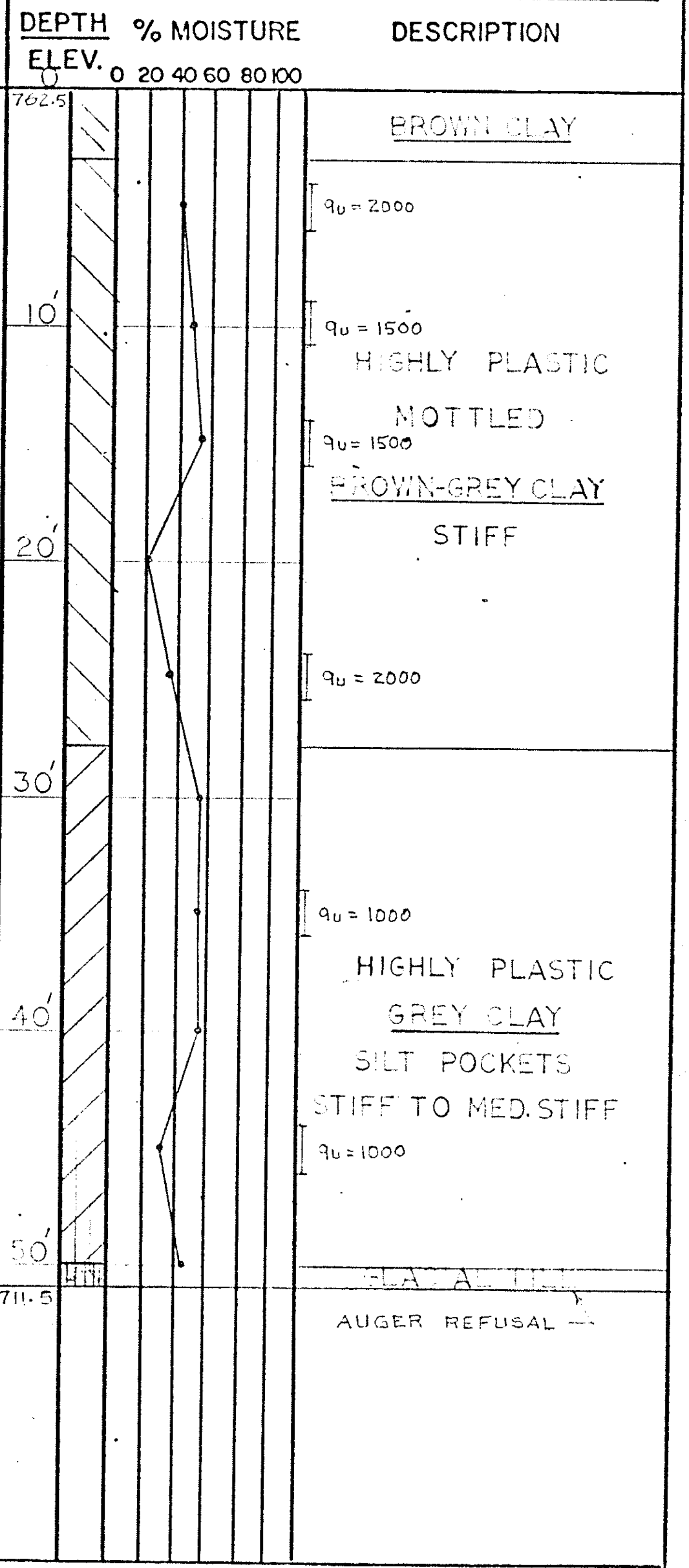
HOLE ADVANCED BY FLIGHT AUGER
LOGGED BY —— DATE SEPT. 29/65
REFERENCE FROM REPORT PREPARED FOR WAISMAN ROSS BLANKSTEIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 25
COORDINATES D-4
LOCATION ENGINEERING - BUILDING



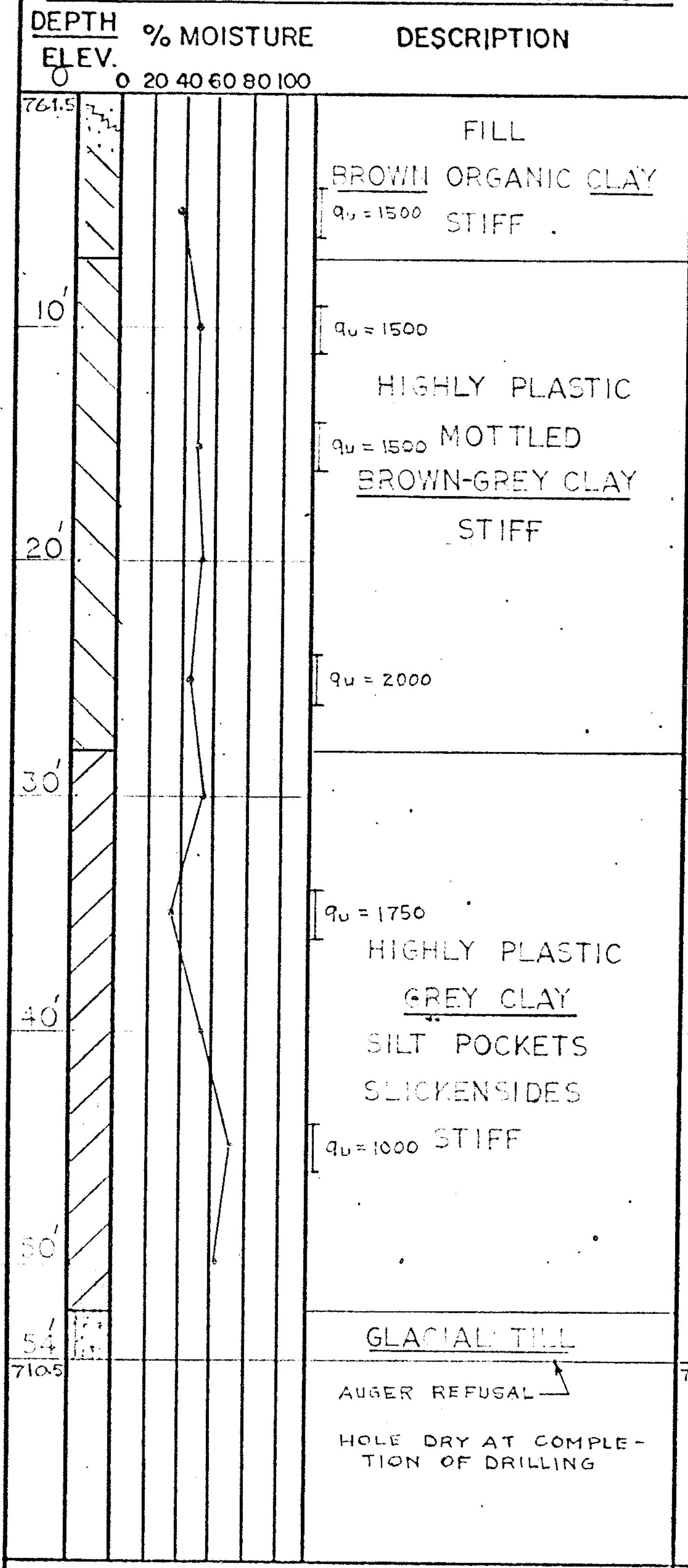
HOLE ADVANCED BY DIAMOND DRILL
LOGGED BY — DATE NOV. 26/65
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 26
COORDINATES D-4
LOCATION ENGINEERING - BUILDING



HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY — DATE NOV. 19/65
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 27
COORDINATES D-4
LOCATION ENGINEERING -BUILDING



FILL

BROWN ORGANIC CLAY
STIFF

HIGHLY PLASTIC
MOTTLED
BROWN-GREY CLAY
STIFF

HIGHLY PLASTIC
GREY CLAY
SILT POCKETS
SLICKENSIDES
STIFF

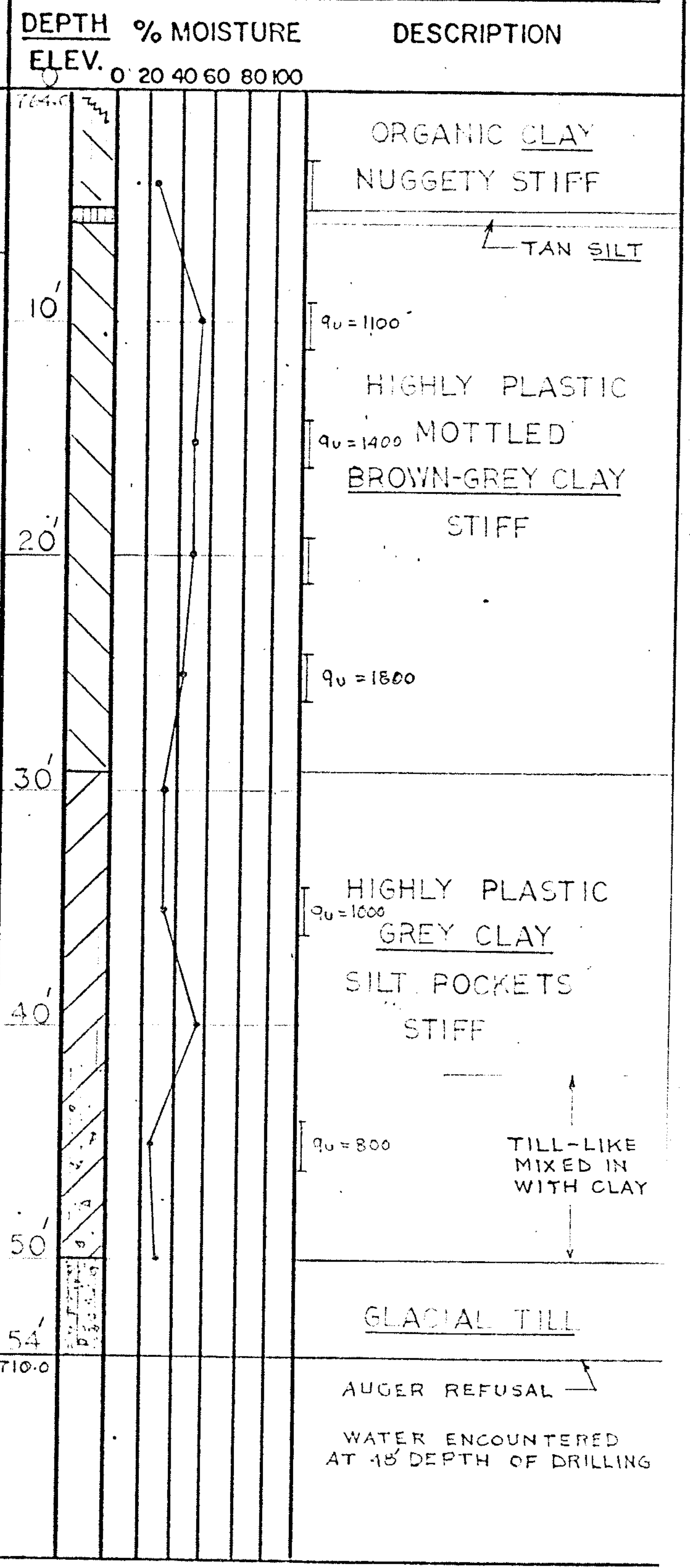
GLACIAL TILL

AUGER REFUSAL

HOLE DRY AT COMPLETION OF DRILLING

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY — DATE NOV. 19/65
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 28
COORDINATES D-4
LOCATION ENGINEERING -BUILDING



ORGANIC CLAY
NUGGETY STIFF

TAN SILT

HIGHLY PLASTIC
MOTTLED
BROWN-GREY CLAY
STIFF

HIGHLY PLASTIC
GREY CLAY
SILT POCKETS
STIFF

TILL-LIKE MIXED IN WITH CLAY

GLACIAL TILL

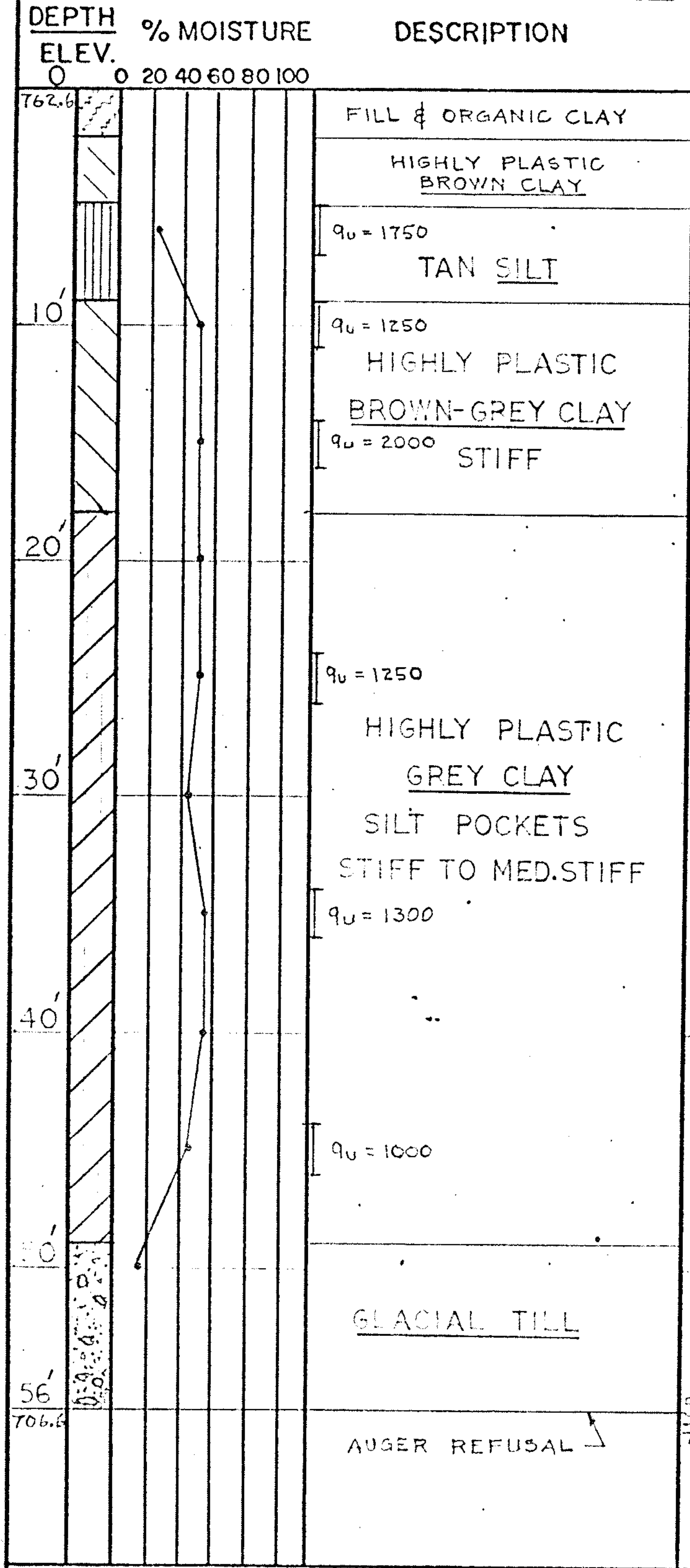
AUGER REFUSAL

WATER ENCOUNTERED AT 48' DEPTH OF DRILLING

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY — DATE NOV. 19/65
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 29
COORDINATES E-3
LOCATION BULLER BUILDING

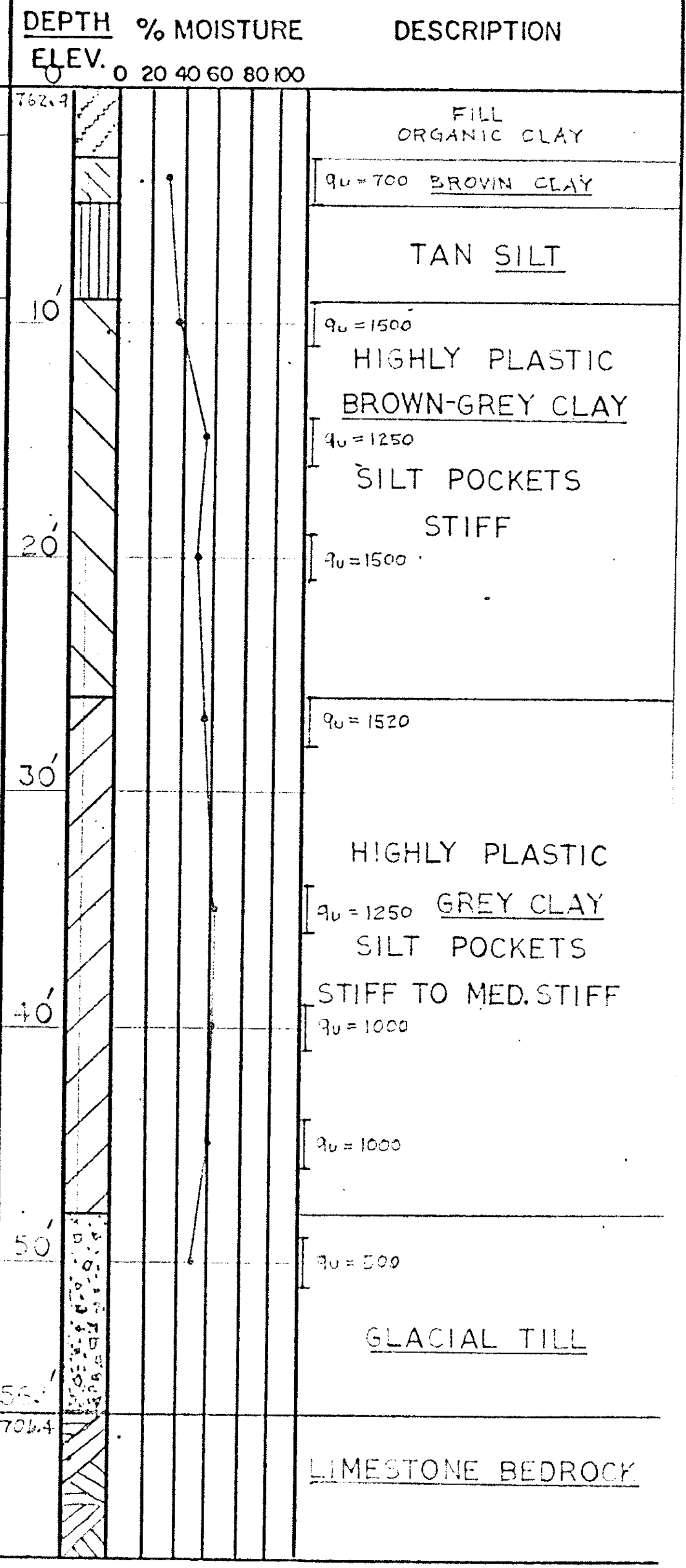
DEPTH ELEV. | % MOISTURE | DESCRIPTION



HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY —
DATE APR. 29/66
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

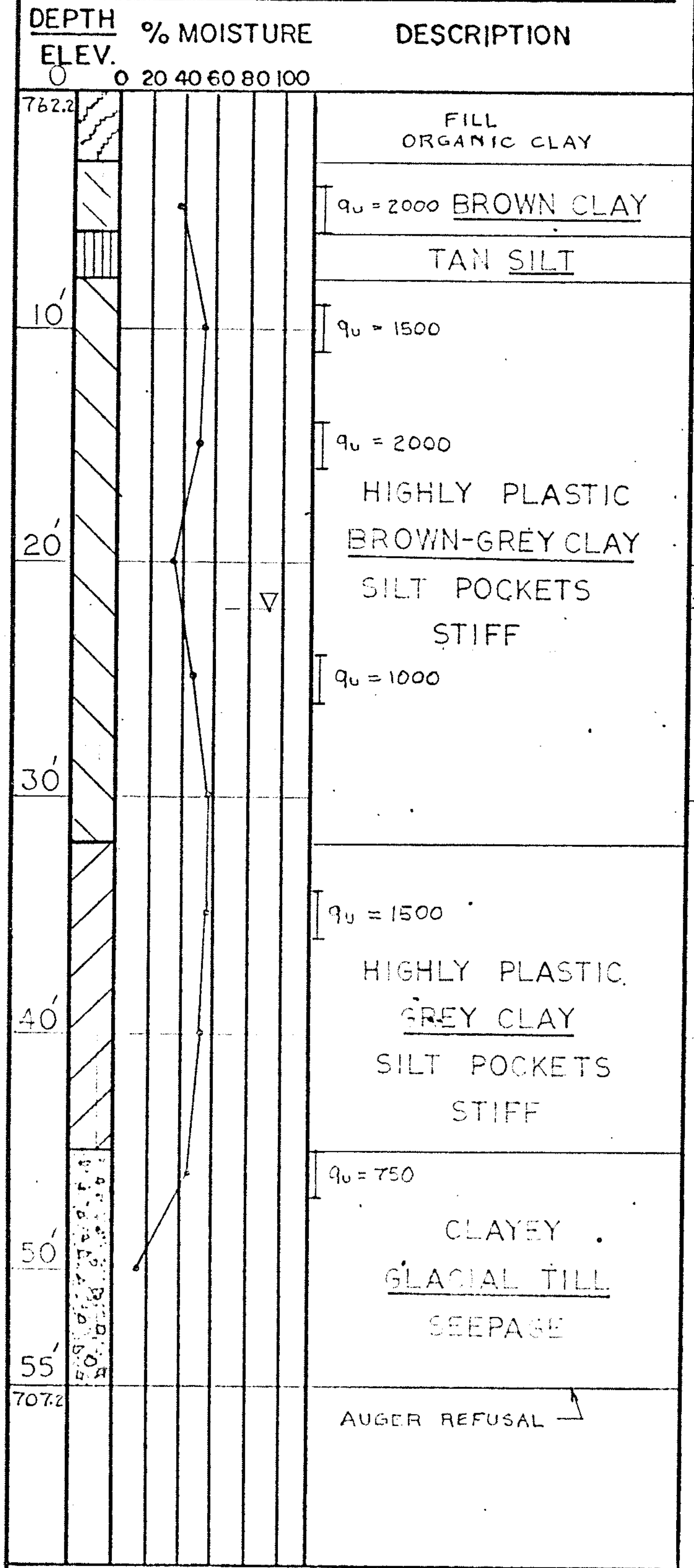
LOG OF TEST HOLE NO. 30
COORDINATES E-3
LOCATION BULLER BUILDING

DEPTH ELEV. | % MOISTURE | DESCRIPTION



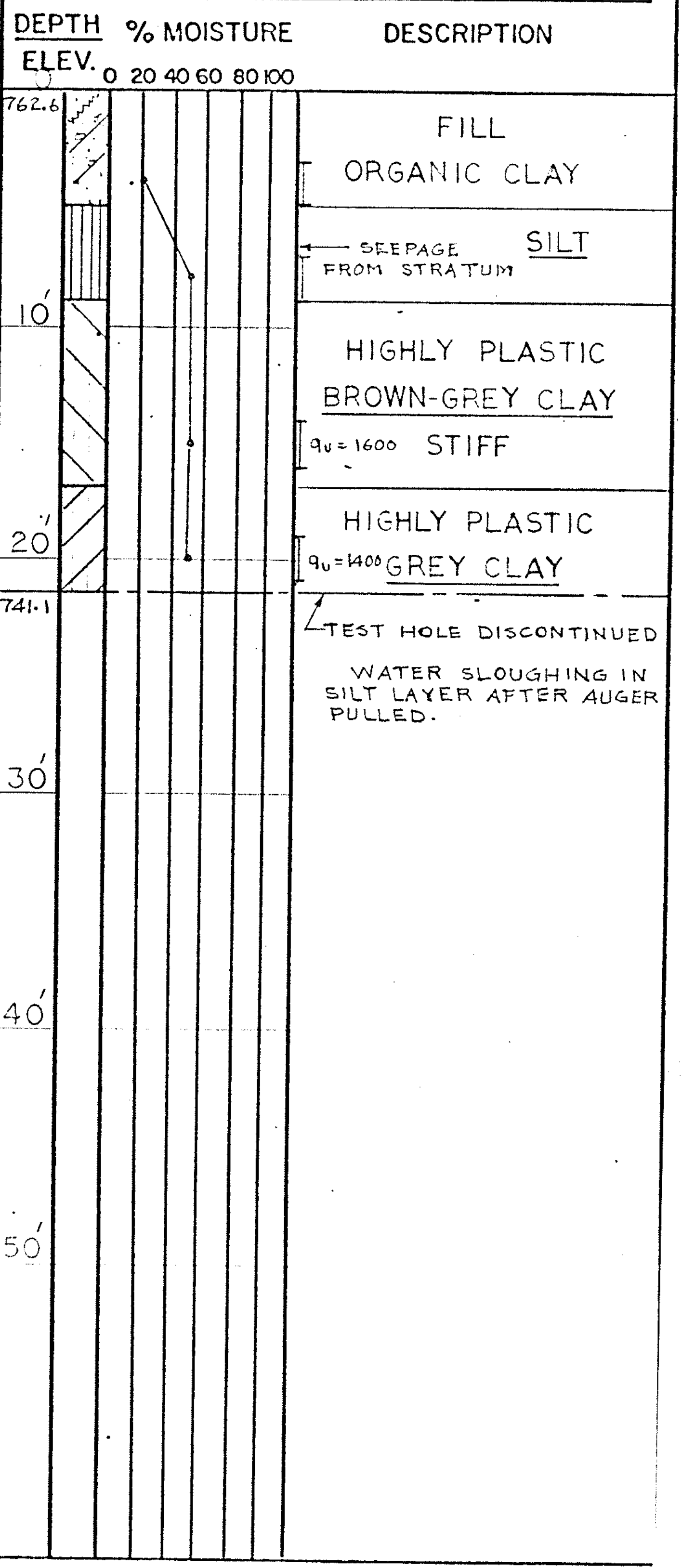
HOLE ADVANCED BY DIAMOND DRILL
LOGGED BY —
DATE MAY [illegible]/66
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 31
COORDINATES E-3
LOCATION BULLER BUILDING



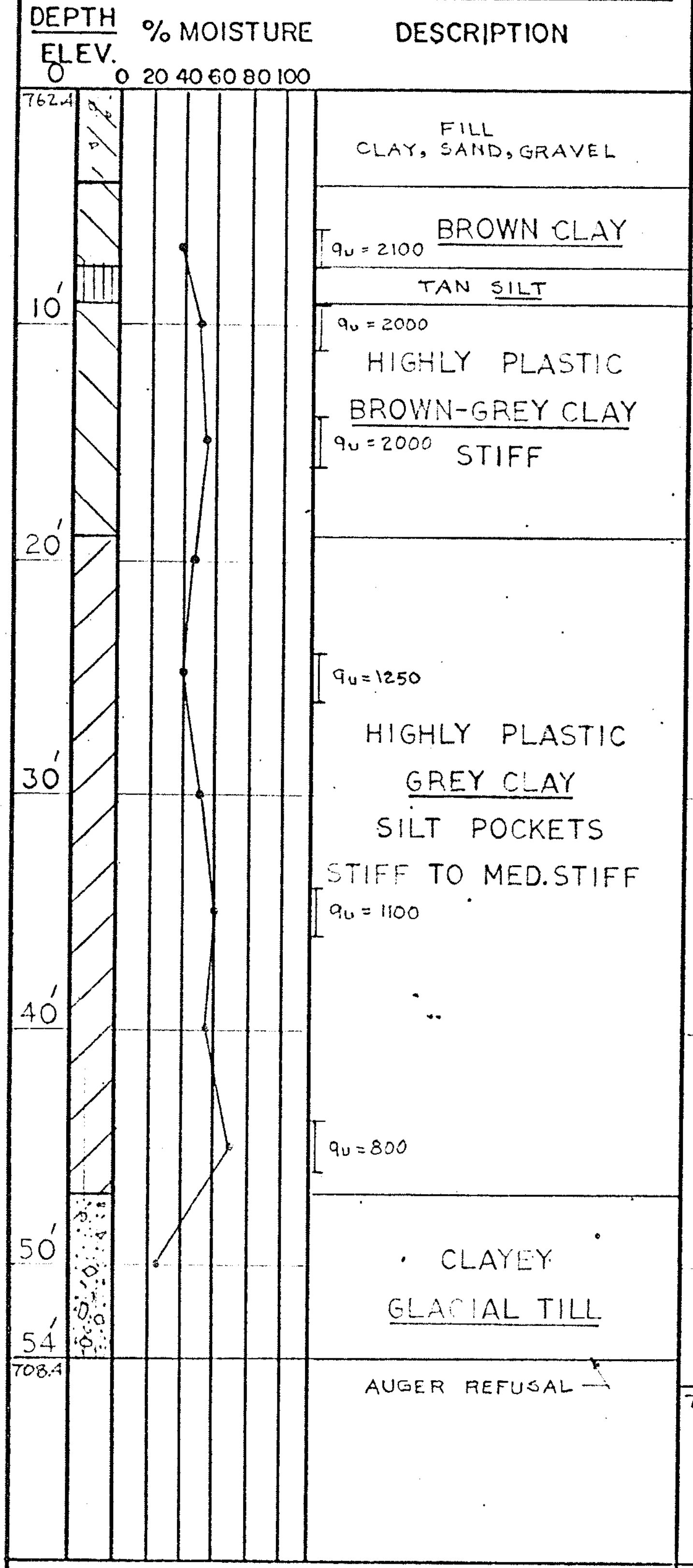
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY — DATE [illegible]
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 32
COORDINATES E-3
LOCATION BULLER BUILDING



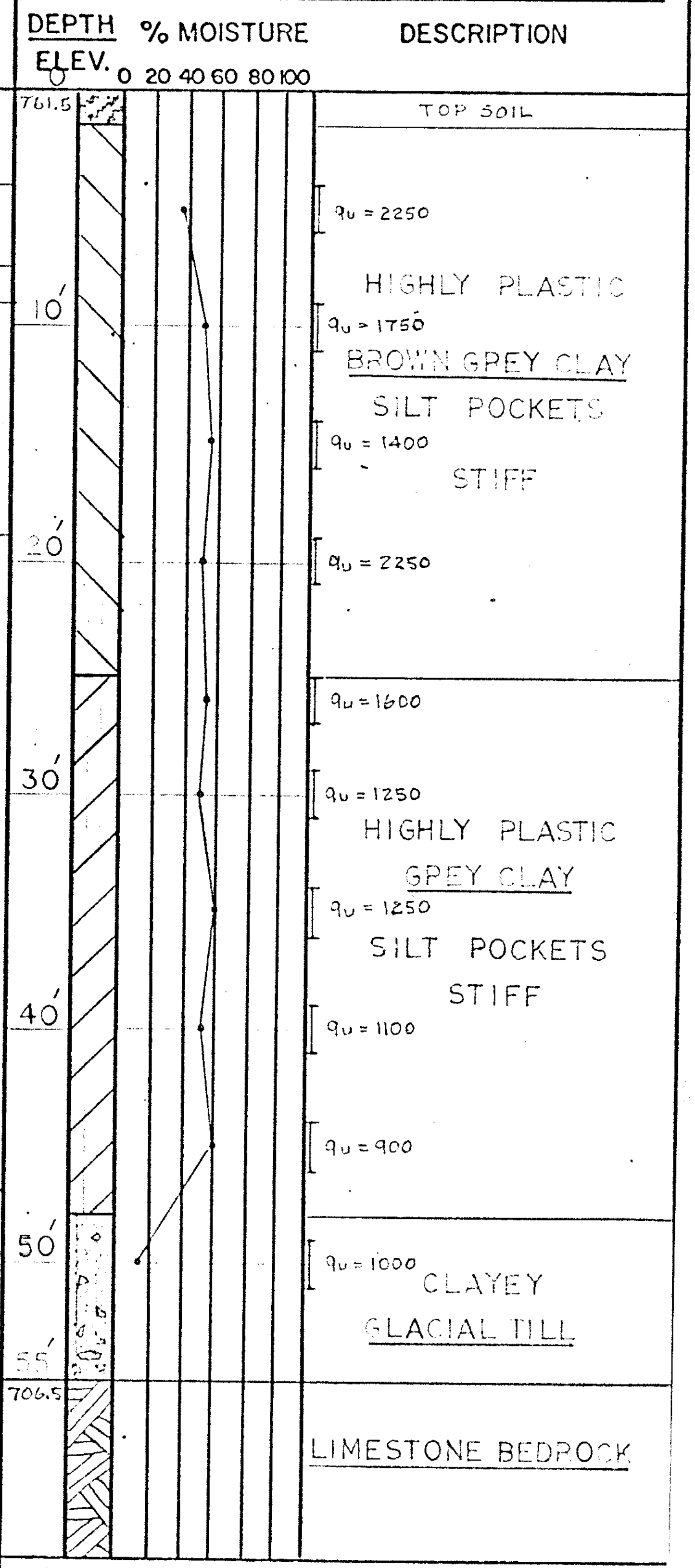
HOLE ADVANCED BY [illegible]" HOLLOW STEM
LOGGED BY — DATE [illegible]
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 33
COORDINATES E-3
LOCATION DUFF ROBIN -BUILDING



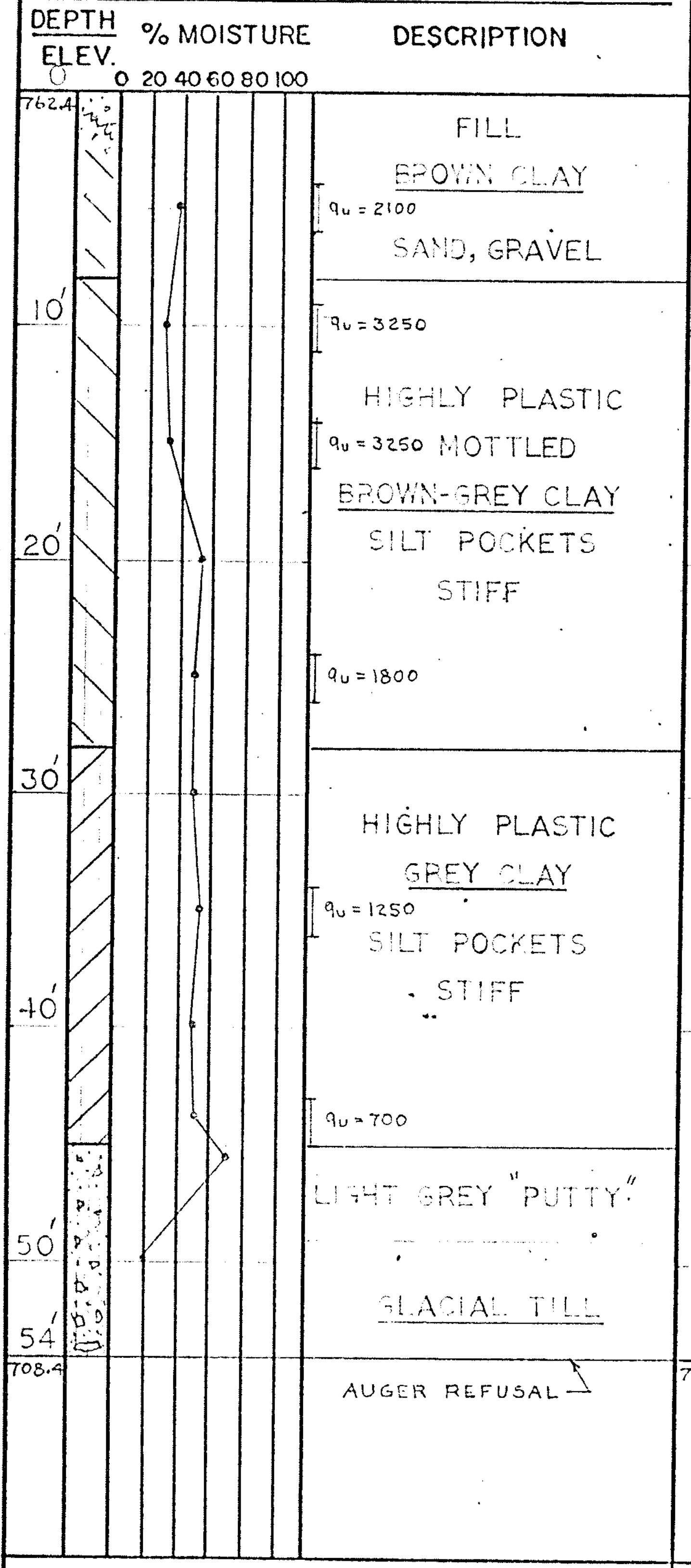
HOLE ADVANCED BY [illegible]" POWER AUGER
LOGGED BY —
DATE APR. [illegible]/[illegible]
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 34
COORDINATES F-3
LOCATION DUFF ROBIN -BUILDING



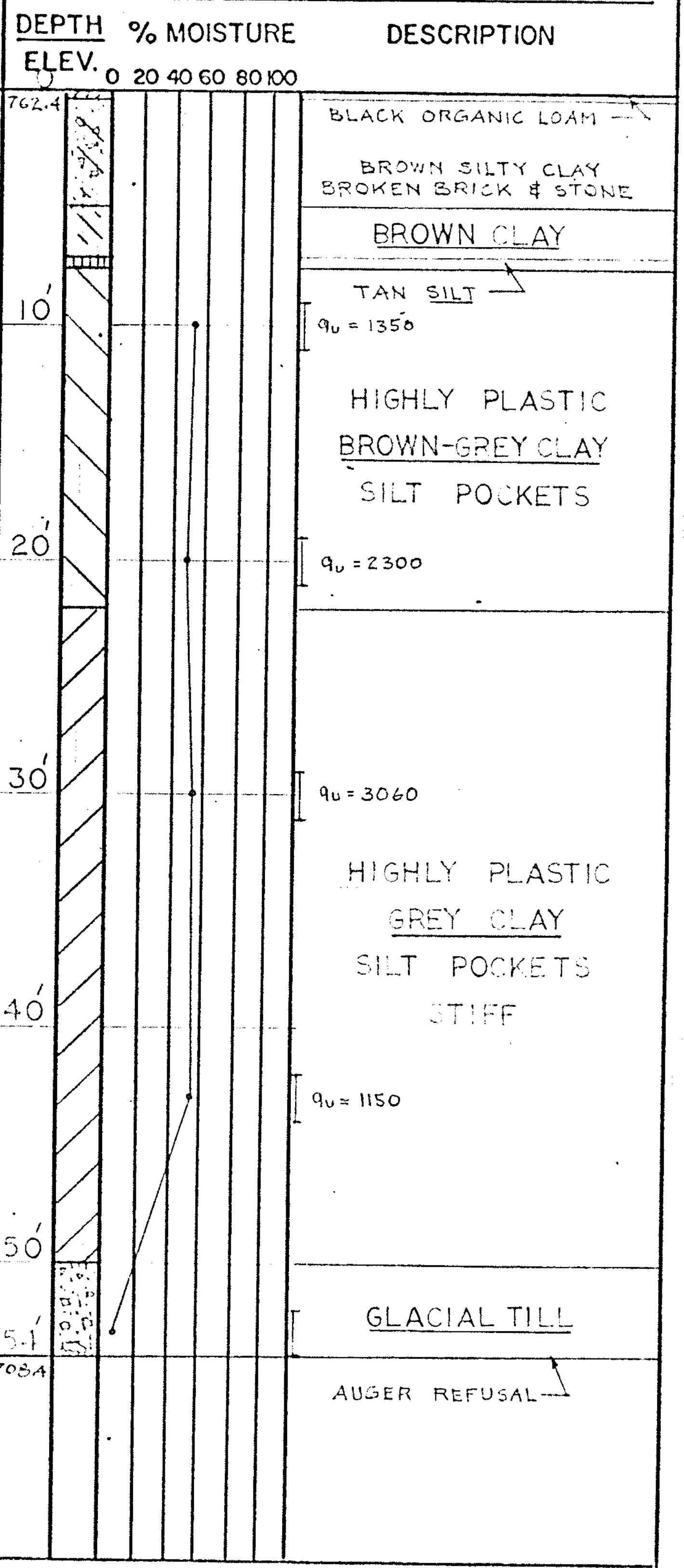
HOLE ADVANCED BY DIAMOND DRILL
LOGGED BY —
DATE MAY 19/66
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 35
COORDINATES F-3
LOCATION DUFF ROBIN - BUILDING



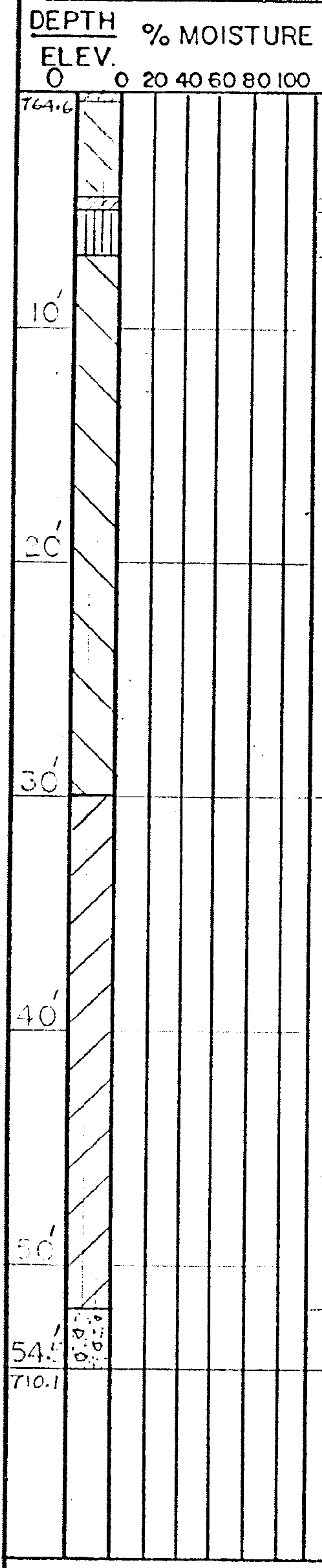
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY —— DATE APR. 29/66
REFERENCE FROM REPORT PREPARED FOR GREEN BLANKSTEIN RUSSELL BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 36
COORDINATES D-2
LOCATION ST. PAULS COLLEGE.



HOLE ADVANCED BY 16" FLIGHT AUGER
LOGGED BY A.C.M. DATE FEB. 4/[illegible]
REFERENCE FROM REPORT PREPARED FOR [illegible] ABOFFRY LUSSIER SIGURDSON BY M. BLOCK & ASSOCIATES

LOG OF TEST HOLE NO. 37
COORDINATES D-2
LOCATION ST. PAUL COLLEGE



DESCRIPTION

BLACK ORGANIC LOAM

FILL & BROWN CLAY

BLACK SILTY CLAY

TAN SILT

HIGHLY PLASTIC
BROWN-GREY CLAY
SILT POCKETS
STIFF

HIGHLY PLASTIC
GREY CLAY
SILT POCKETS
STIFF

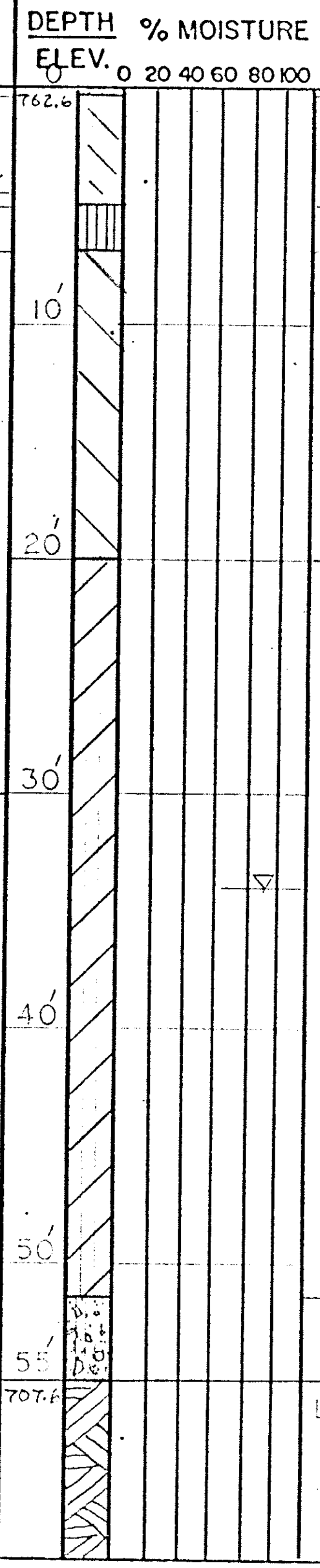
GLACIAL TILL

AUGER REFUSAL

NOTE. NO WATER — SEEPAGE

HOLE ADVANCED BY 15" FLIGHT AUGER
LOGGED BY A.E.M. DATE FEB.4/71
REFERENCE FROM REPORT PREPARED FOR GABOURY LUSSIER SIGURDSON BY M.BLOCK & ASSOCIATES

LOG OF TEST HOLE NO. 38
COORDINATES D-2
LOCATION ST. PAUL COLLEGE



DESCRIPTION

BLACK ORGANIC LOAM

BROWN CLAY

TAN SILT

HIGHLY PLASTIC
BROWN-GREY CLAY
SILT POCKETS
STIFF

HIGHLY PLASTIC
GREY CLAY

WATER LEVEL
AFTER DRILLING
WAS COMPLETED

SILT POCKETS
STIFF

NOTE. GROUND WATER PUMPED FOR 1 HR. LOWERED THE WATER LEVEL FROM 34' TO 49'-6" DEPTH

GLACIAL TILL

LIMESTONE BEDROCK
SHATTERED
WATER SEEPAGE

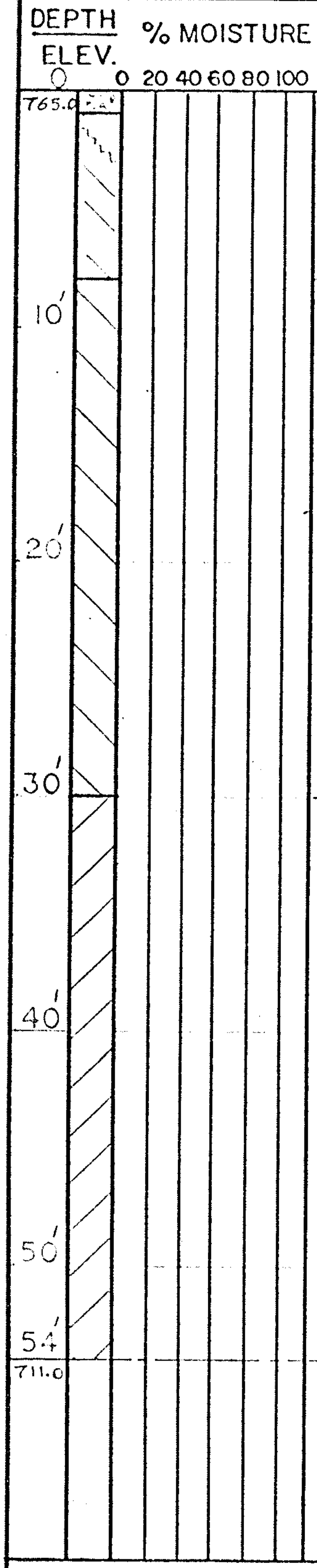
HOLE ADVANCED BY 30" FLIGHT AUGER
LOGGED BY A.E.M. DATE FEB.5/71
REFERENCE FROM REPORT PREPARED FOR GABOURY LUSSIER SIGURDSON BY M.BLOCK & ASSOCIATES

LOG OF TEST HOLE NO. 39

COORDINATES D-5

LOCATION CHILLED WATER -PLANT

DEPTH ELEV. | % MOISTURE | DESCRIPTION



GRANULAR FILL

HIGHLY PLASTIC
BROWN CLAY

HIGLY PLASTIC
MOTTLED
BROWN GREY CLAY
SILT POCKETS
STIFF

HIGHLY PLASTIC
GREY CLAY
SILT POCKETS
STIFF

HOLE TERMINATED

NOTES

1. HOLE TERMINATED @ 54' IN GREY CLAY

2. NO WATER SEEPAGE

HOLE ADVANCED BY 16" POWER AUGER

LOGGED BY J. A. DATE JAN. 25/71

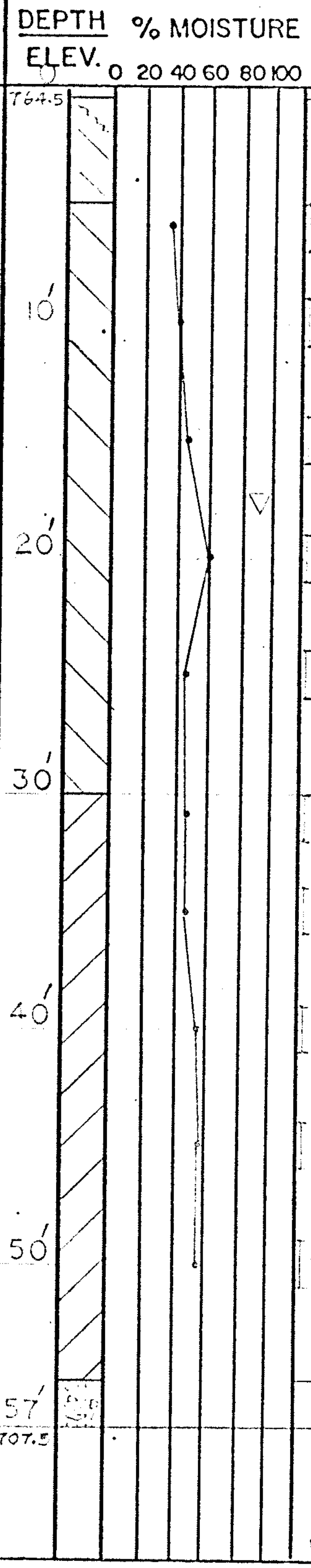
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 40

COORDINATES D-5

LOCATION AGRICULTURE UTILITIES TUNNEL

DEPTH ELEV. | % MOISTURE | DESCRIPTION



GRANULAR FILL

HIGHLY PLASTIC
BROWN CLAY

$q_u = 2000$

$q_u = 1750$

HIGHLY PLASTIC
MOTTLED

$q_u = 1500$

BROWN-GREY CLAY
SILT POCKETS

$q_u = 1300$

STIFF

$q_u = 1800$

$q_u = 2200$

$q_u = 1000$

HIGHLY PLASTIC
GREY CLAY

$q_u = 1500$

SILT POCKETS
STIFF

$q_u = 100c$

$q_u = 850$

GLACIAL TILL

AUGER REFUSAL

NOTE

WATER ROSE TO 18 FT. DEPTH IN MINUTES.

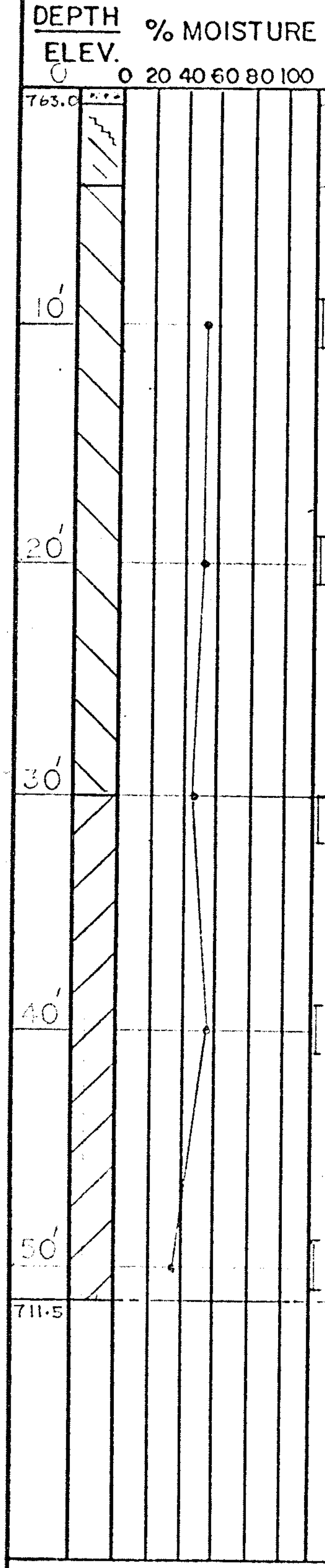
HOLE ADVANCED BY 15" POWER AUGER

LOGGED BY J. A. DATE JULY 6/70

REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 41
COORDINATES D-5
LOCATION AGRICULTURE UTILITIES TUNNEL

DEPTH ELEV. | % MOISTURE | DESCRIPTION



GRANULAR FILL

HIGHLY PLASTIC BROWN CLAY STIFF

q_u = 1250

HIGHLY PLASTIC
BROWN-GREY CLAY
MOTTLED

q_u = 1850

SILT POCKETS
STIFF

q_u = 2400

HIGHLY PLASTIC
GREY CLAY

q_u = 1200

SILT POCKETS
STIFF

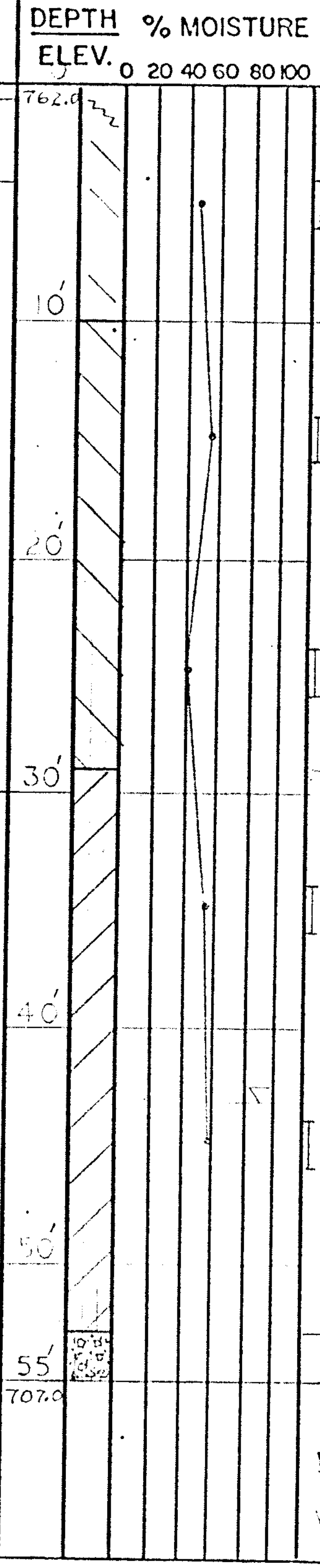
q_u = 950

NOTES.
1. HOLE TERMINATED IN CLAY - NO REFUSAL.
2. NO WATER SEEPAGE TO 51.5 FT DEPTH.

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J.A. DATE JULY 6/70
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 42
COORDINATES D-5
LOCATION AGRICULTURE UTILITIES TUNNEL

DEPTH ELEV. | % MOISTURE | DESCRIPTION



FILL
HIGHLY PLASTIC

q_u = 1000

BROWN CLAY
STIFF

q_u = 1650

HIGHLY PLASTIC
BROWN-GREY CLAY
SILT POCKETS
STIFF

q_u = 2600

q_u = 1100

HIGHLY PLASTIC
GREY CLAY
SILT POCKETS
STIFF

q_u = 1000

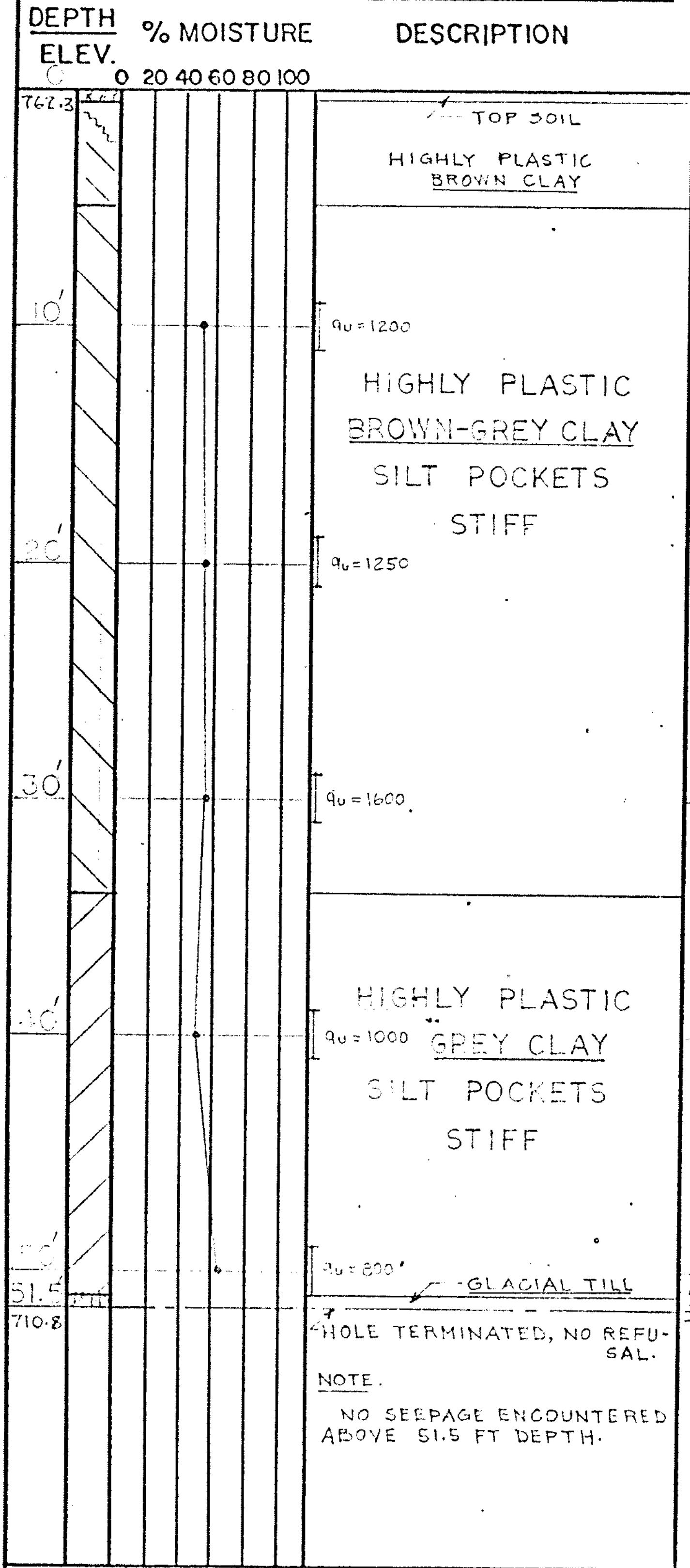
GLACIAL TILL

AUGER REFUSAL

NOTE.
ENCOUNTERED HEAVY WATER FLOW AT 53' DEPTH.

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J.A. DATE JULY 6/70
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 43
COORDINATES D-5
LOCATION AGRICULTURE UTILITIES TUNNEL

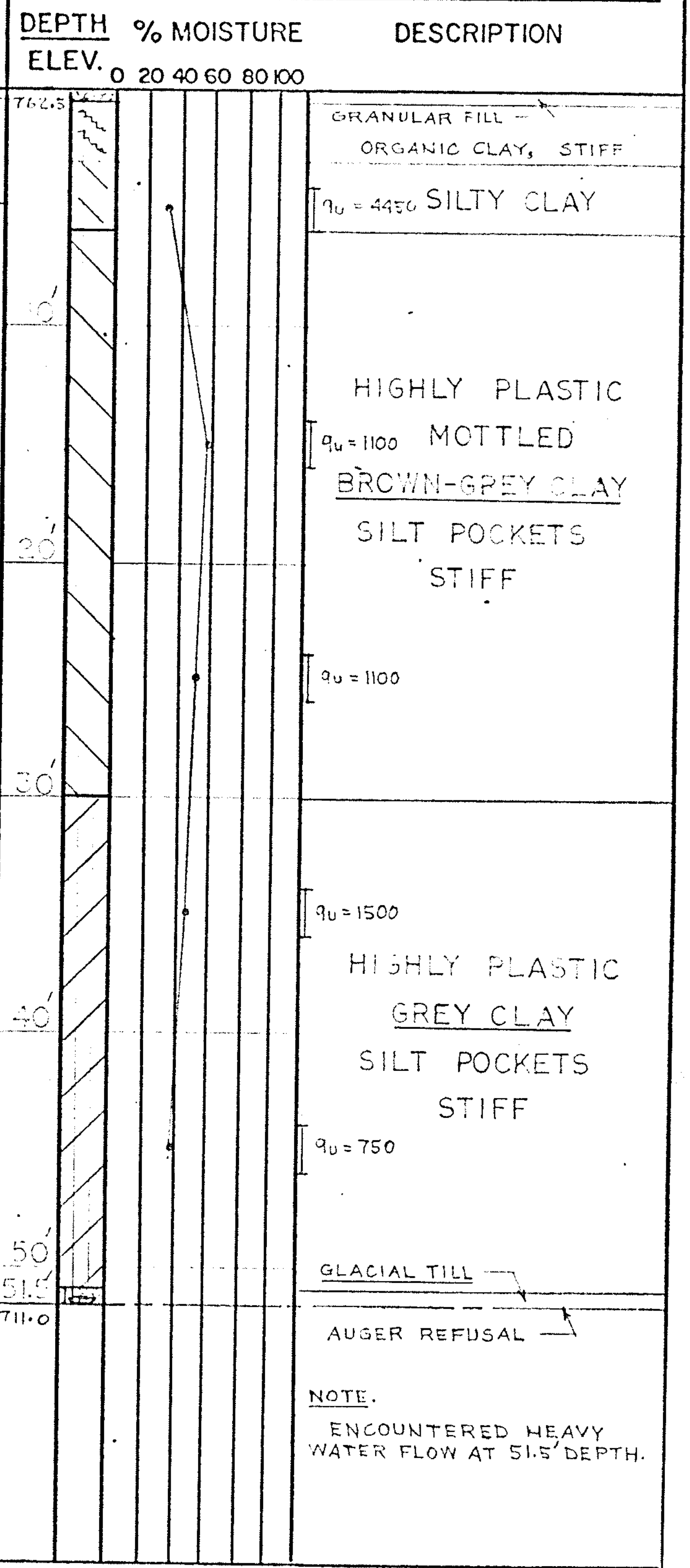


NOTE.

NO SEEPAGE ENCOUNTERED ABOVE 51.5 FT DEPTH.

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J.A. DATE JULY 7/70
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 44
COORDINATES C-5
LOCATION AGRICULTURE UTILITIES TUNNEL

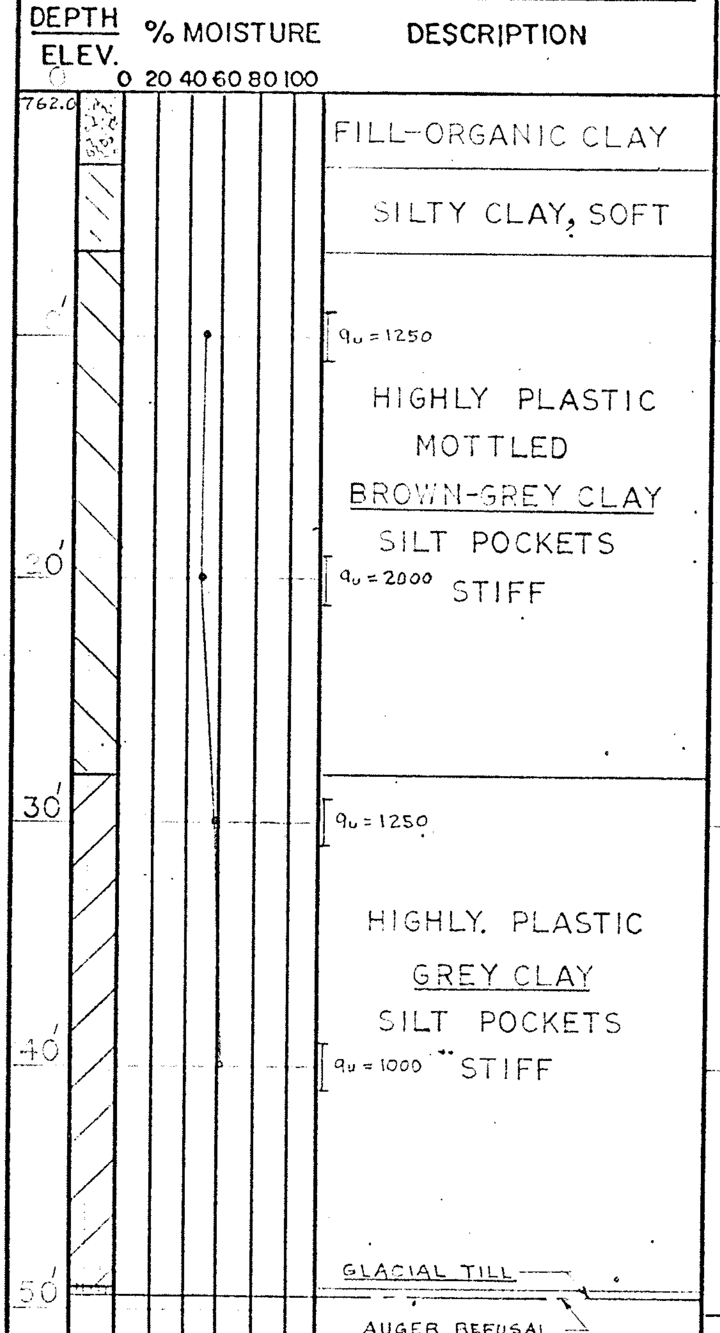


NOTE.

ENCOUNTERED HEAVY WATER FLOW AT 51.5' DEPTH.

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J. A. DATE JULY 7/70
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 45
COORDINATES C-5
LOCATION AGRICULTURE UTILITIES TUNNEL

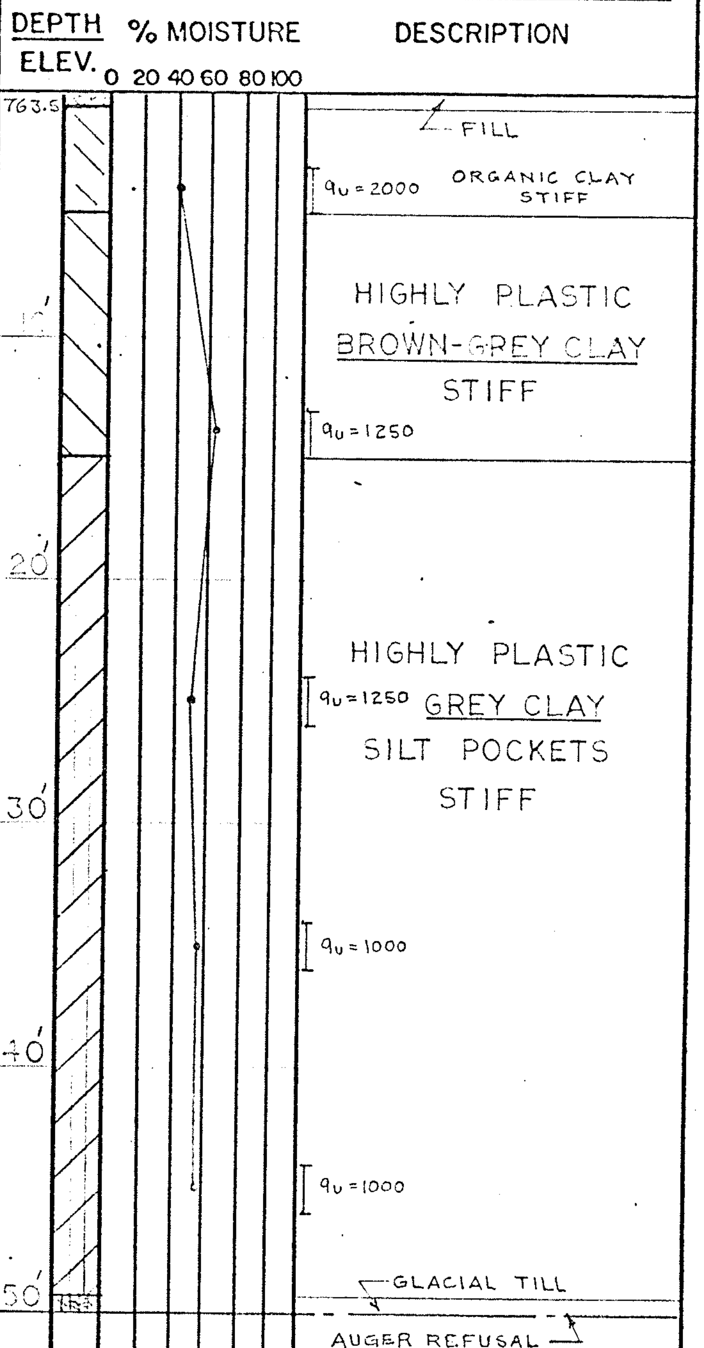


NOTES.

1. AUGER REFUSAL AT 49.3' DEPTH ON BOULDER?
2. NO SEEPAGE ENCOUNTERED ABOVE 49.3' DEPTH.

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J. A. DATE JULY 7/70
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 46
COORDINATES C-5
LOCATION AGRICULTURE UTILITIES TUNNEL



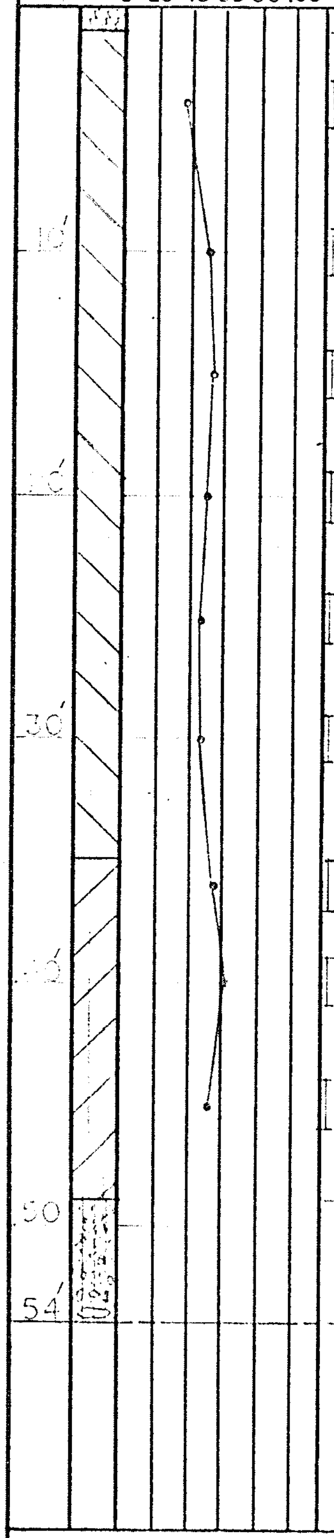
NOTES.

1. AUGER REFUSAL AT 50 FT DEPTH ASSUMED ON BOULDER.
2. ENCOUNTERED HEAVY WATER FLOW @ 50' DEPTH.

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J. A. DATE JULY 7/70
REFERENCE FROM REPORT PREPARED FOR REID CROWTHER & PARTNERS BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 47
COORDINATES E-3
LOCATION N.E. MULTI-PURPOSE BUILDING

DEPTH ELEV. | % MOISTURE 0 20 40 60 80 100 | DESCRIPTION



TOP SOIL

$q_u = 500$

$q_u = 1000$

HIGHLY PLASTIC
$q_u = 1100$ MOTTLED
BROWN-GREY CLAY
SILT POCKETS
$q_u = 1250$ STIFF

$q_u = 1200$

$q_u = 1500$

$q_u = 800$

HIGHLY PLASTIC
$q_u = 800$ GREY CLAY
SILT POCKETS
$q_u = 800$ STIFF

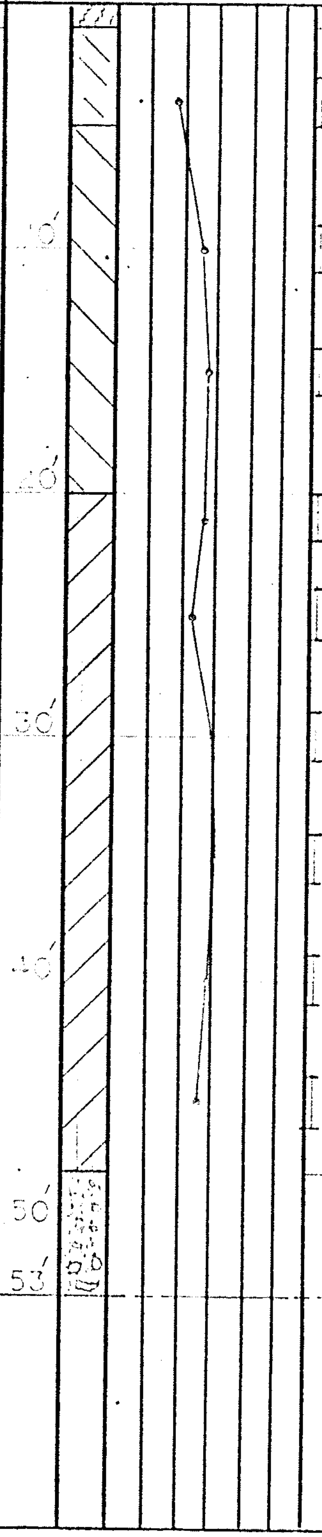
CLAYEY
GLACIAL TILL

AUGER REFUSAL

HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY L.A. DATE DEC.30/70
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER PARKIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 48
COORDINATES E-3
LOCATION N.E. MULTI-PURPOSE BUILDING

DEPTH ELEV. | % MOISTURE 0 20 40 60 80 100 | DESCRIPTION



TOP SOIL

$q_u = 650$ BROWN CLAY

HIGHLY PLASTIC
$q_u = 1200$ MOTTLED
BROWN-GREY CLAY
SILT POCKETS
$q_u = 1200$ STIFF

$q_u = 1250$

$q_u = 1500$
HIGHLY PLASTIC
GREY CLAY
$q_u = 1000$
SILT POCKETS
STIFF

$q_u = 1000$

$q_u = 750$

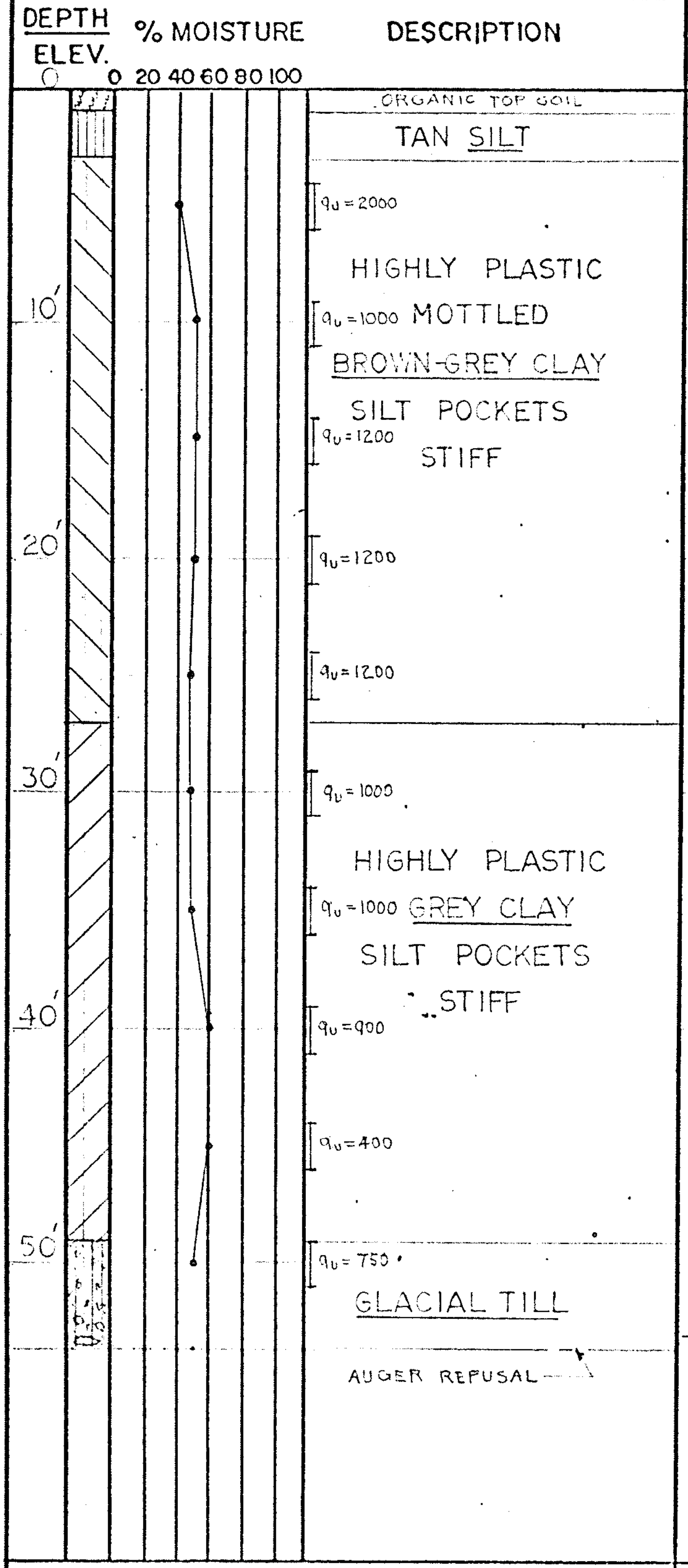
$q_u = 550$

CLAYEY
GLACIAL TILL

AUGER REFUSAL

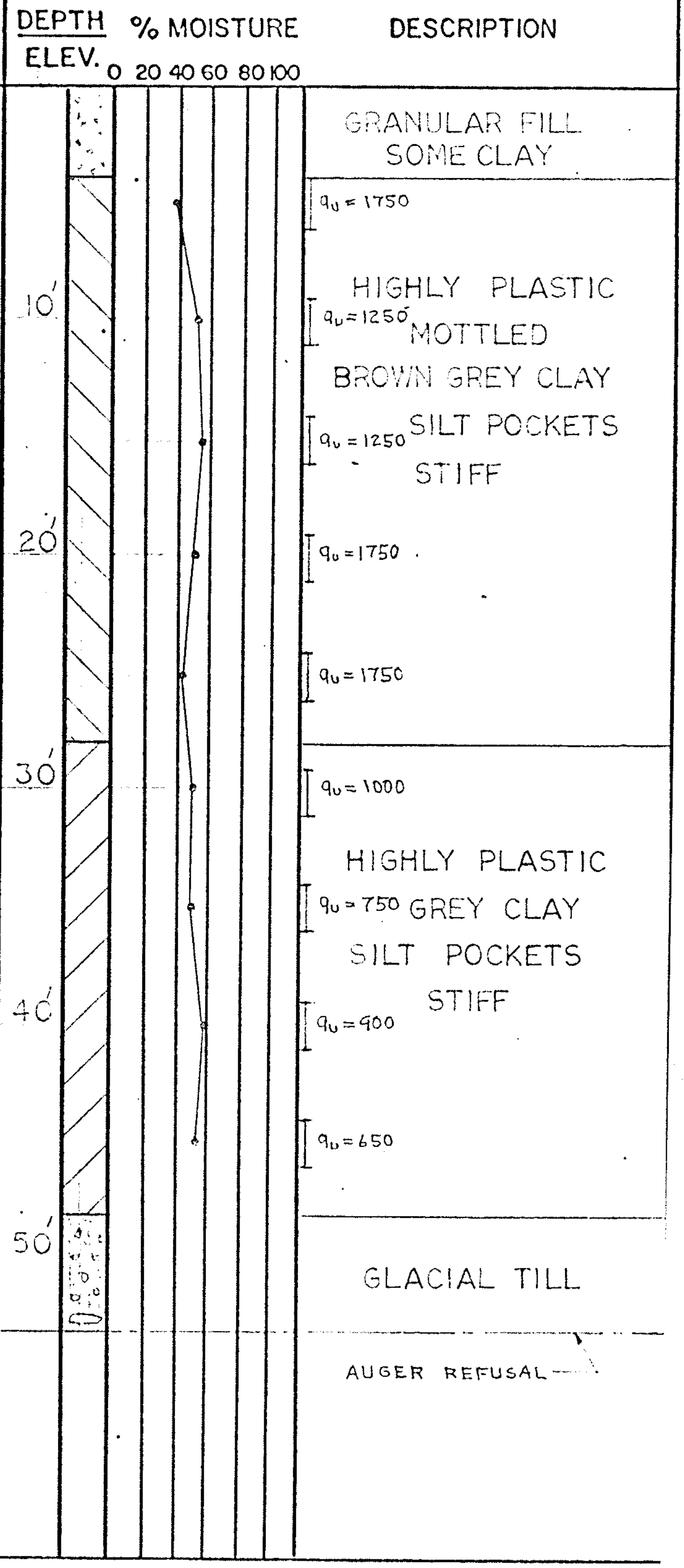
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY L.A. DATE DEC.30/70
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER PARKIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 49
COORDINATES E-3
LOCATION N.E. MULTI-PURPOSE BUILDING



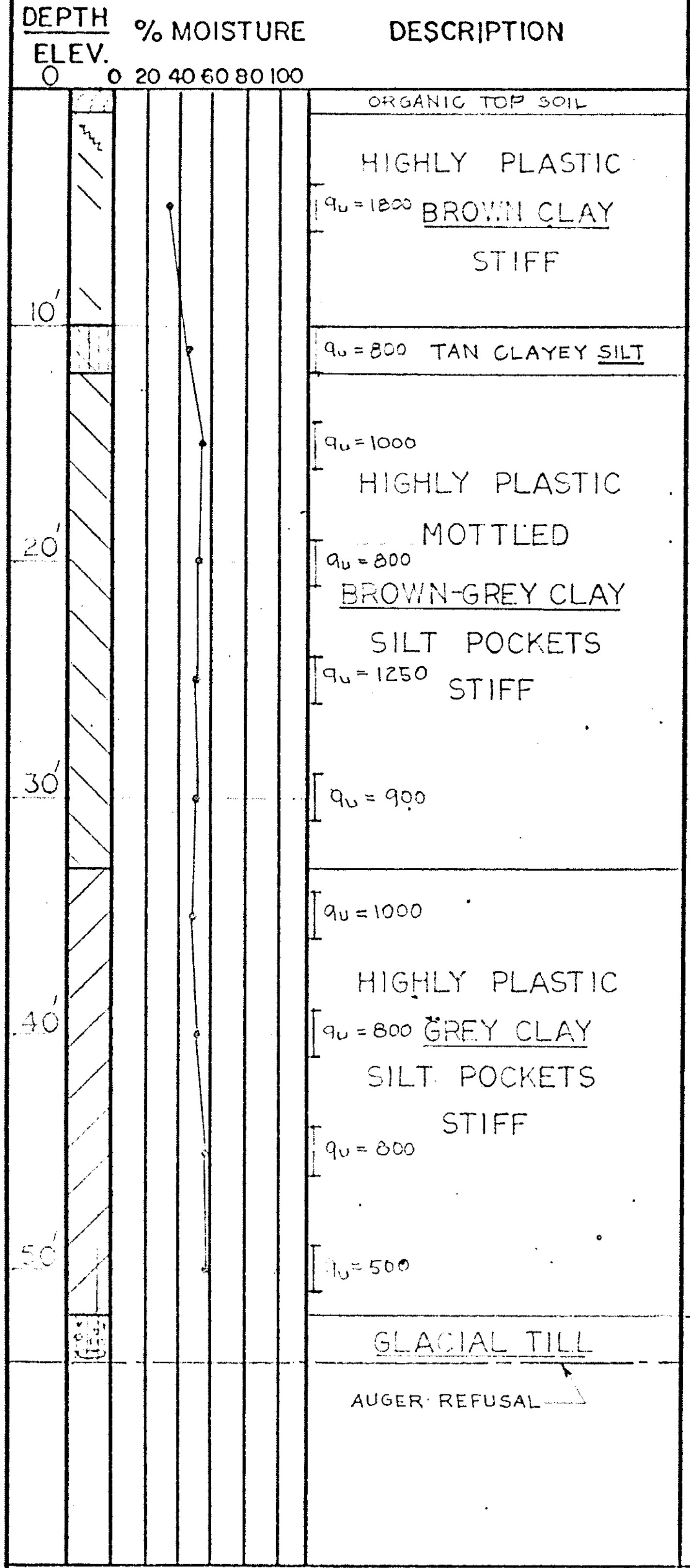
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J. A. DATE DEC. 30/70
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER PARKIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 50
COORDINATES E-3
LOCATION N.E. MULTI-PURPOSE BUILDING



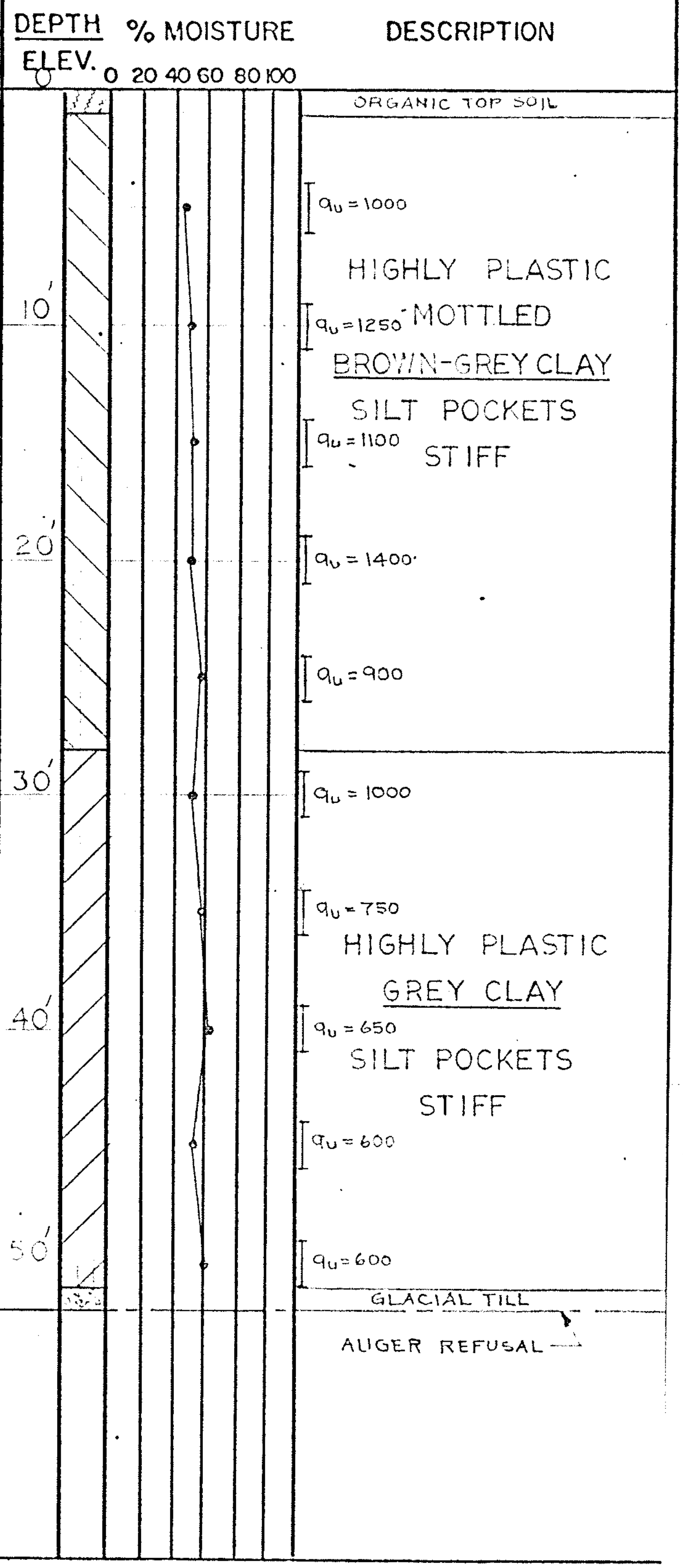
HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J. A. DATE DEC. 30/70
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER PARKIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 51
COORDINATES E-3
LOCATION N.E. MULTI-PURPOSE BUILDING



HOLE ADVANCED BY 15" POWER AUGER
LOGGED BY J.A. DATE DEC.31/70
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER PARKIN BY RIPLEY KLOHN LEONOFF

LOG OF TEST HOLE NO. 52
COORDINATES E-3
LOCATION N.E. MULTI-PURPOSE BUILDING



HOLE ADVANCED BY 16" POWER AUGER
LOGGED BY J.A. DATE DEC.31/70
REFERENCE FROM REPORT PREPARED FOR SMITH CARTER PARKIN BY RIPLEY KLOHN LEONOFF

APPENDIX B

CALCULATION OF FOUNDATION BEARING CAPACITY AND SETTLEMENT (ADMINISTRATION BUILDING)

I. Bearing Capacity The net ultimate bearing capacity was computed using Hansen's General Bearing Capacity Theory (Equation 4.1).

Total Foundation Loads

From Table (5.1) the net load on the foundation is:

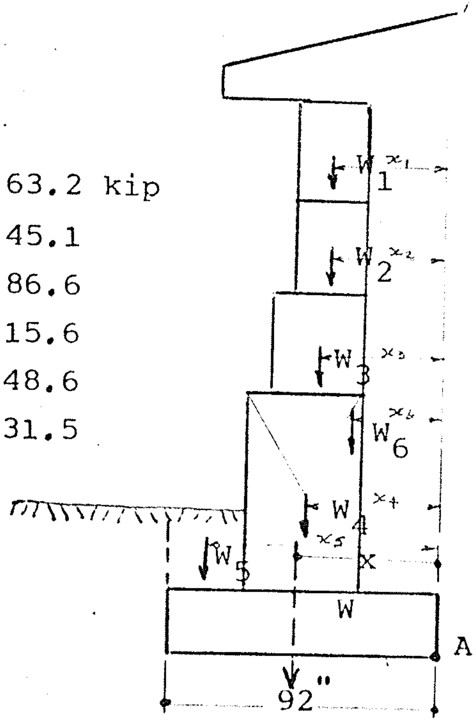
$$Q_t = Q_{dn} + Q_1$$

$$= 341.2 + 31.5$$

$$\approx 373 \text{ Kips .}$$

Point of Application of Resultant Load

W_1 = 63.2 kip
W_2 = 45.1
W_3 = 86.6
W_4 =115.6
W_5 = 48.6
W_6 = 31.5



x_1 = 38 inch
x_2 = 38
x_3 = 44
x_4 = 46
x_5 = 77
x_6 = 33

The individual loads on the foundation are :

W_1 = Roof D.L. + 3rd Flr. Wall,

W_2 = 2nd Flr.Wall + 3rd Flr.Slab + Steel Beams + Flr. & Ceiling D.L.,

W_3 = 1st Flr.Wall + 2nd Flr.Slab + Steel Beams + Flr. & Ceiling D.L.,

W_4 = Basement Wall + Footing + 1st Flr.Slab + Steel Beams + Flr. & Ceiling D.L. + Basement Flr. D.L.,

W_5 = Weight of Soil,

W_6 = Resulting L.L. on all floors.

Therefore, the total load is :

$$\Sigma W = 63.2 + 45.1 + 68.6 + 115.6 + 48.6 + 31.5$$

$$= 373 \text{ kips.}$$

By taking moments of the loads about A :

$$\Sigma M_A = 0$$

$$\Sigma W \times X = (63.2)(38) + (45.1)(38) + (86.6)(44) + (115.6)(46) + (48.6)(77) + (31.5)(33) \quad \text{inch kip}$$

$$373\ X = 2{,}400 + 1{,}710 + 3{,}020 + 5{,}300 + 3{,}740 + 1{,}040$$

Therefore, the location of the resultant load is :

$$X = \frac{17,210}{373}$$

$$= 46.1 \quad \text{inches.}$$

That is, the resultant acts approximately through the centroid of the footing.

Design Soil Pressure

From the net load on the foundation, the net soil pressure is:

$$\text{Net Soil Pressure} = \frac{\text{Net Foundation Load}}{\text{Footing Area}}$$

$$= \frac{373,000}{(7.7)(15)}$$

$$= 3,230 \quad \text{psf.}$$

Net Ultimate Bearing Capacity

Using the Equation (4.1.a) for cohesive soil:

$$q_{ult.net} = cN_c s_c d_c i_c$$

$$= (800)(5.4)(1.1)(1.0)(1.0)$$

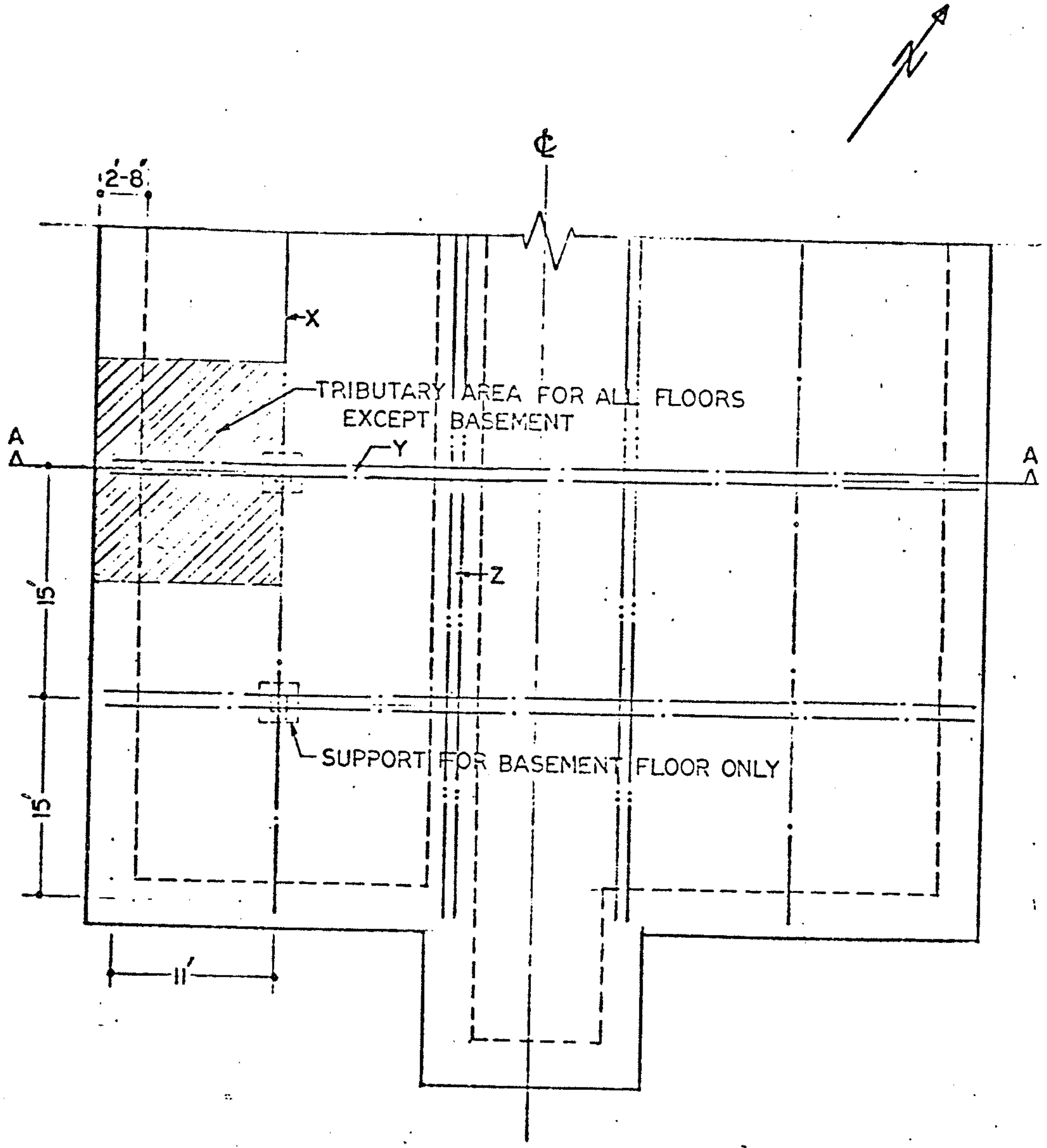
$$= 4,520 \quad \text{psf.}$$

Factor of Safety

$$\text{S. F.} = \frac{Q_{ult.net}}{\text{Soil Pressure}}$$

$$= \frac{4,520}{3,230}$$

$$= 1.40$$



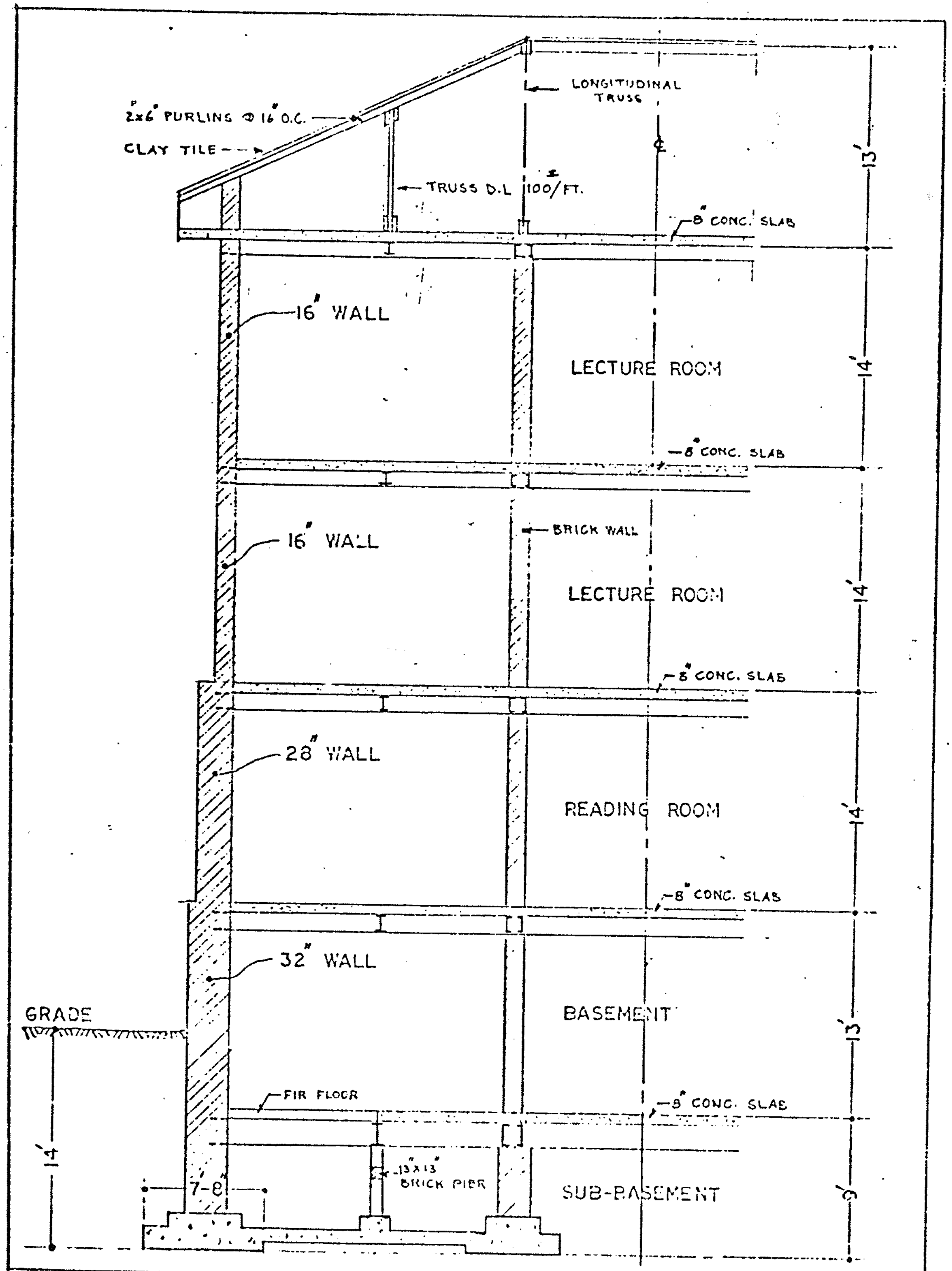
NOTES:

X.	FOR ALL FLOORS	1-12" I-BEAM 31.5#
Y.	FOR BASEMENT FLOOR	2-18" I-BEAMS 55#
	FOR OTHER FLOORS	2-15" I-BEAMS 80#
Z.	FOR ALL FLOORS	2-12" CHANNELS 20.5#

ADMINISTRATION BUILDING
TYPICAL FLOOR PLAN

SCALE
10 ft.

FIGURE B-1



ADMINISTRATION BUILDING
SECTION A-A

SCALE 1"=10'

FIGURE B-2

II. Settlment Settlements at point A, B, and C (Figure 5.1) were estimated by the approximate method assuming a circular footing supporting the column load, and the bases of the footing are at 10 foot depth below grade.

A. Settlement Calculation at Point A

Using the method of analysis outlined in Chapter V, the settlement is calculated as follows:

Area of footing = 155 sq.ft.

Representative 'r' value = $\sqrt{\frac{155}{\pi}}$ = 6.05 ft.

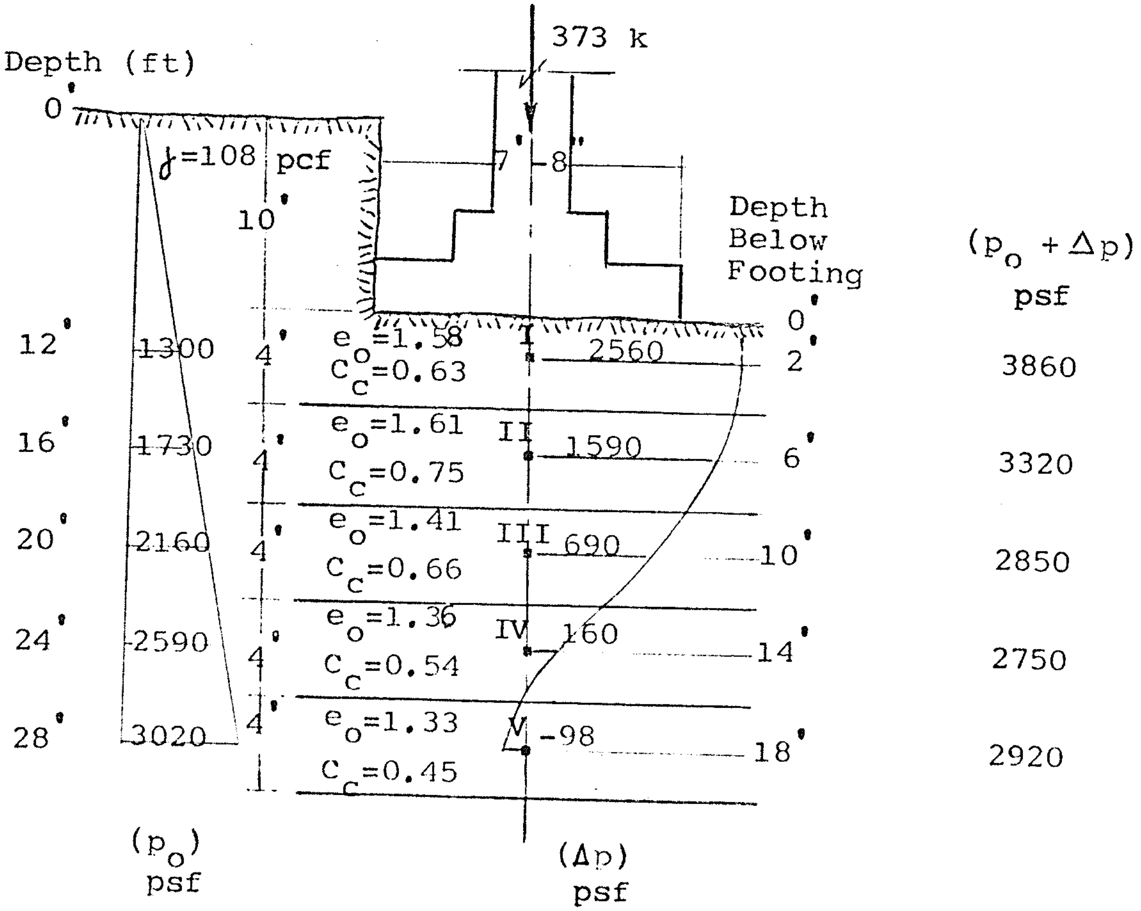


Figure B-3 Soil Sub-division Under Footing at Point A

Stress Increase Due to Footing Loads

Point	Depth (ft)	r/z	I	q = 3230 I (psf)
I	2	3.0	0.96	3100
II	6	1.0	0.66	2130
III	10	0.60	0.33	1230
IV	14	0.43	0.22	710
V	18	0.33	0.14	452

Where: r = representative radius of footing,

z = depth below footing,

q = vertical stress at depth z,

q_o = soil pressure at base of footing.

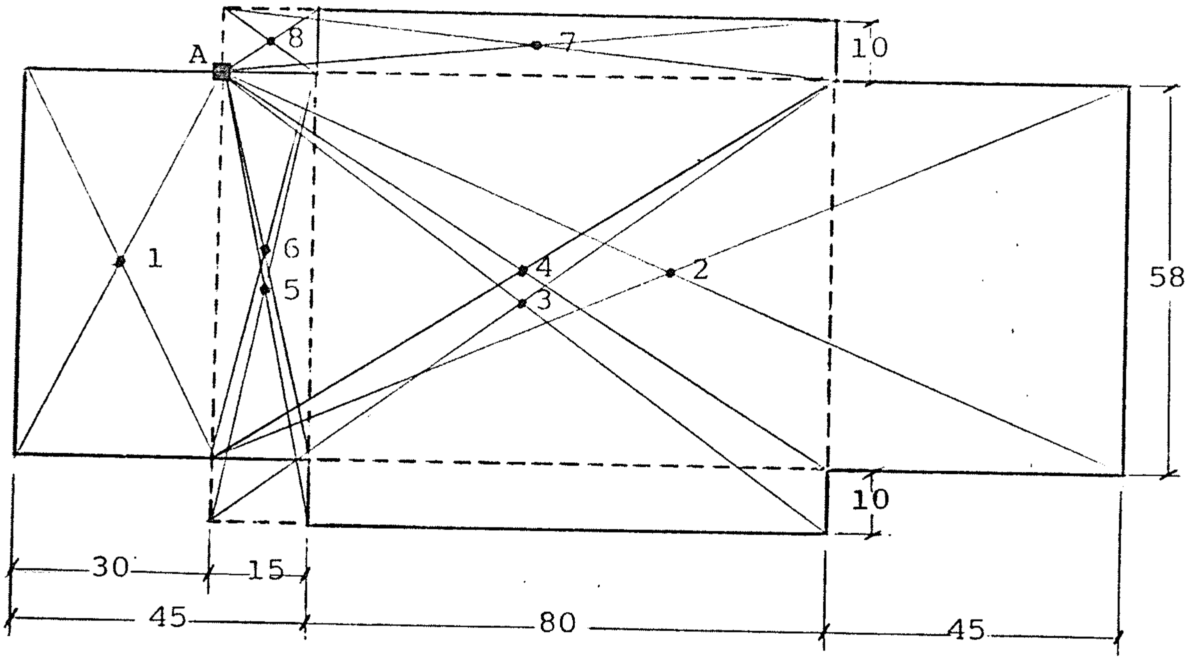
Note: I values are given in Table (5.2)

Stress Decrease Due to Excavation

In plan area, the excavations can be considered to consist of superimposed rectangles. For calculating the stress decrease due to excavation, Newmark's Tables, giving influence number I_σ, for the stress under a rectangular footing were used.

$$\Delta\sigma_z = I_\sigma q$$

Subdivision of Excavation Area into rectangles for use of Newmark's Tables for Rectangular Footings.



Thus the vertical stress decrease is:

Point	Depth (ft)	I_σ	$\Delta\sigma z = (1080) I_\sigma$ (psf)
I	2	0.501	540
II	6	0.501	540
III	10	0.509	540
IV	14	0.510	550
V	18	0.511	550

Note: q = 1,080 psf
= overburden pressure at 10 ft depth

Computed Consolidation Settlement

Settlements were computed using two methods, the first one made use of over-burden pressures and the compressive index in the virgin curve region, and the second one considered the pre-consolidation effect.

Method I. $$\Delta H = \frac{H}{1 + e_o} C_c \log \frac{P_o + \Delta P}{P_o} \qquad (4.6)$$

Method II. $$\Delta H = \frac{H}{1 + e_o} (\Delta e) \qquad (4.7)$$

Computed Consolidation Settlement for Point A

Method	Point					Sum (in)
	I	II	III	IV	V	
I	5.53	3.90	1.57	0.28	-0.13	11.15
II	1.30	0.74	0.40	0.40	-0.21	2.63

By the same method used for point A, the settlement of points B and C are obtained.

APPENDIX C

CALCULATION OF FOUNDATION BEARING CAPACITY AND SETTLEMENT

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Footing L-5 (see Figure 5.2) was analysed to establish the design soil pressure. Settlements at footings L-5, E-5, and P-12 were then computed representing approximate points of highest and lowest settlement values.

I. Bearing Capacity

Net Foundation Loads

From Table (5.4) the net load on the foundation is:

$$Q_t = Q_{dn} + Q_1$$

$$= 74.6 + 44.1$$

$$= 118.7 \quad \text{Kip}$$

Design Soil Pressure

$$\text{Net Soil Pressure} = \frac{\text{Total Foundation Loads}}{\text{Footing Area}}$$

$$= \frac{118,700}{49}$$

$$= 2420 \quad \text{psf.}$$

Net Ultimate Bearing Capacity

From the Equation (4.1.a) for cohesive soil:

$$q_{ult.net} = c\ N_c\ s_c\ d_c$$

$$= (800)(5.4)(1.3)(1.5)$$

$$= 7{,}500 \quad \text{psf.}$$

Factor of Safety

$$\text{S. F.} = \frac{Q_{ult.net}}{\text{Soil Pressure}}$$

$$= \frac{7{,}500}{2{,}420}$$

$$= 3.1$$

II. Settlement

Settlement under footings L-5 were computed using Equations (4.6) and (4.7). Because of the more simple geometry of the building foundation pressures were calculated on the basis of Newmark's Tables and Charts given in most standard text books, for example Terzaghi.[36] The results are summarized in Table 5.5.

APPENDIX D

COMPUTED BEARING CAPACITY AND SETTLEMENT OF SELECTED FRICTION PILES

Pile Foundation 1 (B - 3)

This foundation consists of a cluster of three 16 inch diam. by 30 ft. cast-in-place concrete piles. The pile spacing is 3 ft. centre to centre and has a triangular pile cap of 10 sq.ft.

1. Individual Action:

$$\text{Shear strength of clay} = \frac{2220}{2} = 1110 \text{ psf.}$$

Since the zone of soil shrinkage is about 6 ft.:

$$\text{Effective length of pile} = 30 - 6$$

$$= 24 \text{ ft.}$$

$$\text{Surface perimeter} = \pi\left(\frac{16}{12}\right)(24)$$

$$= 100 \text{ sq.ft/pile}$$

Hence from Equations (6.2) and (6.3.a)

$$Q_s/\text{pile} = (0.8)(1110)(100)$$

$$= 88.8 \text{ kip.}$$

$$Q_s = 3(88.8) \approx 266 \text{ kip.}$$

$$\text{Area of pile tip} = \frac{\pi}{4}\left(\frac{16}{12}\right)^2$$

$$= 1.4 \text{ sq.ft/pile}$$

Hence from Equation (6.6.b)

$$Q_p\text{/pile} = 9\ (1110)\ (1.4)$$

$$= 14.0 \text{ kip.}$$

$$\text{Total } Q_p = 3\ (14) = 42.0 \text{ kip.}$$

Therefore, the ultimate capacity of the pile group is:

$$Q_{ult} = 266 + 42 = 308 \text{ kip.}$$

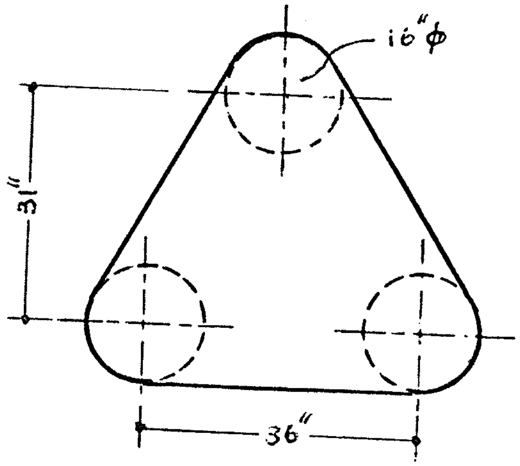
Total column load is 40 kip, hence:

$$\text{S. F.} = \frac{308}{40} = 7.7.$$

2. Group Action:

Group surface perimeter = $(3)\ (\frac{52}{12})\ (24)$

= 312 sq.ft.



Q_s = (0.8)(1110)(312) = 277 Kip.

Bearing area of pile cluster:

$$= \frac{2.6 \times 3}{2} + 3(0.67 \times 3)$$

= 10 sq.ft.

Q_p = (9)(1110)(10) = 100 Kip.

Hence:

Q_{ult} = 277 + 100 = 377 Kip.

Therefore

$$S.\ F. = \frac{377}{40} = 9.4$$

Settlement

In evaluating the stresses in the soil underlying a group of friction piles, the load is placed on a fictitious footing at the lower third point of the effective pile length, and the spread-out of load is taken at 30^o to the vertical.

1. Computation for values of $p_o = \Sigma \gamma z$

From Figure (7.2)

$5(107) = 535$ lb per sq.ft

$25(108) = 2,700$

$3.5(111) = \underline{388}$

$3,623$ lb per sq.ft. $= p_o$ at point I

$5.5(111) = 610$

$5.5(106) = \underline{583}$

$4,816$ lb per sq.ft. $= p_o$ at point II

Note: For soil profile see Figure (7.2)

2. Computation for value of Δp:

A simplified method of computation of soil stresses beneath a pile was used. The area of the fictitious footing is 10 sq.ft. supporting a column load of 20 tons which gives the equivalent load of 2.0 tsf.

Load spread area at 33.5 foot depth:

$$= \frac{8.67 \times 10}{2} + 3(0.67 \times 10)$$

$$= 63 \quad \text{sq.ft.}$$

Hence:

$$\Delta p \text{ at point I} = 2.0 \times \frac{10}{63} = 0.32 \quad \text{tsf.}$$

Load spread area at 44.5 foot depth:

$$= \frac{20.8 \times 24}{2} + 3(0.67 \times 24)$$

$$= 298 \quad \text{sq.ft.}$$

Hence:

$$\Delta p \text{ at point II} = 2.0 \times \frac{10}{298} = 0.07 \quad \text{tsf.}$$

3. Settlement Computations:

The magnitude of total consolidation settlement is expressed by:

$$s = \frac{H}{1 + e_o} \cdot \Delta e \qquad (4.7)$$

Where Δe is the change in void ratio between initial and final pressures and is taken directly from consolidation curves in Figure (3.1).

Thus, settlement at point I is

$$S = \frac{11 \times 12}{1 + 1.72} \times (0.02) = 0.97 \quad \text{inch}$$

Settlement at point II is

$$S = \frac{11 \times 12}{1 + 1.92} \times (0.01) = 0.45 \quad \text{inch}$$

Therefore, the total consolidation settlement

$$= 0.97 + 0.45$$

$$= 1.42 \quad \text{inches.}$$

Pile Foundation 2 (D.5-2)

The footing is supported by a cast-in-place friction pile of 16" in diameter by 25 ft. long. It carries the total column load of 14 Kips.

Pile Capacity

1. Load Carried by Friction:

Assumed the zone of soil shrinkage of 6 feet below basement level.

Effective length $= 25 - 6 = 19$ ft.

Surface area $= \pi (\frac{16}{12})(19) = 80$ sq.ft.

Hence:

$$Q_s = (0.8)(1110)(80)$$

$$= 71.0 \quad \text{Kip.}$$

2. Load Carried by End-bearing:

Area of pile tip $= \frac{\pi}{4} (\frac{16}{12})^2 = 1.4$ sq.ft.

Hence:

$$Q_p = (9)(1110)(1.4)$$

$$= 14.0 \quad \text{Kip.}$$

3. Ultimate Carrying Capacity:

$$Q_{ult} = Q_s + Q_p$$

$$= 71 + 14$$

$$= 85 \quad \text{Kip.}$$

Since column load is 14 Kips

$$S.F. = \frac{85}{14} = 6.1.$$

Settlement

The soil conditions under the footing is shown in Figure (7.3).

1. Computation for Values of $p_o = \Sigma \gamma z$

From Figure (7.3)

5(107) =	535	lb per sq.ft.
25(108) =	2,700	
2.5(111) =	278	
	3,513	lb per sq.ft. = p_o at point I
7(111) =	777	
	4,290	lb per sq.ft. = p_o at point II
7(106)	742	
	5,032	lb per sq.ft. = p_o at point III

2. Computation for Value of Δp:

The area of the fictitious footing is 1.4 sq.ft. carrying the equivalent load of 5 ton per sq.ft. The spread out of load is assumed to be 30 degree measured from the vertical.

Load spread area at 32.5 foot depth:

$$= \frac{\pi}{4} (5.3)^2 = 22 \quad \text{sq.ft.}$$

Δp at point I $= 5 \times \frac{1.4}{22} = 0.32$ ton per sq.ft.

Load spread area at 39.5 foot depth:

$$= \frac{\pi}{4} (13.5)^2 = 143 \text{ sq.ft.}$$

Δp at point II $= 5 \times \frac{1.4}{143} = 0.05$ ton per sq.ft.

Load spread area at 46.5 foot depth:

$$= \frac{\pi}{4} (21.6)^2 = 367 \text{ sq.ft.}$$

Δp at point III $= 5 \times \frac{1.4}{367} = 0.02$ ton per sq.ft.

3. Settlement Computations:

At point I $S = \frac{7 \times 12}{1 + 1.72} \times (0.02) = 0.62$ inch.

At point II $S = \frac{7 \times 12}{1 + 1.84} \times (0.01) = 0.30$ inch.

Settlement at point III is negligible. The total consolidation settlement is therefore:

$$\text{Total settlement} = 0.62 + 0.30$$

$$= 0.92 \text{ inch.}$$

APPENDIX E

COMPUTED CARRYING CAPACITY OF SELECTED END-BEARING PILES

Four typical footings of the University Centre which rest on precast, concrete end-bearing pile were analysed for the bearing capacity. The location, size and column load of piles 607, 719, 720, and 766 considered are shown in Figure (7.4).

Pile No. 607

1. General Data:

This is a 14" hexagonal concrete pile with a net length of 42 feet. The other pertinent physical data are as follow:

Perimeter	3.5	ft.
Cross-sectional area	0.88	Sq.ft.
Weight	5,550	lb.
Set	0.11	in/blow
Column load carried	109	Kip.

2. Computed Safe Design Load Using Pile Driving Formula:

The modified Engineering News formula, which implies a safety factor of 6.0, was used.

$$R_d = \frac{2En}{S + 0.1} \cdot \frac{W_r + e^2 W_p}{W_r + W_p} \qquad (6.1)$$

where the various symbols are as previously defined.

The values of En, W_r, and S were obtained from the contractor's pile driving report and were tabulated below:

Pile No.	En (lb)	W_r (lb)	W_p (lb)	S (inch/blow)
#607	20,000	5,000	5,930	0.11
#719	30,000	5,000	7,640	0.07
#720	20,000	5,000	4,320	0.18
#766	20,000	5,000	5,250	0.21

Hence:

$$R_d = \frac{2(20,000)}{0.11 \text{ x } 0.1} \cdot \frac{5,000 + (0.06)(5,930)}{5,000 + 5,930}$$

$$= \frac{40,000 \text{ x } 5,356}{0.21 \text{ x } 10,930}$$

$$= 94 \quad \text{Kip.}$$

3. Computed Ultimate Capacity Using Static Method:

For calculating the bearing capacity of end-bearing piles, the typical bearing-capacity equations for shallow footings were used. For a circular footing of radius, r, resting on glacial till:

$$q_p = \left[0.6 \gamma \frac{D}{2} N_\gamma + \gamma D_f \ (N_q - 1)\right] \qquad (6.6.d)$$

The values of c and γ of till were taken from Test Hole 9 and the value of the internal friction of till was taken as 44° for the reasons given previously in Section (7.2).

The bearing capacity factors after Vogel for $\emptyset = 44^\circ$ are:

$$N_c = 151, \qquad N_\gamma = 252, \qquad N_q = 147.$$

Conservatively, cohesion may be neglected in a material that has such a high angle of internal friction. Hence the computed net ultimate bearing capacity is:

$$q_p = \left[(0.6)(143-62.4)(\frac{7}{12})(252)+(118-62.4)\right.$$
$$\left.(42.3)(147-1)\right]$$
$$= \quad 7.1 + 341$$
$$\approx \quad 348 \qquad \text{Kip per sq.ft.}$$

The net ultimate load neglecting the weight of the pile itself, therefore, is:

$$Q_{ult} = 348(0.88) = 306 \quad \text{Kip.}$$

If a factor of safety of 2.5 is chosen, then the allowable safe design load of the pile is 123 Kips.

The bearing values for piles 719, 720 and 766 were similarly calculated with appropriate substitution of shape factors. The results are shown in Table (7.2).

APPENDIX F

COMPUTATION OF CARRYING CAPACITY OF CAISSON

Caisson 1 (E-3) of the Education Building (Figure 7.1) was selected and its carrying capacity analysed. The caisson has a shaft of 28 inch diam. and is belled out to 32 inch diam. at the base.

The load-carrying area is therefore:

$$\text{Area} = \frac{\pi}{4}\left(\frac{32}{12}\right)^2 = 5.6 \quad \text{sq.ft.}$$

Since the caisson rests on the limestone bedrock, for which the allowable bearing capacity is 60 Kips per square foot by the Code, Section 4.2.2.1(2); the safe carrying capacity of the caisson, using Equation (6.9), is

$$\begin{aligned} Q_{safe} &= q_d \; A_p \\ &= (60)(5.6) \\ &= 334 \quad \text{Kip} \end{aligned}$$

The column load carried is 313 Kips which is less than 334 Kips, the allowable capacity of the caisson using the Code value. Hence the design is safe.

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