FOUNDATION INVESTIGATION

OF

GREATER WINNIPEG SUBSOILS

A THESIS

PRESENTED TO

THE FACULTY OF GRADUATE STUDIES AND RESEARCH

UNIVERSITY OF MANITOBA

In partial fullfillment

of the Requirements for the Degree

of Master of Science in Civil Engineering

BY

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SEPTEMBER 1957

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ABSTRACT

The subsoil of Greater Winnipeg generally consists of lacustrine and fluvial deposits of clay and silty clay materials. Because of the highly active nature of these materials, foundation and soil conditions have been noted for their unusual severity. The purpose of this thesis is to acquaint the engineer with the results of past experience and recently completed soil investigations in relation to the local foundation problems.

More than 350 boring records were collected, including approximately 100 for which the physical properties had been determined. These properties were studied and condensed to permit an investigation into the foundation problems of the Greater Winnipeg area.

September 1957

TABLE OF CONTENTS

ACKNOWLEDGEMENTS

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The author is also indebted to Professor Andrew Baracos for his technical guidance and assistance.

Acknowledgement is also made to the Subterranean Limited, Smith and Hansen and Baldry Engineering for the use of their boring records.

CHAPTER I

INTRODUCTION

Foundation and soil conditions in the Greater Winnipeg area have been noted for their unusual severity. Foundation failures, including the major failure of the Transcona Grain Elevator and many others of lesser magnitude have not too infrequently occurred. It is the purpose of this thesis to acquaint the engineer with the results of past experience and recently completed soil investigations in relation to foundation problems. The following problems merit attention.

1. Adequate bearing capacity
2. Total settlements
3. Differential movements including settlements and heave
4. Effects of lateral earth pressures
5. Frost
6. River bank stability and subsidence
7. Chemical deterioration of concrete

For this study, a large number of available boring records were collected. More than 350 boring records were obtained including approximately 100 from the University of Manitoba for which the physical properties of the materials had been determined. These properties were studied and are presented in a condensed form in the following chapters. This information is supplemented with a brief discussion of the pertinent geology of the area.

Records of past experience, including the work of the Winnipeg Branch of the Engineering Institute of Canada (1) and others have been reviewed (2-10).

CHAPTER II

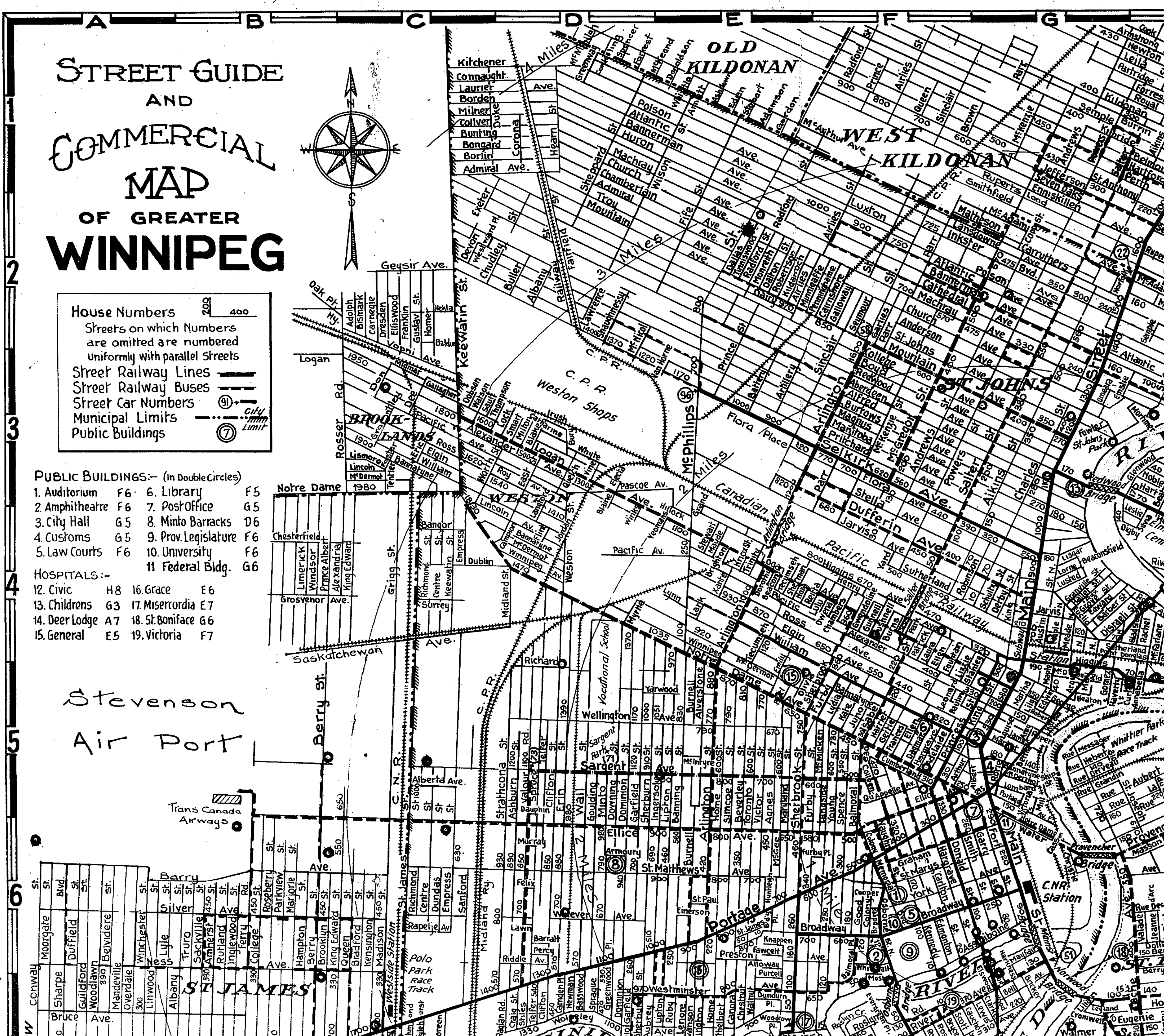
EXTENT OF THE AREA

The extent of the Greater Winnipeg area included in this study is shown in Figure 1. The Greater Winnipeg area includes the City of Winnipeg, the City of St. Boniface, the Town of Tuxedo, the Rural Municipalities of Fort Garry, St. Vital, East Kildonan, North Kildonan, West Kildonan, Rosser, St. James, and the Village of Brooklands.

The predominant natural feature of this area is that it is divided into three sections by the north flowing Red River and the east flowing Assiniboine River. This feature has influenced the layout of the streets which run north and south from the Assiniboine River and east and west from the Red River. The main thoroughfares tend to run radially from Portage Avenue and Main Street.

The ground surface of the area is generally flat and is about 760 feet above sea level. The elevation of the ground surface in the western portion of the area reaches 771 feet while in the central area, near the Redwood Bridge, the elevation is only 748 feet. The City of Winnipeg datum, elevation 0.00 feet, corresponds to the Geodetic elevation of 727.57 feet.

SCALE
1000' 2000' 3000' 4000'
Published by Roy T. Pickard
Copyright Canada
No. 14 HIGHWAY
To United States
No. 3 HY. -West
No. 1 HY. -East
No. 8 & No. 9-North
NORTH KILDONAN
EAST KILDONAN
WEST KILDONAN
OLD KILDONAN
Kildonan Park
ELMWOOD
ST. JOHNS
ST. BONIFACE
ST. VITAL
RIVER HEIGHTS
CRESCENTWOOD
FORT GARRY
C.P.R. Weston Shops
Union Stock Yards
St. Boniface Golf Course
St. Boniface Cemetery
Windsor Park Golf Course
Niakawa Golf Course
Polo Park Race Track
Whittier Park Race Track
Canadian National Railways
ASSINIBOINE
RED RIVER
SEINE RIVER
Portage Ave.
Main St.


STREET GUIDE AND COMMERCIAL MAP OF GREATER WINNIPEG
House Numbers 200 400
Streets on which Numbers are omitted are numbered uniformly with parallel Streets
Street Railway Lines
Street Railway Buses
Street Car Numbers
Municipal Limits
City Limit
Public Buildings
PUBLIC BUILDINGS:- (In Double Circles)
1. Auditorium F6
2. Amphitheatre F6
3. City Hall G5
4. Customs G5
5. Law Courts F6
6. Library F5
7. Post Office G5
8. Minto Barracks D6
9. Prov. Legislature F6
10. University F6
11 Federal Bldg. G6
HOSPITALS:-
12. Civic H8
13. Childrens G3
14. Deer Lodge A7
15. General E5
16. Grace E6
17. Misericordia E7
18. St. Boniface G6
19. Victoria F7
Stevenson Air Port
Trans Canada Airways
OLD KILDONAN
WEST KILDONAN
ST JOHNS
BROOKLANDS
WESTON
C.P.R. Weston Shops
ST JAMES
ST. CHARLES
TUXEDO
RIVER HEIGHTS
CRESCENTWOOD
ASSINIBOINE
RIVER
RED
FORT GARRY
Assiniboine Park
Tuxedo Golf Course
Polo Park Race Track
C.N.R. Station
C.N.R. Yards
Canadian National Railways
PORTAGE AVE. WEST - CONTINUED
CONTINUED BELOW
No 1 HIGHWAY Trans Canada Hy. - West
No. 14 HIGHWAY To United States No. 3 HY. - West
SCALE
0 1000' 2000' 3000' 4000'
Published by Roy T. Pickard
Copyright Canada

GEOLOGY OF GREATER WINNIPEG

The geological history of this region has an important bearing on soil and related foundation conditions and for this reason the pertinent geological features will be discussed.

Bedrock

The underlying rock in Southern Manitoba consists chiefly of Ordovician limestone beds with a thin covering of shale in the western portion of this area. The bedrock in the Greater Winnipeg Area is approximately sixty to seventy feet below the ground surface.

Glaciation

During the Pleistocene ice age, the central portion of North America was covered by the Wisconson continental glacier or ice sheet. As this sheet of ice advanced southward, it smoothed the limestone beds as well as adding shale and clay to the stones and rockflour which were derived from the granites and limestones which were brought into this area from the north. The ice sheet overrode this material and consolidated it to a very great extent. This consolidated material is known locally as "hardpan" and it is on this material that most deep foundations are supported.

When the ice sheet melted, it retreated from the south-east and south-west, as is shown by the directional trend of the morainic hills. The meltwaters of the glacier flowed toward, rather than away from the glacier, and formed a lake which is known as Lake Agassiz. The drainage flowed toward the glacier because of a high ridge of land which is now the headwaters of the Red and Mississippi Rivers. The only outlet of major importance was in the north, which is now the Nelson River, and it was blocked by the glacier. Lake Agassiz grew to a size approximately 700 miles long and 250 miles wide before the glacier had receded enough to uncover the northern outlet. Greater Winnipeg lies in the Lake Agassiz basin which is known locally as the Red River Valley.

The formation of Lake Agassiz enabled fluvial and lacustrine material to be deposited over the boulder till, which resulted in local smoothing of the surface. In Greater Winnipeg the depth of these fine grained lacustrine materials is about fifty feet. This large depth is the result of the large depth of water which stood over Greater Winnipeg. There is evidence in the beaches of Western Manitoba that Lake Agassiz, at one time, was over 500 feet deep at the site that is now Greater Winnipeg.

The wave action of this lake was of sufficient strength to erode the ice laid deposits. This wave action removed the finer material and left the coarse material. The finer material was transported and deposited in the lower areas of the lake bottom. As the lacustrine deposits increased in depth, stratification took place and the clays were interbedded with thin layers of rockflour and silt or very fine sand. When the lake receded, the wave action diminished and only the finer materials were transported and deposited.

When the glacier melted from the south-west, glacial Lake Souris was formed. The main body of the glacier blocked the eastward outlet and as a result the flow from this lake was to the south. When the glacier had receded enough the eastward outlet, the Pembina Channel, was opened and the water from Lake Souris flowed into Lake Agassiz. The flow from Lake Souris brought sandy and silty clay material as well as water worked till from the south western Manitoba regions. This type of alluvial deposit occurred where streams flowed into the Lake Agassiz basin.

The stratification of the clays in the Lake Agassiz basin can be attributed to the summer and winter flows of the rivers which flowed into Lake Agassiz. In summer, flows of maximum discharge carried the heavier material into the lake, while in winter, flows of lesser discharge carried the lighter, finer material into the lake.

The surface covering over the Greater Winnipeg area consists of as much as several feet of organic topsoil or in some cases artificial fill. These materials are generally removed for any construction and for this reason will not be discussed as a foundation material. Beneath this surface covering is generally found the brown silty clay. This layer is generally up to 6 feet in thickness. The brown silty clay is usually underlain by a yellow silt stratum about one foot thick. Below the yellow silt is a layer about 16 feet thick, of brown clay with occasionally one or more thin layers of yellow silt. This material is generally varved. About 5 feet of grey brown clay lies below the brown clay and below this material is generally found about 20 feet of varved grey clay. The grey brown clay is a mixture of the brown clay and the grey clay. A mixture of rockflour, silt, gravel and sand, only partially consolidated and usually several feet in thickness, underlies the grey clay. Beneath this layer the same material is found, but it is highly consolidated. This material is the "hardpan". Below the "hardpan" is the Ordovician limestone bedrock.

A more detailed concept of the subsurface features can be obtained from the accompanying profiles. The location of these profiles marked A-A, B-B, and C-C, is shown in Figure 1. The location of the profiles was dictated by the locations of the borings. They were chosen, as much as practicable, to show the general variations in the subsoil conditions. The ground surface profile outside the City of Winnipeg is shown as dashed lines since no elevations were available for these areas. Adequate records were not available for the full length of the profile. Information in such cases can be added as it becomes available.

As can be seen from the profiles, the depths to the different materials are remarkably uniform. This can be attributed to the manner in which the material was deposited.

Figure 2

PROFILE A - A

Scales: Horizontal: 1 inch = 1000 feet

Vertical: 1 inch = 10 feet

Legend:

Topsoil

Sand

Brown Silty Clay

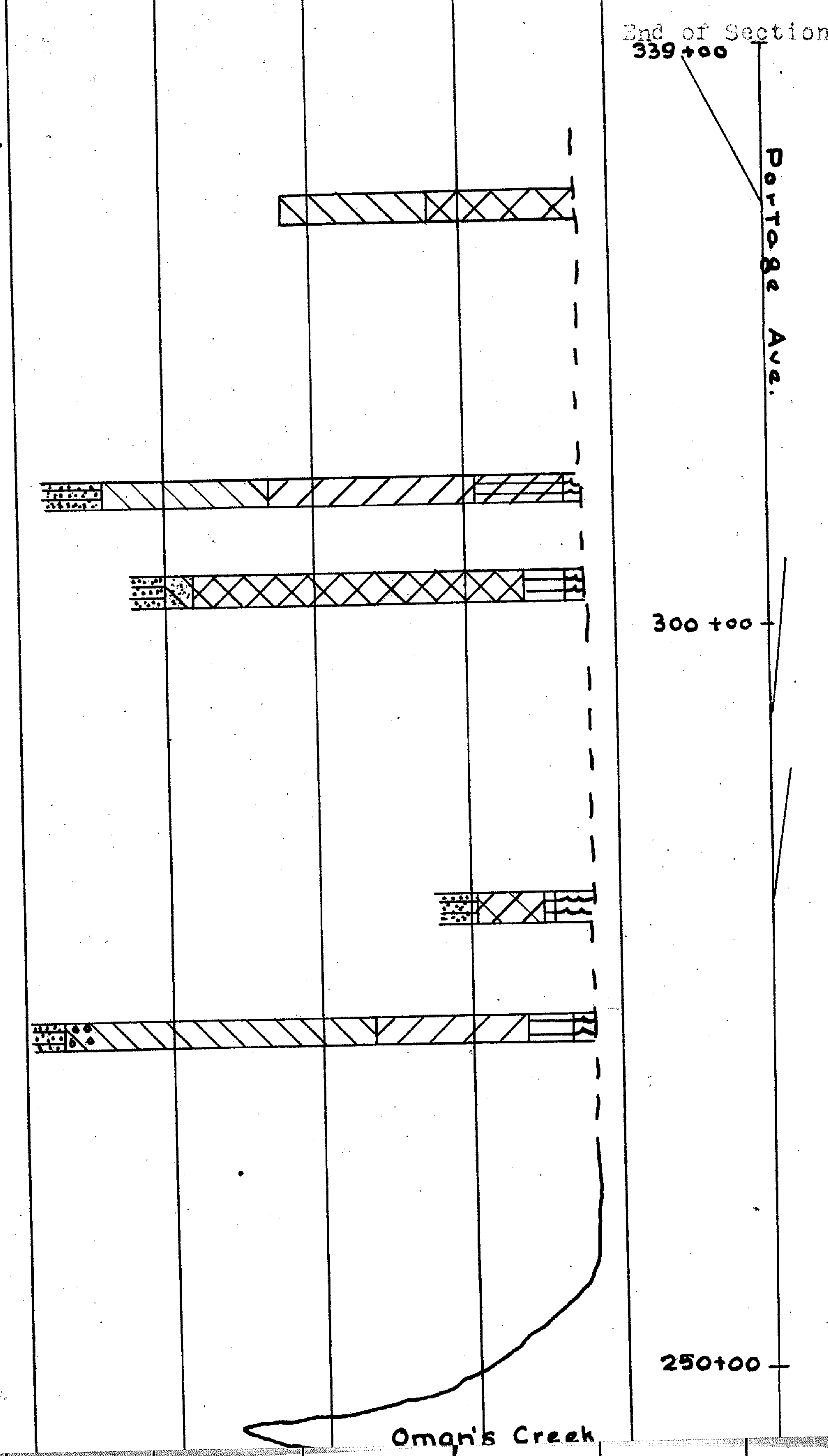
Yellow Silt

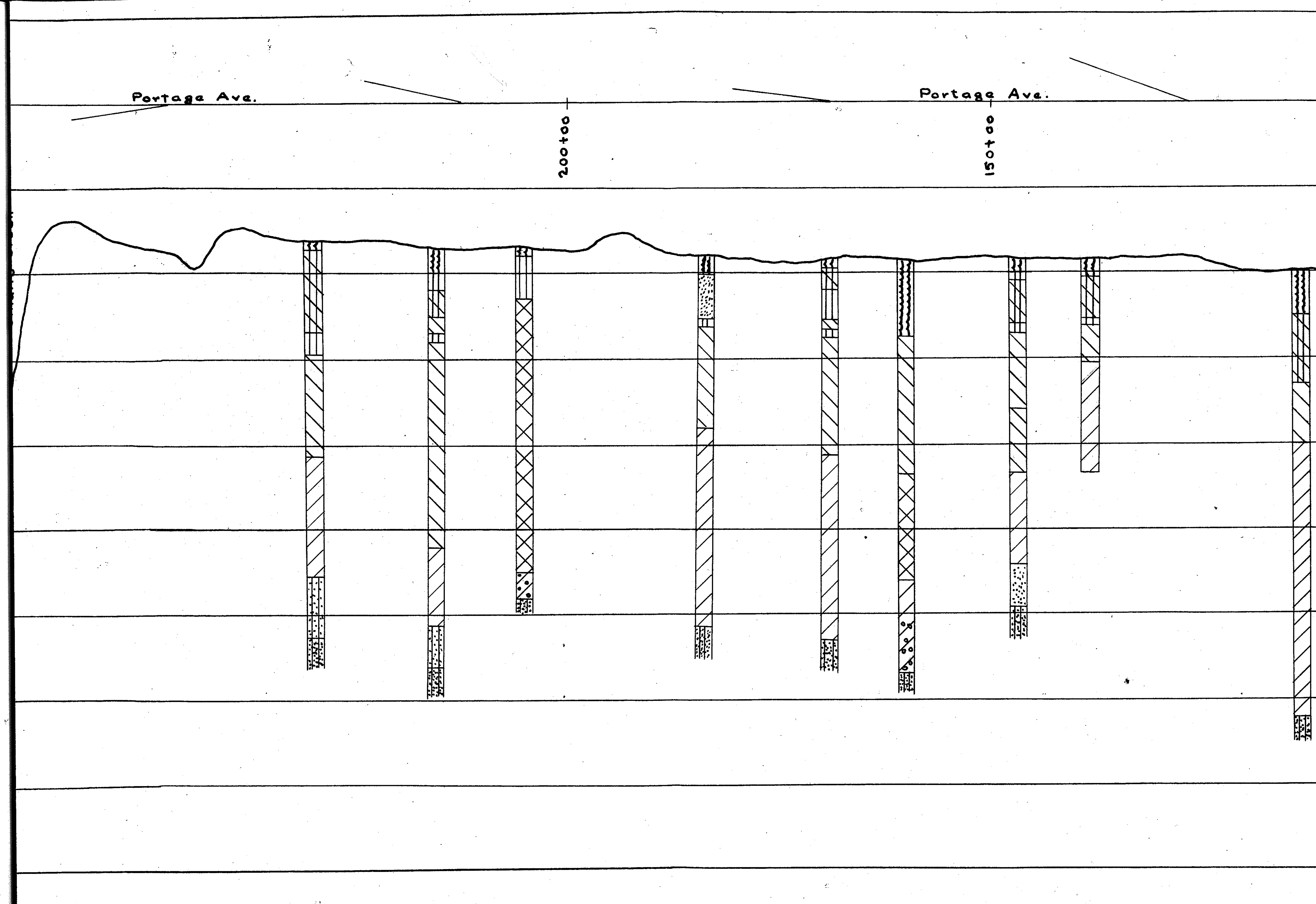
Brown Clay

Grey Brown Clay

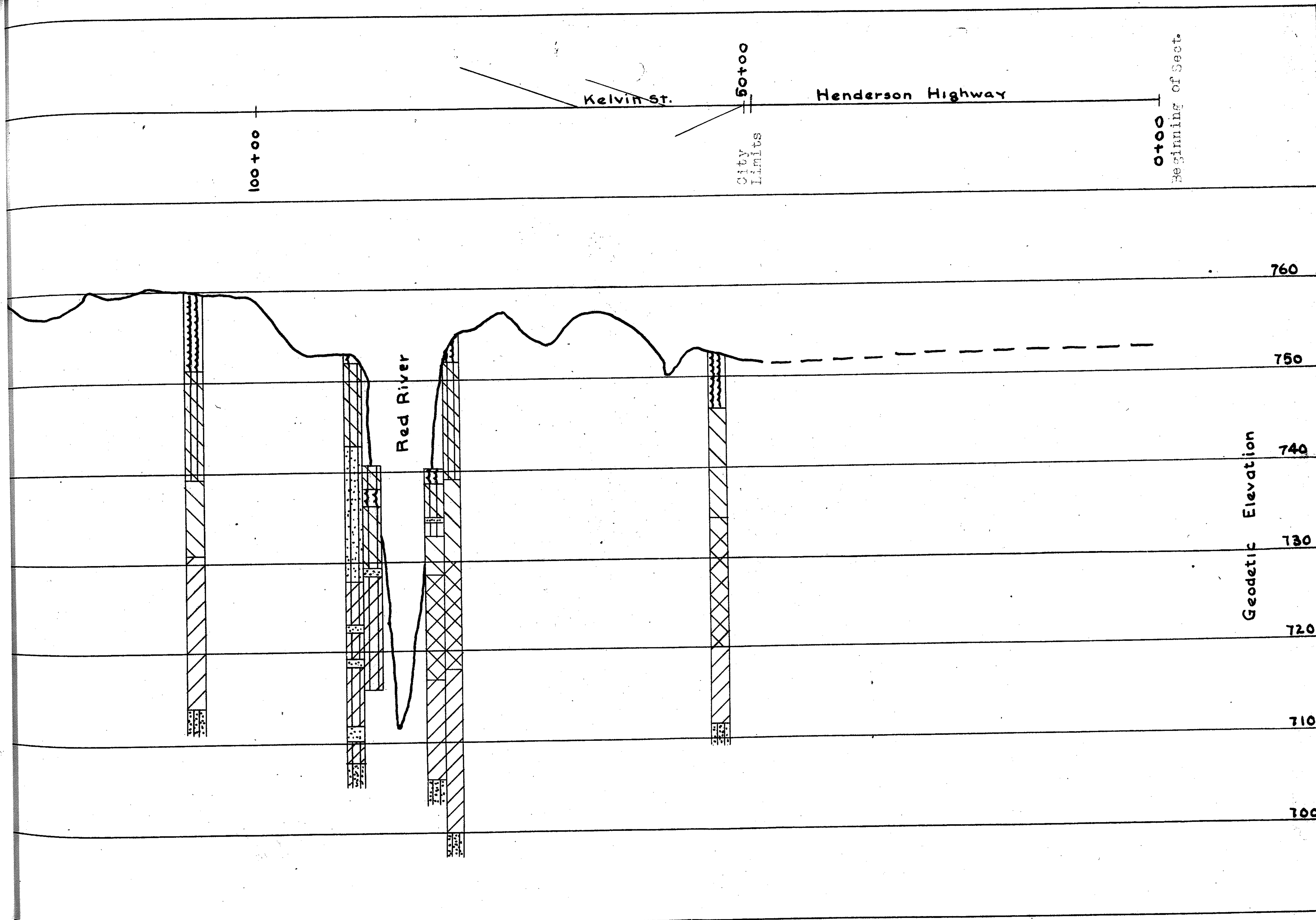
Grey Clay

Hardpan


End of Section
339+00
Portage Ave.
300+00
250+00
Oman's Creek



Portage Ave.
200+00
Portage Ave.
150+00



Kelvin St.
50+00
Henderson Highway
100+00
City Limits
0+00
Beginning of Sect.
Red River
Geodetic Elevation
760
750
740
730
720
710
700

Figure 3

PROFILE B - B

Scales: Horizontal: 1 inch = 1000 feet

Vertical: 1 inch = 10 feet

Legend:

Topsoil

Sand

Brown Silty Clay

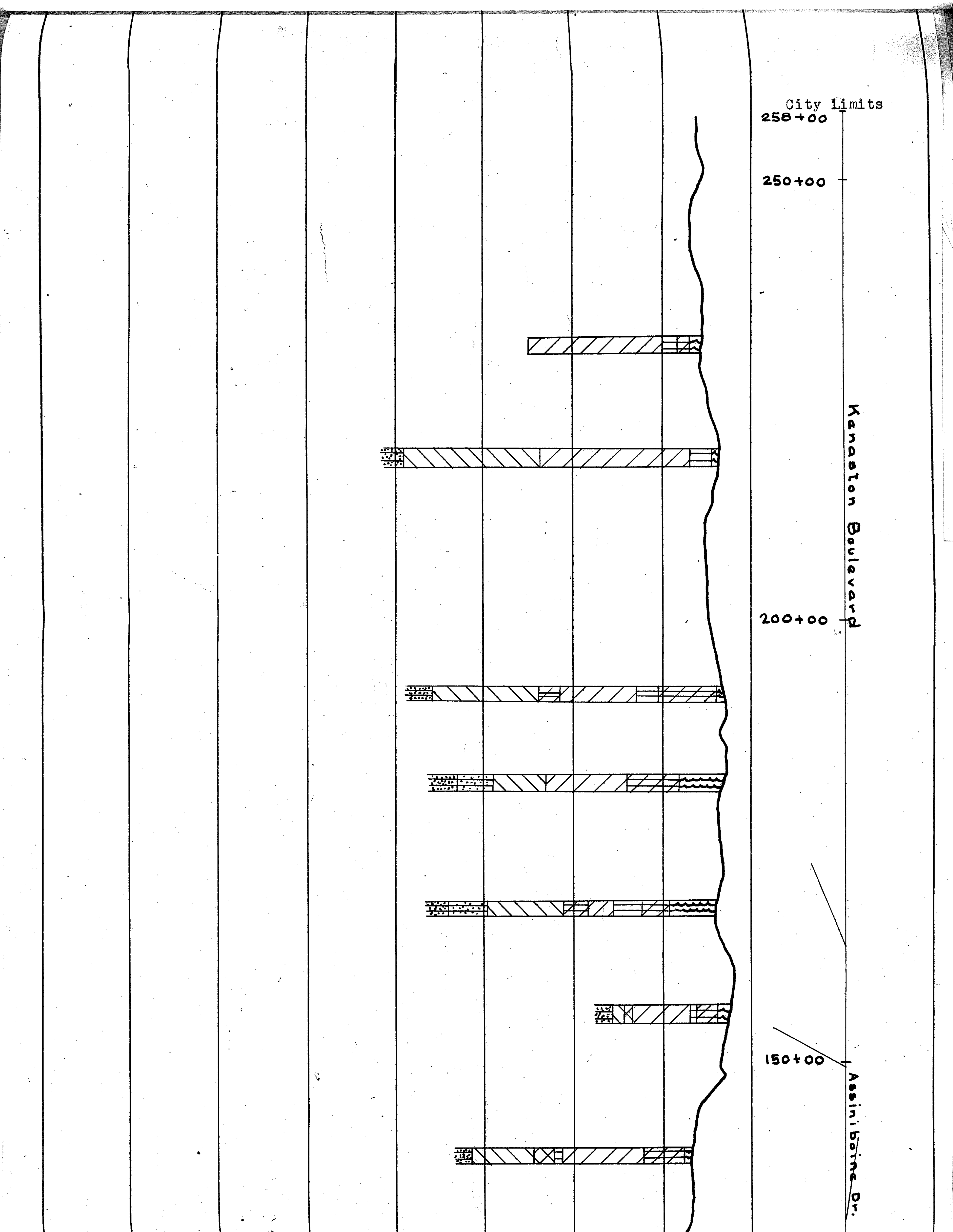
Yellow Silt

Brown Clay

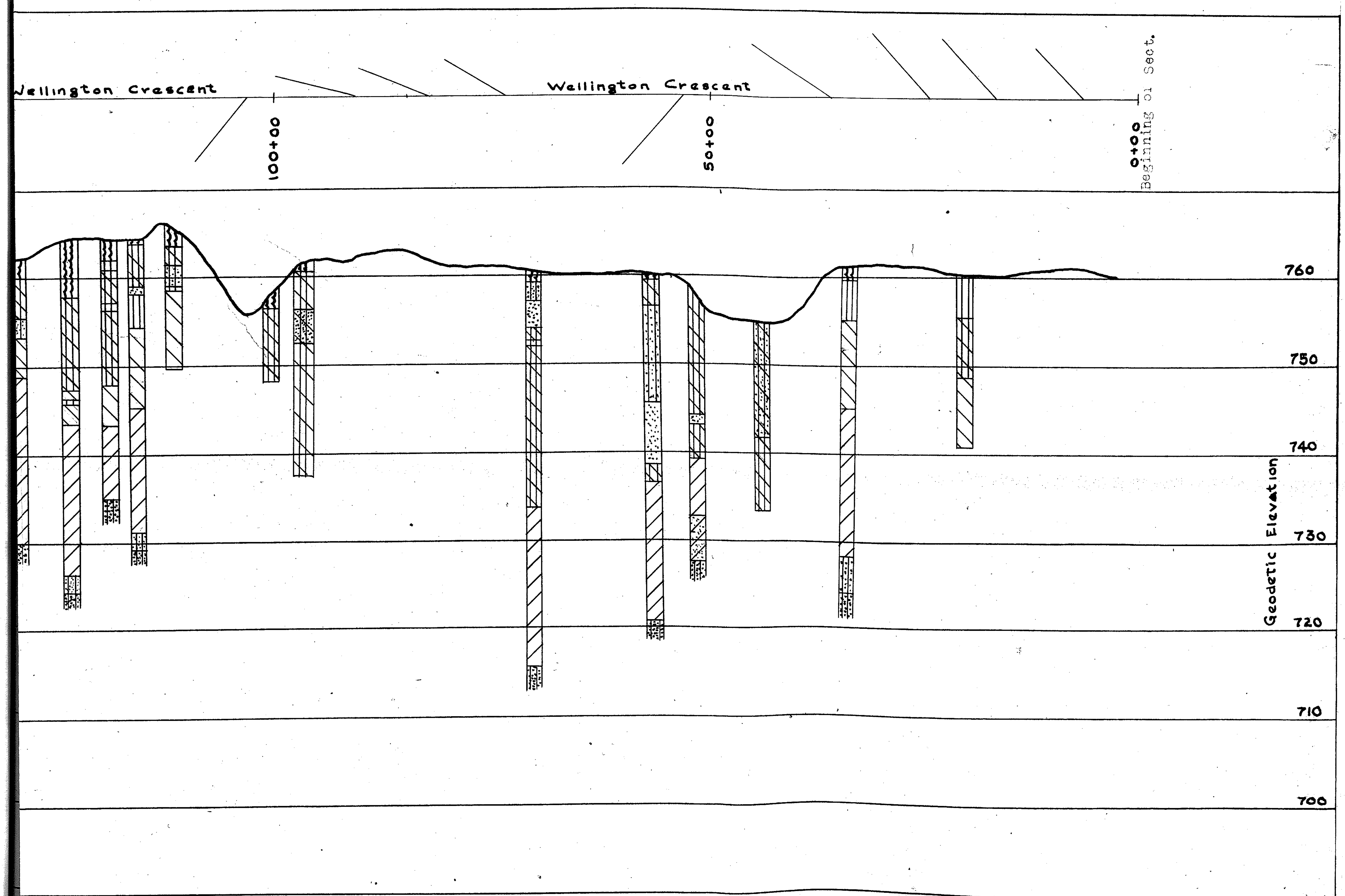
Grey Brown Clay

Grey Clay

Hardpan



City Limits
258+00
250+00
Kenaston Boulevard
200+00
150+00
Assiniboine Dr.



Wellington Crescent
Wellington Crescent
100+00
50+00
0+00
Beginning of Sect.
760
750
740
730
720
710
700
Geodetic Elevation

Figure 4

PROFILE C - C

Scales: Horizontal: 1 inch = 1000 feet

Vertical: 1 inch = 10 feet

Legend: Topsoil

Sand

Brown Silty Clay

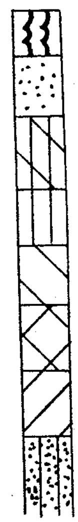
Yellow Silt

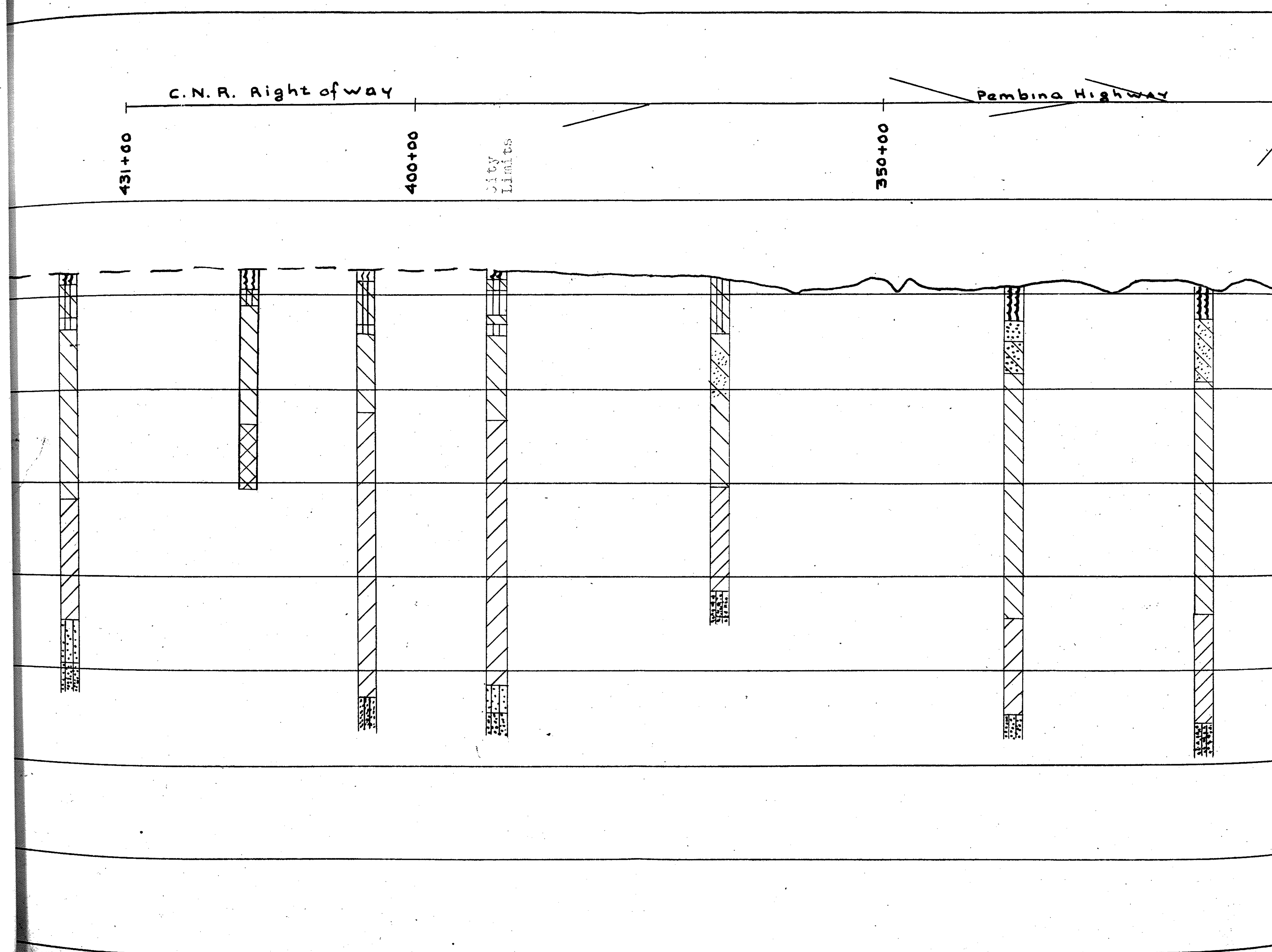
Brown Clay

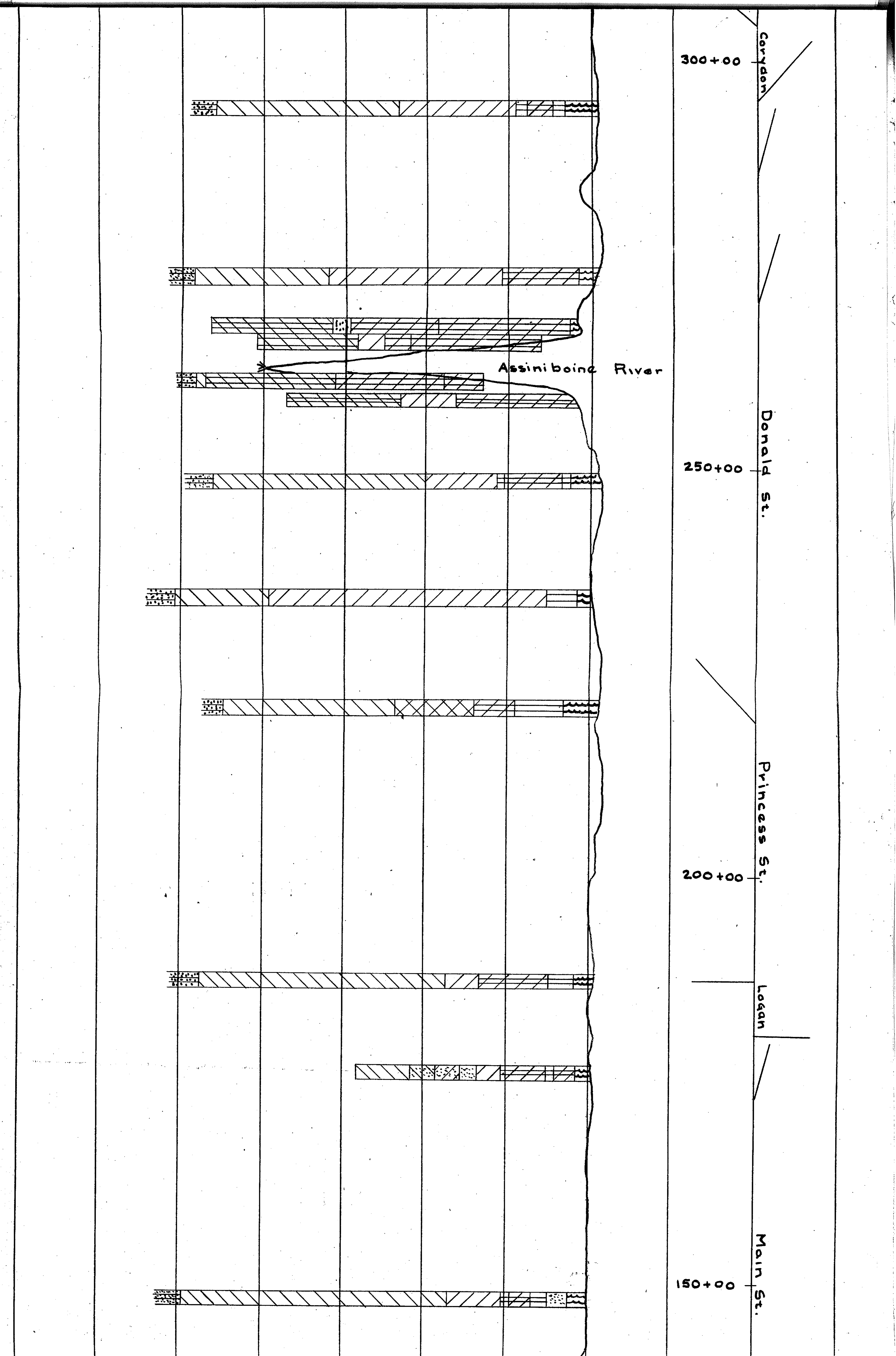
Grey Brown Clay

Grey Clay

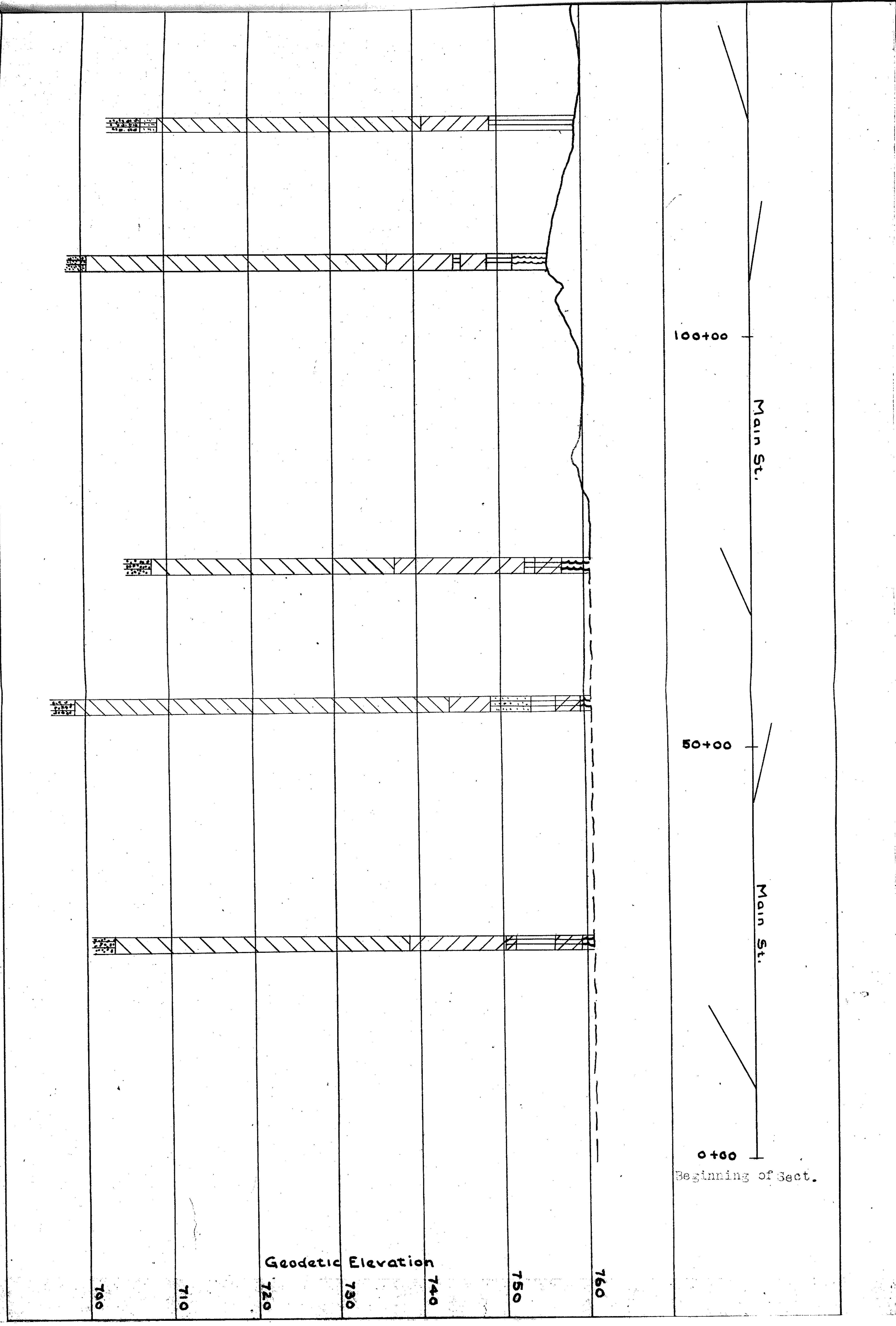
Hardpan




C.N.R. Right of way
Pembina Highway
431+00
400+00
City Limits
350+00



Corydon
300+00
Donald St.
250+00
Assiniboine River
Princess St.
200+00
Logan
Main St.
150+00


100+00
Main St.
50+00
Main St.
0+00
Beginning of Sect.
Geodetic Elevation
700
710
720
730
740
750
760

CHAPTER IV

SIGNIFICANT PROPERTIES OF GREATER WINNIPEG SOILS

The soils of Greater Winnipeg are predominately clays and silty clays. The most significant properties of these materials are the natural moisture content, the degree of saturation, the unconfined compressive strength, the Atterberg Limits, the compression index, the swelling pressure, the preconsolidation pressure, and the grain size distribution. These properties have been studied extensively at the University of Manitoba. The following is a discussion of these properties and the procedures employed in their determination.

Compressive Strength

The procedure generally followed at the University of Manitoba to determine the unconfined compressive strength was to load a cylindrical specimen, in single compression, to failure. The failure load, expressed as the load per unit of cross-sectional area, based on the actual cross-sectional area at failure, determines the unconfined compressive strength. The actual cross-sectional area was assumed to be equivalent to the increased cross-sectional area of the specimen due to its lateral expansion under load.

A table has been made showing the generally accepted relation between the consistency and unconfined compressive strength. This table (11) is shown on the following page.

Consistency	Unconfined Compressive Strength (tons per Sq ft)
Very soft	less than o.25
Soft	0.25 - 0.50
Medium	0.50 - 1.0
Stiff	1.0 - 2.0
Very stiff	2.0 - 4.0
Hard	over 4.0

Besides serving as an indication of the consistency of a soil, the unconfined compressive strength serves as a measure of the shearing resistance. The shearing strength, t, is given by the formula $t = S_n \tan \phi + C$ where S_n is the normal load stress at impending failure, ϕ is the angle of internal friction and C is the cohesion. It has been found by many undrained triaxial shear tests on Winnipeg saturated clays that the angle of internal friction is negligible. Therefore the shearing stress is equal to the cohesion. Mohr's circle diagram (12) shows that the unconfined compressive strength, for clays having ϕ equal to zero, is twice the cohesion.

The bearing capacity of a soil depends primarily on the shearing resistance of the material. The shearing strength also enters into the analysis of the stability of slopes of canals, earthcuts and river banks, the determination of pressures on retaining walls, and the design of subgrades under runways and roads.

The Atterberg Limits

The liquid limit, plastic limit and shrinkage limit determinations were performed at the University of Manitoba according to A. S. T. M. Designation D 423-54T, D 424-54T, and D 427-39 respectively.

The Atterberg Limits and the Plasticity Chart were used in the investigation of the plastic characteristics of the local clays.

Natural Water Content

The natural water content was determined by dividing the weight of water in a soil by the dry weight of the soil. The water content enabled an estimation to be made, when used with the Atterberg Limits, of the compressibility of the clays.

Compression Index, Swelling Pressure, and Preconsolidation Pressure

The consolidation test was performed using a floating ring consolidometer. There were two diameters used, 2.53 inches and 1.49 inches while the height of sample was approximately $\frac{1}{2}$ inch.

The sample was first loaded with a small load and allowed to swell, due to the addition of water, under this small load. The sample was loaded in increments, which were approximately double the previous increment, until a load of approximately 6 tons per square foot was on the large diameter samples and 16 tons per square foot on the small samples.

The void ratio at the end of each load increment was calculated. The pressures and void ratios were plotted to form a curve on semi-logarithmic graph paper. This curve employed can be found in most standard text books on soil mechanics.

The compression index and preconsolidation pressure were obtained by the generally accepted methods (12). The swelling pressure was determined as the pressure required to return the sample to its original volume, that is, to its original void ratio.

Grain Size Analysis

The grain size analysis was determined at the University of Manitoba according to A.S.T.M. Designation D 422-54T. The purpose of this test is primarily for soil identification based on grain size.

Other Properties

Besides the previously discussed properties, the dry and wet densities, the degree of saturation as well as the consolidation characteristics, on undisturbed samples are of practical importance. These will be discussed in the following chapters where they are thought applicable.

BROWN SILTY CLAY

The brown silty clay is generally found immediately below the organic topsoil. This material can be located as deep as 27 feet while in other places it is at the ground surface. It is generally first encountered at a depth of 4 feet. The thickness of the layer may be as high as 12 feet but is generally about 6 feet.

The brown silty clay lies in the zone where seasonal moisture changes occur. Because of its very active clay content the material is subject to large volume changes. In dry periods the material shrinks and cracks and these cracks permit subsequent precipitation to reach the deeper soils rapidly. In wet periods the material swells. The repetition of shrinkage and swelling results in a typical "nugget" structure.

When in a dry condition this material becomes highly fissured and loses its cohesive properties, however, when in a saturated condition it behaves as a cohesive material.

Atterberg Limits

The following values were based on 17 tests. The average liquid limit was 50.1 and the high and low values were 84.7 and 25.0. The plastic limit average value was 22.7 while the values ranged from a high of 37.0 to a low of 12.0. The high and low plasticity index values were 62.6 and 7.6 while the average was 27.4.

The material has medium compressibility, medium plasticity, very low permeability, and medium volume changes with changing moisture content. These properties are indicated by the previously stated limits. These properties indicate that it would provide a fair foundation for very light structures, as long as it retained its moisture.

Degree of Saturation

These values have a wide range. The lower value of 58% occurred near the top of the layer and complete saturation was near the bottom of the layer. The average value of 87% was based on 31 tests.

The smaller degree of saturation revealed an angle of internal friction of only 5 degrees in an undrained triaxial test. This indicated that generally a negligible angle of friction can be assumed for strength calculations.

Unconfined Compressive Strength

On the basis of 35 tests, the brown silty clay exhibited an average strength of 2683 lb per sq ft and high and low values of 7440 lb per sq ft and 969 lb per sq ft respectively.

The lower strength values generally occur near the bottom of the layer since the higher moisture contents make the material more plastic than at the surface.

Th unconfined compressive strengths indicate that the consistency of the silty clay is generally stiff but may range from soft to very stiff.

<u>Consolidation Characteristics</u>

The brown silty clay is of medium compressibility as indicated by the average value of the compression index of 0.230. This value, as well as the high and low of .261 and .154, was based on 4 tests.

The preconsolidation load indicated that the material was normally loaded, since average preconsolidation pressures were found approximately equal to the present overburden pressure. Conclusive evidence cannot be drawn from 4 tests but the tests do give an indication of how the material was loaded.

The swelling pressure, based on 2 tests, had an average value of 1450 lb per sq ft. Since the brown silty clay is the material on which most house basements are supported, basement floors will generally be damaged because of this pressure.

Brown Silty Clay - Summary

	High	Average	Low	Number of Tests
Depth to top of stratum (feet)	12	4	0	164
Depth to bottom of stratum (feet)	27	10	3	164
Moisture content (%)	63.8	33.2	25.0	31
Dry density (lb/cuft)	102	84	62	38
Wet density (lb/cuft)	128	112	101	38
Degree of saturation (%)	100	87	58	31
Unconfined compressive strength (lb/sqft)	7440	2683	969	35
Plastic limit (%)	37.0	22.7	12.0	17
Liquid limit (%)	84.7	50.1	25.0	17
Plasticity index (%)	62.6	27.4	7.6	17
Compression index	0.261	0.230	0.154	4
Swelling pressure (lb/sqft),	1500	1450	1400	2
Preconsolidation pressure (lb/sqft)	1600	1000	800	4

CHAPTER VI

YELLOW SILT

The yellow silt, which is an alluvial deposit, is generally located immediately below the brown silty clay. It varies in thickness from a few inches to three or four feet, but is generally about twelve to eighteen inches thick.

Atterberg Limits

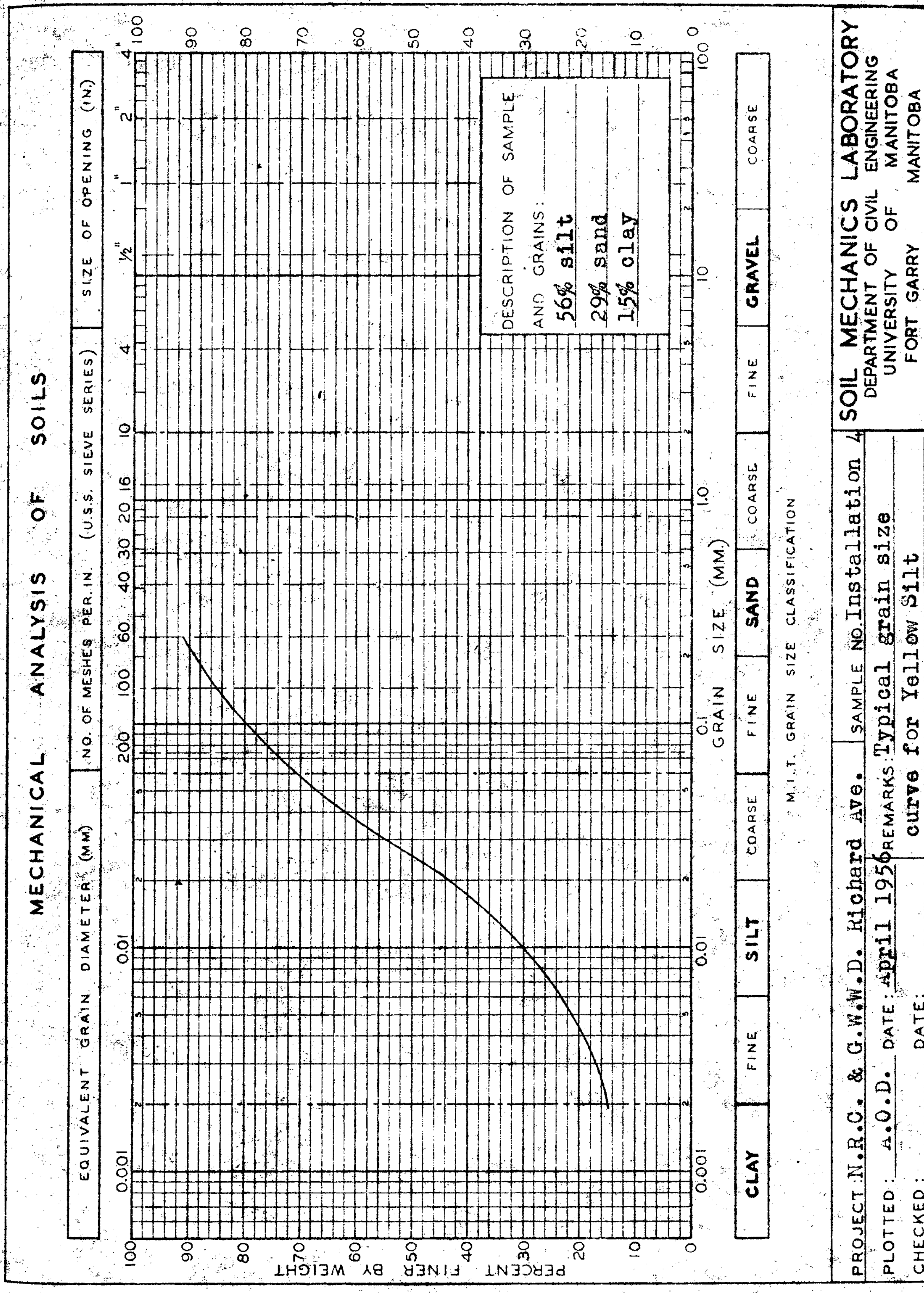
The results of three tests yielded an average value of the liquid limit of 28.4. The average plastic limit and plasticity index were 16.2 and 12.2 respectively.

The yellow silt, on the basis of these limits, is slightly plastic, and subject to only slight volume changes with accompanying moisture changes. Because of these characteristics the material is locally often misdescribed as "yellow clay". It is believed that the presence of small quantities of montmorillinite accounts for these properties. Grain size analysis have conclusively proved that the material is a silt. A typical grain size curve is shown for the yellow silt in Figure 5.

Degree of Saturation

The average value based on 5 tests was 92.7% and the high and low values were 100% and 73% respectively. The lower values occurred near the surface of the material while complete saturation generally occurred at the bottom of the layer.

Figure 5


MECHANICAL ANALYSIS OF SOILS
EQUIVALENT GRAIN DIAMETER (MM)
NO. OF MESHES PER. IN. (U.S.S. SIEVE SERIES)
SIZE OF OPENING (IN.)
PERCENT FINER BY WEIGHT
GRAIN SIZE (MM)
DESCRIPTION OF SAMPLE AND GRAINS:
56% silt
29% sand
15% clay
CLAY
FINE
SILT
COARSE
FINE
SAND
COARSE
FINE
GRAVEL
COARSE
M.I.T. GRAIN SIZE CLASSIFICATION
PROJECT N.R.C. & G.W.W.D. Richard Ave.
SAMPLE NO. Installation 4
PLOTTED: A.O.D. DATE: April 1956
REMARKS: Typical grain size curve for Yellow Silt
CHECKED: DATE:
SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

Unconfined Compressive Strength

The average, based on three compression tests, was 1652 lb per sq ft ranging from high and low values of 2715 lb per sq ft and 1010 lb per sq ft respectively. This indicates that the yellow silt ranges from soft to stiff consistency.

The settlement of foundations, that lie in or effect this material, is believed to be caused by the lateral displacement of the yellow silt from under the footing. This would be a result of the relatively low strength of the yellow silt.

Consolidation Characteristics

The average compression index, based on four tests, was 0.125, while the range was from 0.37 to 0.075. This indicates a material of low compressibility.

Based on four tests, the average preconsolidation pressure was 825 lb per sq ft. The high and low values were 1400 lb per sq ft and 250 lb per sq ft. The comparable depth of material over the location where the samples were taken gives an indication that the material was normally consolidated.

No swelling pressures were obtained on two samples. The other two samples had swelling pressures of 1200 lb per sq ft and 550 lb per sq ft.

Yellow Silt - Summary

	High	Average	Low	Number of Tests
Depth to top of stratum (feet)	16	10	0	159
Depth to bottom of stratum (feet)	19	11	3	159
Moisture content (%)	46.2	26.8	15.1	5
Dry density (lb/cuft)	101.0	93.3	75.0	5
Wet density (lb/cuft)	126.0	119.3	108.0	5
Degree of saturation (%)	100.0	92.7	73.0	5
Unconfined compressive strength (lb/sqft)	2715	1652	1010	3
Plastic limit (%)	18.6	16.2	12.1	3
Liquid limit (%)	31.8	28.4	26.0	3
Plasticity index (%)	19.7	12.2	7.4	3
Compression index	0.370	0.125	0.075	4
Swelling pressure (lb/sqft)	1200	550	0	4
Preconsolidation Pressure (lb/sqft)	1400	825	250	2

BROWN CLAY

The brown clay found in the Greater Winnipeg area is of glacial lake origin. It is generally laminated or varved with layers of silty clay, silt or fine sand about one-sixteenth of an inch in thickness. Summer and winter variation in the gradation of the materials deposited in the glacial lake are believed to have caused these varves.

The brown clay, or "chocolate clay" as it is locally called, generally lies below the "yellow silt" stratum but where the silt does not occur it lies below the brown silty clay. The brown clay is generally first encountered about 10 or 12 feet below the ground surface, but has been found at depths as shallow as 2 feet and as deep as 18 feet. The thickness of the material varies greatly. In some localities it is not present while in other sections it reaches a thickness of 25 feet. Generally, the thickness of the layer is 15 to 17 feet.

Atterberg Limits

Based on 36 tests, the liquid limit varies from a high value of 116.5 to a low value of 37.6, while the average was 88.6. The average plastic limit was 30.0 and the high and low values were 40.0 and 14.1 respectively. The average plasticity index was 58.7, ranging from a high of 88.4 to a low of 23.5

The Atterberg Limits indicate that the brown clay is highly plastic, highly compressible, practically impervious and subject to large volume changes with accompanying moisture changes. These properties are generally undesirable for satisfactory foundations, as are confirmed by the strength and consolidation tests.

Degree of Saturation

The average degree of saturation based on 73 tests was 97.3%. The majority of the tests revealed complete saturation but some of the tests had values as low as 85.5%. The lower values correspond to samples obtained at shallower depths. The high degree of saturation verified the assumption used in calculations that the sample is completely saturated. Undrained triaxial shear tests have indicated that for nearly saturated or saturated brown clays, the angle of internal friction is negligible.

Unconfined Compressive Strength

Based on 87 tests, the average unconfined compressive strength was 2054 lb per square foot which corresponds to a medium consistency.. The consistency had a range from very stiff, as indicated by the high unconfined compressive strength of 4750 lb per square foot, to soft, corresponding to an unconfined compressive strength of 865 lb per square foot.

There is no definite variation in strength of the brown clay with depth except that the very high strength values occur near the surface of the layer.

Consolidation Characteristics

The compression index based on 11 tests averaged 0.39 with high and low values of 0.60 and 0.26 respectively. The average vlue of 0.39 would indicate a material of fairly high compressibility.

The average preconsolidation load based on 4 tests was 1700 lb per sq ft with the values ranging from 2500 lb per sq foot to 1100 lb per square foot. The 4 tests were performed on samples from a depth of about 15 feet. The average preconsolidation load is approximately equivalent to the present overburden and because of this, any heavy load will cause appreciable settlements.

The swelling pressure of this material can be very high. Studies conducted at the University of Manitoba have revealed swelling pressures of 48,000 lb per square foot, if the material was first oven dried (13). The average swelling pressure obtained from 8 tests was 2150 lb per square foot. The high and low values were 4300 lbs per sq foot and 830 lb per square foot respectively.

It will be noted that the swelling pressure can be in excess of the allowable bearing pressure discussed under "Allowable Bearing Capacities". Footings are therefore subject to possible heaving and ground supported floors, which constitute light loading, will invariably be subject to heaving.

Brown Clay - Summary

	High	Average	Low	Number of Tests
Depth to top of stratum (feet)	16	11	2	147
Depth to bottom of stratum (feet)	40	25	11	147
Moisture content (%)	57.3	48.3	26.7	76
Dry density (lb/cuft)	99.0	76.8	64.5	73
Wet density (lb/cuft)	125.0	109.5	96.5	83
Degree of saturation (%)	100.0	97.3	85.5	73
Unconfined Compressive strength (lb/sqft)	4750	2054	865	87
Plastic limit (%)	40.0	30.0	14.1	36
Liquid limit (%)	110.5	88.6	37.6	36
Plasticity index (%)	88.4	58.7	23.5	36
Compression index	0.600	0.300	0.260	11
Swelling pressure (lb/sqft)	4300	2150	830	8
Preconsolidation pressure (lb/sqft)	2500	1700	1100	4

GREY BROWN CLAY

The grey brown clay, which is usually varved, is a mixture of the overlying brown clay and the underlying grey clay. The material has been found as shallow as 6 feet and as deep as 35 feet and is generally encountered first at a depth of 20 feet. The thickness of the material is usually about 5 feet, although the material is not found at all in some localities.

Atterberg Limits

The results of 9 tests indicated an average liquid limit of 93.2, the range being from 110.0 to 70.0. The plastic limit averaged 30.0, the high and low values being 35.5 and 26.0 respectively. The average plasticity index was 63.1 while the high was 74.5 and the low 51.0.

The limits indicate that the material is highly compressible, highly plastic, practically impervious and subject to large volume changes with accompanying moisture changes. Because this material generally lies below the depth of seasonal moisture variation, excessive volume changes generally do not occur.

Degree of Saturation

The high average value of 98.1% for the degree of saturation indicates that the material is usually found in a saturated condition. This factor generally permits the angle of internal friction to be neglected when designing foundations that affect this material.

Unconfined Compressive Strength

On the basis of 49 tests, the average unconfined compressive strength value was 2169 lb per square foot. This indicates that the material has a stiff consistency, but it may vary from very stiff to soft as indicated by the strength values of 3760 lb per square foot and 1120 lb per square foot.

Consolidation Characteristics

The grey brown clay has fairly high compressibility as indicated by the average compressive index of 0.366. The range of values for 3 tests was from 0.45 to 0.30.

The preconsolidation load was 1440 lb per square foot. These samples were from a depth of about 11 feet, which would indicate that the grey brown clay is generally normally consolidated.

The swelling pressures of this material were about 980 lb per square foot. Although this value is not too great, damage can still occur from basement floor heave.

Grey Brown Clay - Summary

	High	Average	Low	Number of tests
Depth to top of stratum (feet)	28	20	6	176
Depth to bottom of stratum (feet)	35	25	8	176
Moisture content (%)	63.0	56.0	31.4	57
Dry density (lb/cuft)	86.5	69.1	53.0	51
Wet density (lb/cuft)	114.0	103.2	98.0	51
Degree of saturation (%)	100.0	98.1	88.8	50
Unconfined compressive strength (lb/sqft)	3790	2169	1120	49
Plastic limit (%)	35.5	30.1	26.0	9
Liquid limit (%)	110.0	93.2	70.0	9
Plasticity index (%)	74.5	63.1	51.0	9
Compression index	0.450	0.366	0.300	3
Swelling pressure (lb/sqft)	1100	980	800	3
Preconsolidation pressure (lb/sqft)	-	1440	-	1

GREY CLAY

The grey clay is a lacustrine deposit. Because of its grey-bluish color, it has been locally called "blue clay". The material is varved in the same manner as the brown clay.

The grey clay is generally first encountered at a depth of 25 feet, but was located as shallow as 15 feet and as deep as 62 feet. The thickness of the layer is generally about 20 feet, but has been found 46 feet thick.

Atterberg Limits

The liquid limit, based on 17 tests, averaged 75.5, the high and low being 95.0 and 37.0 respectively. The average plastic limit value was 25.1 and the high value was 31.9 while the low was 16.0. The plasticity index ranged from a high of 68.0 to a low of 20.0, the average value being 50.4.

The limits indicate a material of high plasticity, high compressibility, very low permeability, and one that is subject to large volume changes with moisture changes. These properties are similar to the brown clay and are generally undesirable for satisfactory foundations. Satisfactory foundations can be built on this material because it is below the zone of moisture changes.

Unconfined Compressive Strength

The grey clay ranges in consistency from stiff to medium as indicated by the unconfined compressive strengths of 3570 lb per square foot and 1188 lb per square foot. On the basis of 44 tests, the average unconfined compressive strength was 2182 lb per square foot, which indicated that the material is generally of medium consistency.

Degree of Saturation

This material is always found nearly saturated or saturated. The average value, based on 50 tests, was 98.2%, the low was 88.8 while the majority of samples were completely saturated.

This confirms the belief that this material is usually completely saturated and the angle of internal friction may be taken as being negligible for rapid undrained loading.

Consolidation Characteristics

Only one consolidation test was performed on this material. The compressibility of the grey clay was very high as indicated by the compression index of 0.58.

The swelling pressure of the grey clay was 950 lb per square foot. This pressure is generally never significant as, this soil is found at great depth and below the zone of seasonal moisture variations.

No values of the preconsolidation pressure were available. This is probably due to the fact that it is usually quite difficult to obtain from the pressure-void ratio curve.

Grey Clay - Summary

	High	Average	Low	Number of Tests
Depth to top of stratum (feet)	35	25	15	154
Depth to bottom of stratum (feet)	62	45	15	154
Moisture content (%)	61.2	41.2	27.0	44
Dry density (lb/cuft)	102.0	79.4	63.0	39
Wet density (lb/cuft)	130.0	111.6	101.0	42
Degree of saturation (%)	100.0	98.2	88.8	38
Unconfined compressive strength (lb/sqft)	3570	2182	1188	44
Plastic limit (%)	31.9	25.1	16.0	17
Liquid limit (%)	95.0	75.5	37.0	17
Plasticity index (%)	68.0	50.4	20.0	17
Compression index	-	0.580	-	1
Swelling pressure (lb/sqft)	-	950	-	1
Preconsolidation pressure (lb/sqft)	-	-	-	-

"HARDPAN"

"Hardpan" is the term given locally to the cemented rockflour, silt, gravel and sand which covers the limestone bedrock. The thickness of this material is over 33 feet in some places and is not found in other localities. The depth to this material is very shallow in the western portion of the area. The "hardpan" dips downward from this portion and reaches depths of about 60 feet below the surface in the north Main Street area.

Samples were obtained for this material. It was impossible to obtain a good representative sample of the "hardpan" because of the presence of large stones which prevented the sampling tube from being fully driven into the material. The samples did contain enough material to enable some of its physical properties to be determined.

Atterberg Limits

The results of 7 tests revealed an average liquid limit of 17.0, the high and low being 21.6 and 14.2. The average plastic limit was 12.0 while the range was from 14.6 to 10.5. The plasticity index ranged from a high of 7.5 to a low of 2.7, the average being 4.9. These values indicate that the material has very slight compressibility and plasticity.

Degree of Saturation

The average value was 90.0% and ranged from a high of 100.0% to a low of 42.0%. These values were based on eight tests.

Unconfined Compressive Strength

Very high strengths were generally encountered as indicated by the average value of 7950 lb per sq ft. The strength was as high as 15,125 lb per sq ft and as low as 3570 lb per sq ft. These values were based on eight tests.

It must be noted here that the "hardpan" material which was tested, was from outside of the Greater Winnipeg area. The majority of the values were obtained from a soil profile for the proposed Greater Winnipeg Floodway which lies just east of the Greater Winnipeg area. Although the material was from outside the area covered in this thesis, it is believed that the discrepency between these physical properties and those of the "hardpan" material in the Greater Winnipeg area is small.

The non-cemented gravel, sand, silt and rockflour which generally overlies the "hardpan" revealed very small unconfined compressive strengths, the average of 2 tests being 937 lb per sq ft. This low strength would indicate that the material would have to be displaced when piles are end bearing on the "hardpan".

"Hardpan" - Summary

	High	Average	Low	Number of Tests
Depth to top of stratum (feet)	60	47	8	188
Degree of saturation (%)	100.0	90.0	42.0	8
Unconfined compressive strength (lb/sqft)	15,125	7,950	3,570	8
Plastic limit (%)	14.6	12.0	10.5	7
Liquid limit (%)	21.6	17.0	14.2	7
Plasticity index (%)	7.5	4.9	2.7	7

ALLOWABLE BEARING CAPACITIES

The ultimate bearing capacity of a soil is given by Terzaghi (14) as:

$$q_{ult} = cN_c + wD_f(N_q - 1) + \frac{wBN_w}{2} + wD_f$$

where q_{ult} = ultimate bearing capacity

c = cohesion

w = unit weight of soil

B = width of footing

For long continuous footings N_c, N_q, and N_w are dimensionless coefficients dependent on the angle of internal friction. Their values are given in most modern text books on soil mechanics and foundations.

When the angle of internal friction is negligible, as is generally the case for Greater Winnipeg soils, N_q becomes unity and N_w equals zero. The equation then becomes:

$$q_{ult} = cN_c + wD_f$$

Later work by Skempton (15) gives the following formula for N_c, for rectangular footings.

$$N_c = 5\left(1 + \frac{B}{5L}\right)\left(1 + \frac{D_f}{5B}\right)$$

where L = length of the footing

In determining the area required for a footing it is convenient to use for the basis of computation, the net soil pressure. This is the pressure at the base of the footing in excess of that at the same level due to the surrounding surcharge and can be expressed by the following equation:

$$q_d = q_{ult} - wD_f$$

where q_d = net bearing pressure

The net bearing pressure is then the total load above the base of the footing including the dead load, live load, the weight of the footing and the weight of the soil above the footing less wD_f, where D_f is as shown in Figure 6.

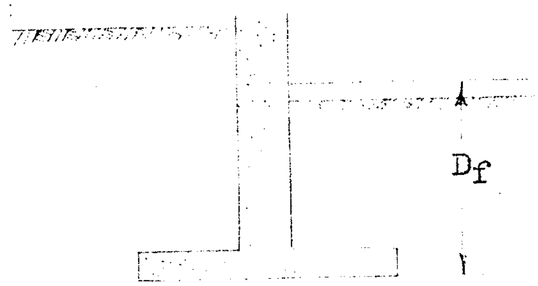


Figure 6

The calculated allowable bearing capacity for each layer of material, when not underlain by weaker material, is shown in Figure 7. These values were arrived at by using a value for N_c of 5.7 which is the value given by Terzaghi for long footings, provided the material is saturated and has no angle of internal friction. This bearing value is conservative for square footings.

The general equation then reduced to:

$$q_{ult} = \frac{5.7c}{S.F.} + wD_f = \frac{2.85\ q_u}{S.F.} + wD_f$$

For the purpose of easy tabulation the factor wD_f was neglected. It sould be considered for the design of deep spread footings and raft foundations.

A factor of safety of 3.0 is usually recommended against a bearing capacity failure. It has also been found that this safety factor limits total settlements to one inch and differential settlements to three quarters of an inch on cohesive soils. If the subsurface conditions are well known or uniform, a safety factor of 2.0 may be used.

The City of Winnipeg Building Code (16) gives the permissible loads per sq ft to which the different natural soils may be subjected. These values are shown in Figure 8. The Building Code is typical of older codes insofar as it does not take into account the shape of the footing or the effects of the depth at which the footing is located.

The grey clay is referred to in the Building Code as "blue clay" and the grey brown clay is called mixed clay. The brown silty clay and yellow silt are included under the soft wet clay or silt group. A definite limitation of the Building Code is that no mention is made of the brown clay.

CALCULATED ALLOWABLE BEARING CAPACITY

Material	Unconfined Compressive Strength (lb per sq ft)			Allowable Bearing Capacity (lb per sq ft) S.F. = 2			S.F. = 3		
	High	Avg.	Low	High	Avg.	Low	High	Avg.	Low
Brown Silty Clay	7440	2683	969	10600	3790	1370	7050	2550	920
Yellow Silt	2715	1652	1010	3880	2354	1440	2580	1570	970
Brown Clay	4750	2054	865	6760	2926	1230	4510	1950	822
Grey Brown Clay	3790	2169	1120	5400	3090	1596	3600	2062	1062
Grey Clay	3570	2182	1188	5090	3110	1692	3390	2075	1128

Figure 7

WINNIPEG BUILDING CODE ALLOWABLE BEARING PRESSURES

Column A is for industrial and commercial buildings.

Column B is for buildings for human habitation.

Material	A	B
Blue clay with no underlying strata of yellow or brown clay	4000	3000
Mixed clay, moderately dry	2000	1500
Soft wet clay or silt	1000	750

Figure 8

A comparison of the Building Code allowable bearing pressures and the calculated allowable bearing pressures is possible since the strength values obtained for this study are from a good representative selection of the Greater Winnipeg soils.

The allowable bearing pressures for the soft wet clay or silt, as given by the Building Code, are 1000 lb per sq ft and 750 lb per sq ft. These values indicate a safety factor of well over 3.0 is present when dealing with the average bearing pressures of the brown silty clay. The yellow silt has a safety factor slightly over 3.0, which more than satisfies the requirements for safe design.

The Building Code values for the grey brown clay are 2000 lb per sq ft and 1500 lb per sq ft. The 2000 lb per sq ft is comparable to a safety factor of 3.0 while the 1500 lb per sq ft has a safety factor over 3.0. Both of these values are over the generally required safety factor of 3.0.

The grey clay allowable bearing pressures do not agree favorably with the Building Code values. The bearing pressure, as given by the Building Code, of 3000 lb per sq ft has only a safety factor of 2.0 while the 4000 lb per sq ft value has a safety factor less than 2.0.

This would indicate that bearing values lower than those given by the Building Code for the grey clay should be used for design.

Average strengths were used for the calculated bearing pressures. Their use is partly justified for the following reason. It is legitimate to use the average strength of the materials for a depth equal to the width of the footing below the foundation. In the Greater Winnipeg area, generally no layer contains all weak material but rather material whose strength varies with depth.

Although the average strengths were used for comparison, it should be realized that deviations from the average were quite extreme in some areas. In such cases, average values can be in error when particularly weak conditions are encountered. For example, in comparing the low calculated allowable bearing pressure with the allowable pressures given by the Building Code for the grey clay, the 4000 lb per sq ft value has a safety factor of 0.85 and the 3000 lb per sq ft has a safety factor of 1.14. It is likely that square or round footings would be used at this depth rather than long footings and this would increase the safety factor. Terzaghi states that the increase would be 1.3 times as great, resulting in safety factors of 1.10 and 1.48. The grey brown clay has a safety factor of approximately 2.0 for the Building Code values of 2000 lb per sq ft and 1500 lb per sq ft and the brown silty clay and yellow silt values from the

Building Code have safety factors ranging from 2.0 to 3.0. The values given for the grey brown clay, brown silty clay and yellow silt, are for long footings only.

It may be concluded that the Building Code allowable bearing pressures are generally conservative, except for the values given for the grey clay.

The recent practice used to obtain the bearing pressure beneath a footing is to use the net bearing pressures. In this method the term "wD_f" can have marked effects on the design bearing pressure. For shallow spread footings this factor is negligible. For a mattress or raft foundation it cannot be neglected as it is quite appreciable. Theoretically it is possible to have this type of foundation "floating" in the soil by having enough soil displaced by the foundation so that the weight of displaced soil is equal to the weight of the structure.

In 1913 the bearing capacity failure of the Transcona Grain Elevator took place (2). Transcona is seven miles north-east of Winnipeg and is situated in the Lake Agassiz basin. The soil conditions at this site are very similar to those of the Greater Winnipeg area. This failure significantly substantiates the theoretical bearing values obtained from the soil tests.

The structure consisted of a reinforced concrete workhouse and an adjoining bin house. The workhouse was

70 feet by 96 feet in plan, and 180 feet in height with a raft foundation at a depth of about 12 feet. The bin house contained five rows of 13 bins, 92 feet in height and 14 feet in diameter, resting on a reinforced concrete raft 77 feet wide and 196 feet long, and was at a depth of 12 feet below the surrounding ground. The design bearing pressure was 6600 lb per sq ft based on a load bearing test and the dead load was 20,000 tons.

The construction of the elevator began in 1911 and it was ready for filling in September 1913. The storage of the grain was started, with considerable care taken to distribute the grain uniformly. On October 18, when 875,000 bushels of wheat were stored, a vertical settlement of one foot was noted within an hour. Within 24 hours the structure was resting at an angle of 26°53' from the vertical. The west side of the building was 24 feet below its original position and the east side had risen 5 feet above its original elevation.

Calculations based on the dead weight of 20,000 tons, and 875,000 bushels of wheat at 60 pounds per bushel, gave a unit uniformly distributed pressure of 6,200 lb per sq ft on the clay when failure took place.

By using Terzaghi's general equation for the bearing capacity of a saturated clay with an angle of internal friction equal to zero, and also Skempton's equation for N_c, we get:

$$q_{ult} = 5.56\ c + 12w$$

Work done at the University of Manitoba indicated that the unconfined compressive strength of the material at the site was 1850 lb per sq ft. This value along with a value of w equal to 107 lb per cu ft resulted in a calculated ultimate bearing capacity of 6420 lb per sq ft. This value is remarkably close to the actual bearing capacity at failure of 6200 lb per sq ft.

It is reassuring that the study of the Transcona Grain Elevator bearing capacity failure verified the commonly accepted theories on this type of failure.

Piles are another means of supporting heavier structures. In Greater Winnipeg cast-in-place concrete piles or piers are generally used. The Winnipeg Building Code states that when a pier stops at "hard pan" it may be belled out to twice its diameter at an angle of thirty degrees from the vertical, giving a maximum bearing of seven and one half tons per sq ft.

On the basis of the average unconfined compressive strength and an assumed angle of internal friction of 25°, the average cohesion was computed. Using an average depth of 40 feet and the computed cohesion, the calculated allowable bearing capacity was 23.4 tons per sq ft. This value is based on a safety factor of 3.0. The calculations for the bearing capacity are shown in Appendix A. The use of the average unconfined compressive strength is justified since the lower strengths were obtained from damaged samples of the "hardpan".

By comparing the Building Code value with this calculated value, the Building Code value has a safety factor of approximately 6.0 which is apparently very conservative. A high safety factor is however very desirable since the pile can rest on the unconsolidated "hardpan".

Locally, for a saturated clay with no angle of internal friction, the following formula is used for the safe unit frictional support for friction piles.

$$f_s = \frac{qu}{6}$$

This equation is based on a safety factor of 3.0.

In Greater Winnipeg, the frictional resistance of the top 10 feet of soil should be neglected since this is the depth that is generally affected by seasonal moisture changes. This would in most cases be the brown silty clay and the yellow silt. The average unconfined compressive strengths of the underlying material are 2100 lb per sq ft. This would result in a frictional support of 350 lb per sq ft of pile surface area. A value, that is not in the Building Code, of 300 lb per sq ft is commonly used.

TOTAL SETTLEMENTS

In greater Winnipeg the largest part of total settlement is due to the plastic compression or consolidation of the soil. The relative compressibility of the different local soils may be compared by their compression indices. An estimate can be made of the total settlement by using the compression index and the increase in pressure due to any additional load. The following table shows the relative compressibility of the different local materials.

Material	Compression Index
Brown Silty Clay	0.230
Yellow Silt	0.125
Brown Clay	0.390
Grey Brown Clay	0.366
Grey Clay	0.580

The compression index of 0.125 for the yellow silt is locally considered low and the compression index of 0.580 for the grey clay is considered high.

About 1950 the local soils were in a state of drying because of an unusual number of dry years. As far back as 1937 there was evidence of dry spells. The drying out of the soils caused abnormal reductions in their volumes and hence abnormal settlements of structures. The shallow foundations were the most affected by these volume reductions since they were located in the zone of maximum drying.

Underpinning of many of the shallow spread footings to rectify these settlements and to prevent further settlements became a major industry in the Greater Winnipeg area.

After the 1950 flood, which covered a large part of Greater Winnipeg, and which was preceded and followed by unusually wet seasons, the soils had regained some of their lost moisture. The addition of this moisture to the soils has eliminated, to some extent, the settlement caused by the drying out of the soils and in many cases has caused vertical movements because of the swelling action of the soils.

Factors, other than those mentioned, can contribute to total settlement. The elastic compression of the local soils when loaded causes instantaneous settlement, but the settlement is recoverable if the load is removed. This settlement is usually small. Settlement is also caused by the lateral movement of the soil, but this is also generally small.

DIFFERENTIAL MOVEMENTS

It is considered that the two principal causes of differential movements in the Greater Winnipeg area are the changes in moisture content of the soil and footing size variation.

The change in moisture content of a soil results in shrinkage and swelling of the soil. This effect is particularly noticeable in the movement of shallow foundations. The changes in moisture content can occur in localized areas causing differential swelling or contraction. In house foundations these differential movements can prove quite damaging.

Immediately after the 1950 flood, many basement floors had heaved and cracked excessively. The floors were generally of plain concrete, 3 or 4 inches thick. The cause of this damage was directly attributed to the swelling of the clays. The amount of heave occurring in the foundation was maximum under the lightly loaded basement floors and minimum under the heavily loaded basement walls. In some cases the swelling pressures had been large enough to lift the basement floor off of the wall footing.

It must be remembered that footings with the same unit bearing pressure affect the underlying soils to different degrees if they are not the same size. Small footing effects are relatively shallow whereas large footing effects are much deeper with the result that greater settlements occur under large footings. The distribution of pressure beneath a footing can be determined by Newmark's or Westergaard's tables. Westergaard's table is more applicable because the stratified condition upon which the table is based is nearer to the conditions existing in sedimentary soils than the isotropic condition upon which Newmark's table is based.[7] With the pressures known acting on each layer, an estimate can be made of the settlement since the average compressibility of each of the local layers is known.

An estimate has been made of the settlements of two different sized footings with the same unit loading on an average Greater Winnipeg soil profile. The difference in settlement should be noted. The solutions appear in Appendix B.

Other factors which have been observed to have resulted in differential movements in Greater Winnipeg are:

1. Placing footings too close to each other
2. Differences in live load and dead load on different footings in the same structure
3. Variations in soil profile
4. Footings at different elevations
5. Different types of foundations for one structure

These generally reflect poor design and are therefore not given further consideration.

Differential movements of shallow footings may be treated in several ways. When a rigid foundation is constructed as a unit, and subjected to differential movements, the whole structure will rotate as a unit which prevents any damage to the superstructure. This method is being employed on smaller buildings by using steel beams to support the first floor and reinforcing the basement walls and footings. Another method being used to reduce differential movements is to place the footings at a depth where moisture changes do not occur. This is done by using deep spread footings, driven wooden piles or cast in place concrete piles. Basement floor movement has been dealt with by using beam and slab construction for the floors. The movement of any material under the slab has no effect on the floor, provided of course that there is a clear space between the soil and the slab.

EFFECTS OF LATERAL EARTH PRESSURES

Structures such as retaining walls, trench and excavation bracing, bulkheads, cofferdams and basement walls are used locally to support soil masses in a fixed position. A safe assumption for lateral earth pressures generally encountered in Greater Winnipeg employ the "at rest" case. In this state the horizontal pressure S_h, is given by the following formula;

$$S_h = K S_v$$

where K is the coefficient of lateral earth pressure and S_v is the vertical earth pressure which is equal to w H, where H is the distance from the ground surface to the point considered. The distribution of this pressure, against a wall is hydrostatic. The resultant, which acts at 2/3 of the depth that the footing for the wall is placed, would be $\frac{KS_vH}{2}$ or $\frac{KwH^2}{2}$. For local clays K has the high value of unity and the resultant would then be:

$$P_h = \frac{w\,H^2}{2} \quad \text{at a depth } \frac{2H}{3}$$

This pressure can be of sufficient magnitude to damage basement walls. For this reason hollow concrete block basement walls are not allowed in the City of Winnipeg. In houses that did have this type of basement wall, inward bulging, due to the lateral earth pressure, has been observed. Investigations of some walls of this type of construction have shown that the amount of movement can be 2 inches at the mid-height of the wall and is less toward the floor and corners. The use of concrete walls practically eliminates this type of movement and this is one of the reasons why they are used in Winnipeg.

Many older houses were constructed without basements, getting their support from shallow spread footings. At some time since construction partial basements were excavated under the houses and the sides were supported by timber, concrete, brick or stone walls. The masonary walls were poorly designed to withstand horizontal pressures created by the soils. After the 1950 flood many of these walls caved in which indicated that the walls served their purpose as long as the soils were dry but after saturation larger lateral support was required. At the present time in Greater Winnipeg basementless houses are being constructed but no provision is made for later basement excavation. All the facilities generally used in a basement are being placed, generally, on the first floor.

Central Mortgage and Housing, Winnipeg area, specify that the minimum width of footing beneath a basement wall must be 30 inches. The reason for this specification is that the shearing resistance of the clay over this width is sufficient to prevent the wall from sliding due to the lateral earth pressures. A check on the safety of this specification is given in Appendix C.

The lateral earth pressure must be a major consideration in designing retaining walls and abutments. The abutment must support the end of a bridge span and provide at least some lateral support for soil upon which the roadway rests immediately adjacent to the bridge. Several special designs have been used so that the earth pressures do not fully act on the abutment. A "U" abutment permits the approach fill to be sloped towards the front of the abutment. The soil can be sloped so that the lateral earth pressure does not endanger the abutment but is still confined within it. A spill-through abutment is used on some of the railroad bridges in the Greater Winnipeg area. It consists of two or more vertical columns carrying a beam that supports the bridge seat. The fill extends on its natural slope from the bottom of the beam through the openings between the columns, thus eliminating a large percentage of the lateral soil pressure. The importance of the effects of lateral earth pressures was brought out when these pressures caused the abutments of the Osborne Street Bridge to move. The bridge went into compression and underpinning was necessary to relieve the stresses caused by this movement.

FROST

The depth of frost penetration in the Greater Winnipeg area is generally 4 or 5 feet in snow covered areas. Since the silty clays and silts, which are frost susceptible, generally lie in the zone of frost penetration, frost action can occur. The frost action is generally not too severe in regards to building foundations since most of the foundations are below the depth of frost penetration. Shallow footings of houses can be damaged if they are not below the depth of frost penetration. Most house foundations are now being placed below the depth of frost penetration to reduce the effects of frost action. The new type of basementless houses are designed so that the flow of heat from the building into the soil will prevent frost penetration below the building.

Some damage to the roads and streets in Greater Winnipeg has been attributed to frost action. The damage is a result of an ice lense melting in the spring and leaving the soil very plastic. The application of any load on this area can cause deformation of the pavement and eventually a surface break. Frost action under the roads is not the major contributing cause since it is generally felt that the swelling and shrinking of the soils, with changing moisture contents is the most damaging factor. Since the frost action is a relatively minor cause of damaged roads no special protection is made to prevent it.

Most sewers and watermains have been placed below the depth of frost penetration. The primary reason for this is to prevent them from freezing. Due to the depth at which the pipes are placed there is little possibility that they will break due to frost heave.

RIVER BANK STABILITY AND SUBSIDENCE

Construction on the banks of the Red and Assiniboine Rivers is extremely dangerous because of the doubtfulness of the stability of the banks. Many slides have occurred and the dangerous situation that could occur with construction on the river banks was recognized by the Rivers and Streams Control Authority No. 1 after the 1950 flood. The result was a law which prohibits any construction to be made within 150 feet of the summer high level of the rivers unless approved by a qualified engineer.

The stability of the river banks can be very dangerous during the spring and fall months. The reason for this situation is that during these periods the river level drops, resulting in a rapid drawdown state of the river banks.

Flooding in the spring saturates the river banks. The flooding is a result of the spring thaw and adverse climatic conditions. Usually an early cold winter sets in with little snow at first so that the ice on rivers and lakes will be abnormally thick. This is followed by a heavy snowfall and continued cold weather. Then a late cold spring occurs with the result that little snow escapes into the rivers. Generally in May a sudden change of wind to the south comes so that the whole mass of snow melts at once, putting its volume of water into every stream. Any further precipitation places more water in the streams and hence more danger of flood. The inflow of all this water into the main rivers raises the water levels until water levels until the rivers overflow their banks. The soil then becomes saturated if it has not already been saturated by infiltration. When the river level drops, the banks remain saturated because the soils are too impervious to allow free flow from the banks. This condition of saturated river banks also occurs in the fall when the water level is lowered for the winter by the control gates of the Red River at Lockport which is approximately 20 miles north of Winnipeg.

This rapid drawdown state is the worst possible condition to which the bank may be subjected. The additional weight of water increases the forces which tend to cause sliding. It is during this state that most river bank slides occur.

The stability of the banks can be made worse by the addition of a surcharge to the already saturated bank. In Greater Winnipeg temporary dykes are built to prevent local flooding. These dykes constitute an additional load on the bank. At high water level the additional load does not have much effect on the stability of the bank because submerged banks are stable since the weight of water over the slope of the bank resists the forces tending to cause movement. When the river level drops, the load of the temporary dykes can have detrimental effects on the stability of the banks. For this reason temporary dykes are generally removed as soon as possible.

In a recent report by a local firm of consulting engineers it was stated that "most of the river banks in the Greater Winnipeg area are in a state of delicate balance with a factor of safety below what would normally be accepted, making some allowance for unfavorable conditions which might occur in the future".

Precautions should be taken to investigate the bank stability if a structure is proposed near a river bank. In such analyses, the angle of internal friction equal to zero case is generally applicable.

CHAPTER XVII

CHEMICAL DETERIORATION OF CONCRETE

High concentrations in the order of 4000 ppm of soluble sulphates have been measured in the soils of Greater Winnipeg. The predominant sulphates are those of calcium, magnesium and sodium. These sulphates are believed to act on the tricalcium aluminate in the cement promoting the formation of calcium sulpho aluminate. This compound occupies a larger volume than the original tricalcium aluminate and as a consequence splitting as well as leaching of the concrete takes place. This action has been measured to a depth of 2 inches in concrete that had been in the ground about 10 years. This deterioration seriously weakens foundations and other concrete works in contact with the local soils.

In 1919, it was discovered that the recently completed 96 mile concrete aqueduct servicing the Greater Winnipeg Water District was being attached by these sulphates. The investigation of this action was instrumental in the development of alkali-resistant Type 5 cement by T. Thorvaldson of the University of Saskatchewan and the late A. Fleming of the Canada Cement Company. The Type 5 cement, or Kalicrete as it is called in Canada, contains a much smaller amount of tricalcium aluminate than the standard Portland cement with the result that it has a high resistance to alkali. It was also found that steam cured concrete with ordinary cement was also not subject to sulphate attack.

The use of alkali-resistant cement should be limited to areas of high sulphate content. It has been stated that if the sulphate concentration exceeds 350 ppm, deterioration of concrete will take place in the Greater Winnipeg area. There is a great variation in the soluble sulphates. As an example, on River Ave. over a distance of approximately one mile the concentration ranged from 0.4% to 3.1%. This factor would indicate that sulphate tests are a necessary prerequisite to determine if alkali-resistant cement should be used.

Not only are the soluble sulphates detrimental to concrete but they are also detrimental to metal. This is one of the reasons why steel piles are not used in this area. Corrosion of buried cast iron pipe has caused failure of watermains in as short a period as 10 years. To alleviate these watermain failures cement-asbestos pipe and steam cured concrete pipe have been used to resist attack from the soluble sulphates.

CONCLUSIONS

As can be seen from the previous discussions, the foundation problems in the Greater Winnipeg area are many. These problems can generally be attributed to the active nature of the soils, that is, the soil conditions change with changes in moisture content.

Deep foundations generally offer no serious problem since they are below the depth 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" higher bearing values can be used and hence the foundations can support larger loads. If the foundations are founded in the grey or grey brown clay the problem of adequate bearing capacity must be answered. Generally this is a simple design problem. With foundations in this material settlements can be appreciable but are generally smaller than the settlements that would be encountered if the foundation was at a shallower depth. The major drawback of deep foundations is the cost. If an expensive structure is to be constructed it would be worth the added expense to place the structure on deep foundations, since less trouble would be had with future maintenance.

Shallow foundations are the most affected foundations in the Greater Winnipeg area. These foundations are generally located in the zone where seasonal moisture changes occur with the result that they are affected by the shrinking and swelling of the soils. Adequate bearing capacity is another problem that affects the design of shallow foundations since the foundations generally affect the relatively weak layer of yellow silt. Settlements of shallow footings are sometimes excessive since a large depth of compressible material can be affected. Numerous methods have been attempted to construct shallow foundations to minimize the effects of these soil conditions. Some of these methods have proven to be very effective but the problem of building shallow foundations generally does not have a simple solution. Each foundation must be treated individually.

Lateral earth pressure in the Greater Winnipeg area have caused basement walls and retaining walls to crack. This result can be partly attributed to the large value of the coefficient of lateral earth pressure which is generally encountered in Greater Winnipeg soils. The design of structures, which retain the soil, usually employs the "at rest" condition.

The problem of frost affecting foundations is being met by placing foundations below the depth of frost penetration. As a result of this foundation location, frost action has become a relatively minor consideration in foundation design.

Any type of construction on a river bank in the Greater Winnipeg area should not be undertaken without the advice of a qualified engineer. The stability of the banks is in a state of delicate balance and unless preliminary investigations are carried out some of the construction is liable to be involved in unexpected slip failures with disasterous results.

The chemical deterioration of concrete in the Greater Winnipeg area has been combatted with the use of sulphate-resistant cements. It is generally advisable to determine the sulphate concentration of the soil to find out if the use of this special type of cement is warranted.

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Appendix A

Calculation of Bearing Capacity of Hardpan

Data

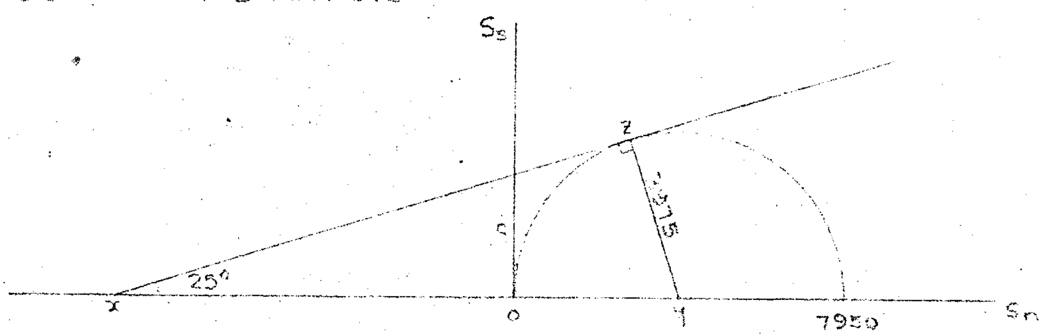
Average unconfined compressive strength = 7950 lb/sq ft

Assumed angle of internal friction = 25°

Depth to hardpan = 40 ft.

Computation

Use Mohr's circle



$$xy = \frac{3975}{\sin 25°} = 9425 \text{ lb/sq ft}$$

$$xo = 9425 - 3975 = 5450 \text{ lb/sq ft}$$

$$c = 5450 \tan 25° = 2540 \text{ lb/sq ft}$$

The bearing capacity is given by

$$Q_p = \pi r^2 (1.3\, c\, N_c + \gamma D_f N_q + 0.6\, \gamma r N_\gamma)$$

For $\phi = 25°$

$$N_c = 25, \quad N_q = 13, \ \& \ N_\gamma = 10$$

Assume $\gamma = 110$ lb per cu ft

$$Q_p = 1\left[(1.3)(2540)(25) + (110)(40)(13) + (0.6)(110)(.565)(10)\right]$$

$$= 82{,}500 + 57{,}200 + 370$$

$$= 140{,}070 \text{ lb/sq ft}$$

$$= 70 \text{ ton/sq ft}$$

With a safety factor of 3

$$Q_p = \frac{70}{3} = \underline{23.4 \text{ ton/sq ft.}}$$

Appendix B

Computation of Settlements of Two Footings

Data

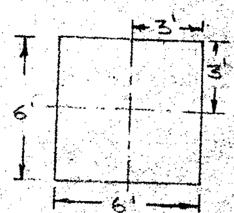
Assume an average soil profile and a unit loading of 1000 lb/sqft on a 6'x6' footing and a 4'x4' footing.

Computation

D		γ	C_c	e
0				
4	A	100		
		112	.230	1.15
10	B			
11	C	119	.125	0.89
		109.5	.390	1.26
23	D			
		108.2	.366	1.32
28	E			
		111.6	.580	1.28
45	F			

Assume footing at 4' depth.

For the 6'x6' footing



For stress at B.

$\frac{m}{z} = \frac{n}{z} = \frac{3}{6} = 0.50$

$R = .055$ (From Westergaard's tables)

$\therefore \sigma_{V_B} = 4(1 \times .055) = \underline{0.22\ K/sqft}$

$\sigma_{V_A} = \underline{1\ K/sqft}$

Average pressure at mid height $= \frac{1.22}{2} = \underline{\underline{0.61}}\ K/sqft$

For stress at C

$\frac{m}{z} = \frac{3}{7} = 0.43$

$\therefore R = .044$

$\sigma_{V_C} = 4(1 \times .044) = \underline{0.176}\ K/sqft$

Average pressure at mid height $= \frac{0.396}{2} = \underline{\underline{0.198}}\ K/sqft$

For stress at D

$\frac{m}{z} = \frac{3}{19} = 0.158$

$\therefore R = .0078$

$\sigma_{V_D} = 4(1 \times .0078) = \underline{.031}\ K/sqft$

Average pressure at mid height $= \frac{0.207}{2} = \underline{\underline{0.103}}\ K/sqft$

For stress at E

$\frac{m}{z} = \frac{3}{24} = 0.125$

$\therefore R = 0.0048$

$\sigma_{V_E} = 4(1 \times .0048) = \underline{0.019}\ K/sqft$

Average pressure at mid height $= \frac{0.050}{2} = \underline{0.025}\ K/sqft$

The increase in pressure on the grey clay is negligible.

Appendix B

Summary of pressures at mid height of each stratum due to loading

A-B $\Delta p = 0.610$ K/sqft

B-C $\Delta p = 0.198$ K/sqft

C-D $\Delta p = 0.103$ K/sqft

D-E $\Delta p = 0.025$ K/sqft

Pressures at mid height before loading:

A-B $P_o = \dfrac{4 \times 100 + 3 \times 112}{1000} = 0.736$ K/sqft

B-C $P_o = \dfrac{4 \times 100 + 6 \times 112 + 0.5 \times 119}{1000} = 1.132$ K/sqft

C-D $P_o = \dfrac{4 \times 100 + 6 \times 112 + 1 \times 119 \times 6 \times 109.5}{1000} = 1.848$ K/sqft

D-E $P_o = \dfrac{4 \times 100 + 6 \times 112 + 119 + 12 \times 109.5 + 25 \times 108.2}{1000} = 2.776$ K/sqft

Final pressure at mid height after loading:

A-B $P_f = 0.736 + 0.610 = 1.346$

B-C $P_f = 1.132 + 0.198 = 1.330$

C-D $P_f = 1.848 + 0.103 = 1.951$

D-E $P_f = 2.776 + 0.025 = 2.801$

$\Delta e_{AB} = 0.230 \log \dfrac{1.346}{0.736} = .060$

$\Delta e_{BC} = 0.125 \log \dfrac{1.330}{1.132} = .0087$

$\Delta e_{CD} = 0.390 \log \dfrac{1.951}{1.848} = .0093$

$\Delta e_{DE} = 0.336 \log \dfrac{2.801}{2.776} = .0017$

	e	$1+e$	Δe	H (in)	$\Delta H = \frac{H}{1+e} \times \Delta e$
A-B	1.15	2.15	.060	72	2.410
B-C	0.89	1.89	.0087	12	0.055
C-D	1.26	2.26	.0093	144	0.592
D-E	1.32	2.32	.0017	60	0.044

total settlement = **3.101 inches**

Appendix B

For the 4'x 4' footing

$\sigma_{VA} = \underline{1}$ K/sqft

For stress at B

$\frac{m}{z} = \frac{n}{z} = \frac{2}{6} = .333 \quad \therefore \quad R = .028$

$\sigma_{VB} = 4(1 \times .028) = \underline{0.112}$ K/sqft

Average pressure at mid height $= \frac{1.112}{2} = \underline{0.556}$ K/sqft.

For stress at C

$\frac{m}{z} = \frac{2}{7} = .286 \quad \therefore \quad R = .022$

$\sigma_{VC} = 4(1 \times .022) = \underline{0.088}$ K/sqft

Average pressure at mid height $= \frac{0.200}{2} = \underline{0.100}$ K/sqft

For stress at D

$\frac{m}{z} = \frac{2}{19} = .105 \quad \therefore \quad R = .003$

$\sigma_{VD} = 4(1 \times .003) = \underline{0.012}$ K/sqft

Average pressure at mid height $= \frac{0.100}{2} = \underline{0.050}$ K/sqft

Layer	Δp	P_o	$P_f = P_o + \Delta p$
A-B	0.556	0.736	1.292
B-C	0.100	1.132	1.232
C-D	0.050	1.848	1.898

$\Delta e_{AB} = 0.230 \log \frac{1.292}{0.736} = 0.0562$

$\Delta e_{BC} = 0.125 \log \frac{1.232}{1.132} = 0.0046$

$\Delta e_{CD} = 0.390 \log \frac{1.898}{1.848} = 0.0047$

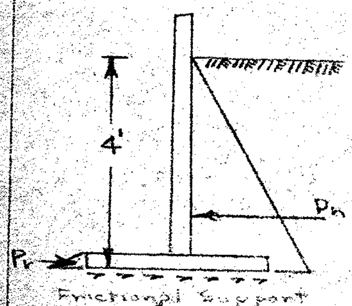
	e	$1+e$	Δe	H (in)	$\Delta H = \frac{H}{1+e} \times \Delta e$
A-B	1.15	2.15	.0562	72	1.882
B-C	0.89	1.89	.0046	12	.029
C-D	1.26	2.26	.0047	144	.299

total settlement = $\underline{\underline{2.110}}$ inches

Appendix C

To determine the safety factor against sliding of a basement wall

Data



Footing lies in brown silty clay
$\therefore \quad \gamma = 112 \text{ lb/cuft}, \ c = \frac{2683}{2}, \quad K = 1$
Footing at 4' depth. Minimum width of footing is 30 inches.
(C.M.H.C., Winnipeg area specification)

Computation

$$P_h = \frac{\gamma H^2}{2} = \frac{112 \times 4^2}{2} = 896 \text{ lb/ft}$$

$$P_r = \frac{112 \times .5}{2} = 14 \text{ lb/ft}$$

The resultant force $= 896 - 14 = 882$ lb/ft

The frictional support $= \frac{30}{12} \times \frac{2683}{2} = 3350$ lb/ft

$$\therefore \quad S.F. = \frac{3350}{882} = \underline{\underline{3.8}}$$