Complete Information Report

City of Winnipeg, WWD Resource Centre

 Date: 11/29/2012

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SOIL MECHANICS INVESTIGATION

PROPOSED EAST RESERVOIR AND PUMPING STATION

ST. BONIFACE, MANITOBA

A report prepared for the

GREATER WINNIPEG WATER DISTRICT

Winnipeg, Manitoba

by

B A R A C O S   A N D   M A R A N T Z

Consulting Engineers

Winnipeg, Manitoba   July, 1960

SOIL MECHANICS INVESTIGATION
GREATER WINNIPEG WATER DISTRICT
PROPOSED EAST RESERVOIR AND PUMPING STATION
ST. BONIFACE, MANITOBA

At the request of the Greater Winnipeg Water District of Winnipeg, Manitoba, a soil mechanics investigation was made for the proposed reservoir and pumping station to be located east of Bourget Road, and adjoining the G.W.W.D. aqueduct and railway, as shown in Drawing No 1. The purposes of the investigation were to determine soil conditions at the site and, on the basis of field and laboratory tests, to make recommendations regarding soil bearing capacities, settlements, reservoir side slopes, embankments, etc.

The writer was given to understand that, if soil conditions were reasonably similar, the design of this reservoir would be comparable to the City of Winnipeg reservoir at Wilkes Ave. and Waverley St.

PROCEDURE

Four test holes were bored with a 4 inch diameter power auger until refusal was encountered; this occurred at depths ranging from 49.5 to 58 ft. The locations of the test holes are shown on Drawing No. 1.

At regular intervals of depth of about 5 ft, 2 inch diameter thin-wall Shelby tube undisturbed soil samples were obtained in test holes 2 and 3. Because of similarity of soil conditions, as indicated by visual examination, a limited number of undisturbed samples were obtained in the other holes, and these were supplemented by moisture content samples. Representative bulk samples were obtained for identification tests.

The samples were tested at the University of Manitoba Civil Engineering Soil Testing Laboratory. Testing included unconfined compression strength, moisture content, degree of saturation, undisturbed density, consolidation, grain size, and liquid and plastic limits. The results of the field and laboratory tests are shown on the Test Hole Log Sheet, Drawing No 2. In addition, the laboratory tests are summarized in the appended report from the University.

Tests were not made for sulphate content detrimental to concrete employing ordinary cement, since the writer had been advised that concrete made of sulphate-resisting cement would be used for this project.

RESULTS OF TESTS

The four test holes showed similar soil conditions with the exception of test hole 3, which showed lower strengths in comparison to the other three test holes. In addition, considerable variation in material was noted in the upper 3 to 4 ft. The test holes showed a covering layer of about one foot or less of organic black, then grey, clayey silt. In hole 2, this was followed by 2 to 3 ft of soft tan silt. In the other holes, the organic topsoil was followed by clay or silty clay, and the tan silt occurred as thin layers, one foot thick or less, within the first four ft of depth. In addition, a thin layer of silt occurred between the 7 and 9 ft depths in holes 2 and 3; for the latter hole, the silt had an appreciable sand content, and water appeared to flow from this layer as the auger was advanced.

Below the silty soils mentioned above, clays of high plasticity were encountered. These clays are glacial lake deposits typical of those found in the Greater Winnipeg area, and consist of an upper brown varved clay layer, underlain by a grey clay layer below about the 25 ft depth. Pockets of gypsum crystals were observed in the brown clay in most of the holes. The brown and grey clays are highly susceptible to excessive swelling on wetting, and shrinking on drying. This action is particularly severe in the brown clay, where values of 103 and 93 were obtained for the liquid limit, and values of 38 and 39 were obtained for the plastic limit. Liquid and plastic limit values of 67 and 23 respectively were obtained for the grey clay at a depth of about 40 ft; these reduced values may be associated with an increasing silt content with depth. The grey clay was noticeably softer as the depth increased. Both brown and grey clays showed complete or nearly complete saturation.

In test hole 2, the unconfined compression strengths of the brown clay were generally low, ranging from about 1200 to 1500 lb per sq ft, except for one higher value of 2300 lb per sq ft in the uppermost portion. In test hole 3, the range of strength values for the same clays was from about 2200 to

2900 lb per sq ft. The grey clays showed unconfined strength values of about 2000 lb per sq ft, and decreased to as low as 1100 lb per sq ft in the lower portions of this material. On the basis of the limited testing, the strengths in test holes 1 and 4 appear to be higher than those in test hole and of the same order as those in test hole 3.

The grey clay was underlain by glacial till, beginning at about the 45 ft depth. This material was predominantly a mixture of silt, rockflour and sand with a low content of fine-sized gravel. The mixture was soft and had a "putty-like" consistency for all of its depth in test holes 1, 2 and 4, until auger refusal occurred at depths of about 50 to 52 ft. In test hole 3, the glacial deposit was penetrated to the 58 ft depth before refusal was met, and the auger action indicated a stiffer consistency, or coarser material, below the 48 ft depth. Attention is drawn to the very low unconfined compression strength of 320 and 396 lb per sq ft in test holes 3 and 4 respectively at about the 46 ft depth. Free water was observed in the till in holes 1, 2 and 3. It is expected that auger refusal occurred because the glacial till became extremely hard and could not be penetrated; locally this material is known as "hardpan".

ANALYSIS AND RECOMMENDATIONS

(1) Safe side slopes for reservoir

On the advice of the engineering staff of the G.W.W.D. it was assumed that a 10 ft high dyke and 23 ft total depth of reservoir would likely be used. For the purposes of analysis it may be noted that two cases constitute worst conditions. The first occurs when the reservoir is completed but prior to filling. Filling increases the safety factor in that pressure of water on the inside faces of the reservoir constitutes a force tending to oppose sliding. Subsequent emptying of the reservoir introduces the second case. At this time the safety factor can be somewhat lower. The excavation for the reservoir causes a decrease in pressure on the soils underlying the reservoir and although this may be partly off-set by filling the reservoir with water, the original soil pressures are not fully restored. There is some evidence to support the belief, that over a long period, stress relief of the soil is accompanied by some loss of strength. The analysis of the slope stability thus requires certain conservative assumptions. One such assumption used, was to base the analysis on the test hole 2 showing the lowest unconfined compression strengths. Account was also taken of other factors which may lower the stability. These in addition to those already mentioned, include soil shrinkage cracks forming in the upper soils subject to seasonal changes in moisture content, and considerable emphasis on the low strengths found at the greater depths. On this basis it was considered that the average ultimate shearing strength of the soils would be approximately 600 lb per sq ft. Stability analyses employing the circular sliding wedge showed a 4:1, (4 horizontal to 1 vertical) slope would be required for a safety factor of 1.5. It may be noted that this slope is less steep than the 3:1 slope used on the Waverly St. reservoir of the City of Winnipeg, which also employed a safety factor of 1.5. Lower soil strengths and, a greater depth to firm material are both factors that result in less favourable conditions at the St. Boniface site. It is recommended that the inside slopes for the St. Boniface reservoirs be no steeper than 4:1 for the total proposed height of 23 ft which includes 10 ft of dyke. Regarding the outside 10 ft high slopes, these can be much steeper. Considerations such as seeding to grass, grass cutting, rain erosion etc. become more important factors for the outside slopes. It is recommended that from stability consideration, slopes of 2:1 or less steep be employed.

(2) <u>The type of lining to be used for reservoir</u>

The clays encountered at the site are susceptible to large volume changes accompanying moisture changes. This is borne out by the unusually high liquid and plastic limits of 103 to 67, and 38 to 23 respectively shown in test hole 2. It may be noted that the very high values occur at the depths contemplated for the bottom of the reservoir. The consolidation tests showed negligible swelling for these same soils. This should not be interpreted that the soils will not swell. Soil drying during excavation could result in subsequent heave of the reservoir lining which would cut off evaporation and permit a build-up of moisture. The success of the lining depends to a great extent on avoiding a soil moisture change, and this is best accomplished by placing the lining as soon as possible after the excavation is completed. If excavation is interrupted by adverse weather it would be desirable to delay, say, the last few feet of excavation until it is possible to immediately place concrete. Excessive drying could result in 3 or 4 inches or possibly more, of differential heave which have been observed under similar conditions on ground supported floors. Even under the best of conditions, some differential movements must be anticipated. It is considered that a rigid lining as obtained by using a reinforced concrete slab will tend to prevent out-of-plane differential movements whereas a flexible lining would tend to warp. If only a minimum of out-of-plane movement can be tolerated, the reinforced concrete lining is recommended.

It is further considered desirable that the lining be placed directly on the clay without the use of any gravel fill. The reason for this recommendation is that granular fill placed under the lining would tend to collect water. Should the reservoir be emptied at a time when the granular fill was saturated, high hydrostatic pressures could develop under the lining which could cause its movement. This will not occur if granular fill is not used. The clay supporting the lining is of very low permeability as indicated by the consolidation tests. Although the clays will probably be saturated, the moisture would be tightly held in the soil and not free to move. Should it be considered desirable to use granular fill under the upper portions of the reservoir, the granular fill should be under-drained to adequately permit the escape of any water that would tend to collect in the granular fill.

The soft tan silt and the silty clay found in the upper two to four feet at the site are frost-heave susceptible. It is considered desirable that these soils not be used to support the lining of the reservoir in any location subject to frost penetration. In such locations it is recommended that these materials be removed and replaced with compacted clay, or under-drained with granular fill to provide at least 6 ft of such non-frost susceptible material. The 6 ft distance is to be measured perpendicular to any exposed surface subject to below freezing temperature. From experience the 6 ft is considered to be a reasonable estimate of frost penetration. Less than 6 ft of non-frost susceptible material may be used if the exposed surface is only subject to freezing for short periods of time and if the depth of frost penetration can be accurately estimated. The removal of the organic top layer and any soft silt found immediately under this layer is also recommended to provide a more satisfactory base for any fill to be placed as a dyke around the reservoir.

Experience in other areas of Winnipeg has shown that in more or less random locations, the soft tan silt layer may extend to several or more feet in depth and in the presence of free water be unstable. Without a very extensive test hole program, such random layers can go undetected. Specifications for excavation should anticipate their possible occurrence and the need for their excavation and include a suitable means for payment of additional excavation and construction costs.

It will be noted that occasional thin layers of silt were encountered at depths of 4 to 10 ft. If such layers are encountered of more than a few inches thickness, excavation into the slope of the dyke should be made and compacted clay backfill used to seal such layers. Seepage from such layers particularly at the greater depths would have an adverse lifting effect on the lining of the reservoir in an empty condition. A practical alternative to removing such layers where found at shallow depth, is to provide underdrainage.

<u>(3) Dyke fill</u>

For those portions of the reservoir consisting of earth fill, it is recommended that the silt excavated at the site be not used. This material cannot be satisfactorily compacted. If the predominantly clay soils are used, it is recommended that sheep foot compaction be employed, using lifts no greater than 6 inches. It is recommended that the fill be compacted to give 95 to 100% of Proctor Compaction maximum dry densities. Higher densities for excavated material are not considered practical as the natural soil moisture contents are well above optimum and cannot be sufficiently dried without causing excessive delays in construction. Excessively wet weather during construction may, anyway, require the work to stop for fairly lengthy periods.

(4) Building foundations

Regarding foundations for any proposed structures, the following comments and recommendations are made. Spread footings or a raft may be employed, on either the brown or grey clays using a net bearing value of 2000 lb per sq ft. On the basis of the test results this will give a safety factor of at least 2.5. If the structures are exceptionally wide or heavy, settlements would have to be calculated. This can only be done when the weight, depth of foundations and plan area of the structure are known.

Augered cast-in-place friction piles extending no deeper than about the 42 ft depth are also possible. Augered end-bearing piles which would have to be longer to reach firm material, or longer augered friction piles may encounter seepage too heavy to be practical. There is a possibility however that over certain areas of the site and depending on the season, such longer piles may be possible. This could only be ascertained by further testing just prior to construction.

On the basis of the strength tests, friction piles may be designed on the basis of 250 lb per sq ft of pile circumferential area which gives approximately a safety factor of 3. The length of pile in the top 10 ft below surface grade should be excluded as possible soil shrinkage in this zone makes frictional support unreliable.

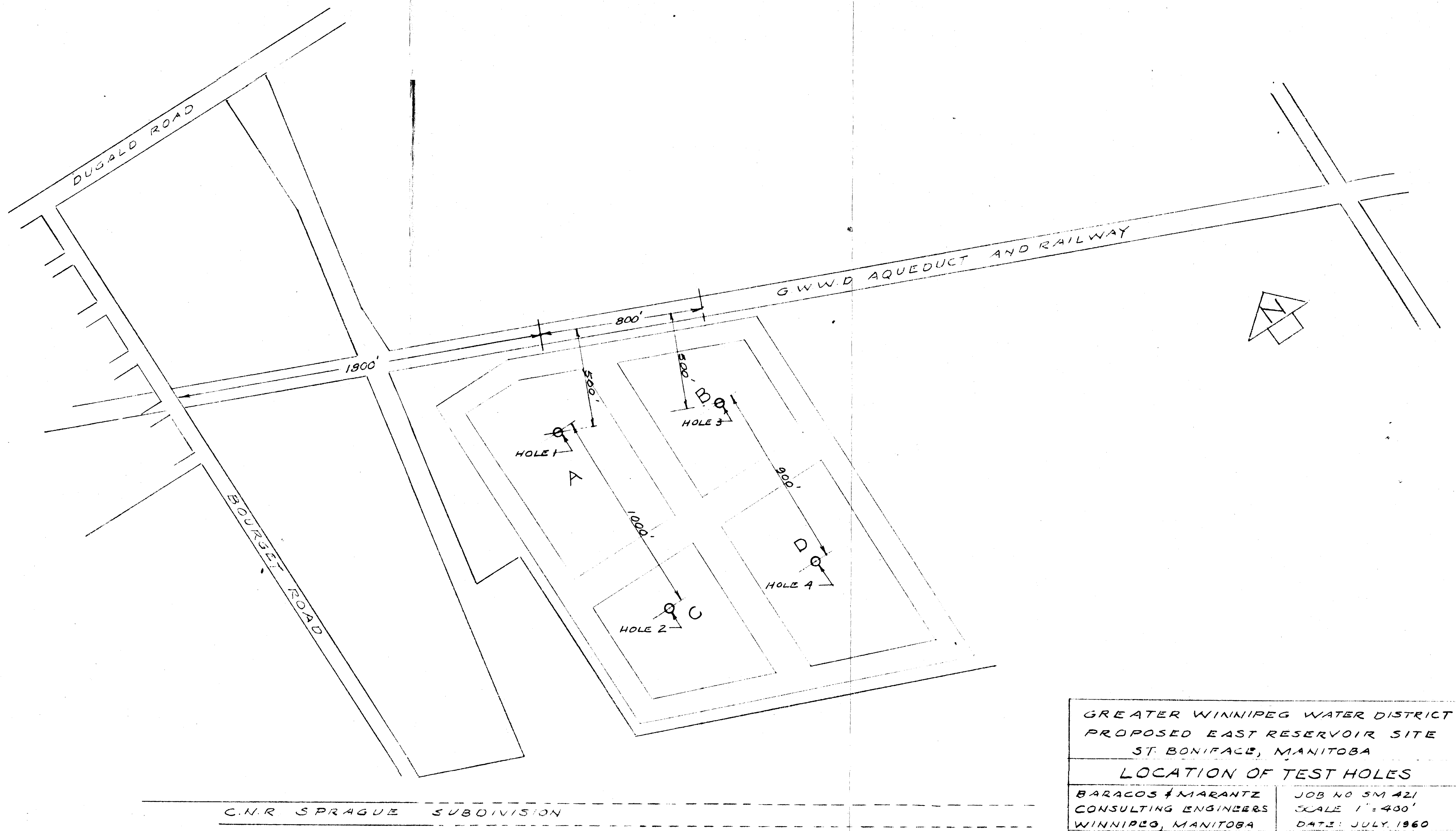
It may be noted that under similar soil conditions, driven 14 inch hexagonal precast concrete piles driven to refusal in the cemented silt, sand, gravel and rockflour mixture have supported loads of up to 60 tons. Refusal has been obtained after about 4 ft of penetration into the firm material. For heavy loads, where little settlement can be tolerated, driven end-bearing piles are recommended. Specific recommendations as to type of driven pile can be made when the type of structures proposed have been finalized.

Submitted August 10, 1960.

PROVINCE OF MANITOBA
ANDREW BARACOS
REGISTERED ENGINEER

A. Baracos

A. Baracos, P. Eng.
for BARACOS AND MARANTZ



GREATER WINNIPEG WATER DISTRICT PROPOSED EAST RESERVOIR SITE ST. BONIFACE, MANITOBA	
LOCATION OF TEST HOLES	
BARACOS & MARANTZ CONSULTING ENGINEERS WINNIPEG, MANITOBA	JOB NO SM 421 SCALE 1" = 400' DATE: JULY 1960
DRAWING NO. 1	

TEST HOLE LOG SHEET

PROJECT PROPOSED EAST RESERVOIR

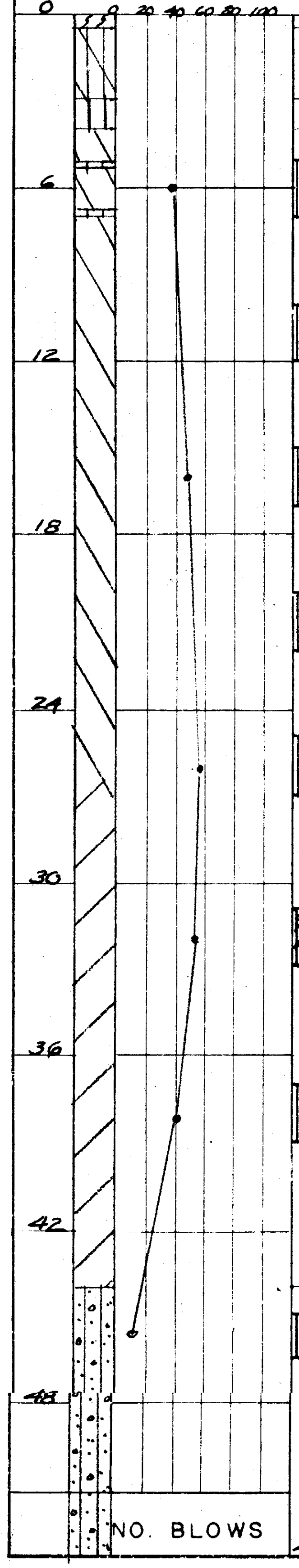
SITE ST. BONIFACE, MANITOBA

FOR GREATER WINNIPEG WATER DISTRICT WINNIPEG, MANITOBA

TEST HOLE NO. 3

LOCATION SEE DRAWING 1
HOLE ADVANCED BY 4 IN. POWER AUGER
LOGGED BY OM DATE JULY 4/60

DEPTH ELEV FT. | % MOISTURE | DESCRIPTION



BLACK ORGANIC SILTY CLAY

GREY-BROWN SILTY CLAY

SOFT TAN SILT

q_u=2900 γ_m=117 S=100

SILT LAYERS SEEPAGE

STIFF BROWN CLAY; VARVED, SCATTERED GYPSUM POCKETS

WATER AT THIS LEVEL AFTER 24 HR.

q_u=2210 γ_m=110 S=100

q_u=2620 γ_m=106 S=100

GRADUAL CHANGE TO GREY CLAY, MEDIUM STIFFNESS, BECOMING SOFTER WITH DEPTH

q_u=2040 γ_m=107 S=100

q_u=1110 γ_m=113 S=100

q_u=320 γ_m=153 S=100

PINK MIXTURE OF SAND, SILT, ROCK-FLOUR & SCATTERED GRAVEL; SOFT, PUTTYLIKE

AUGER ACTION INDICATES DENSE MATERIAL

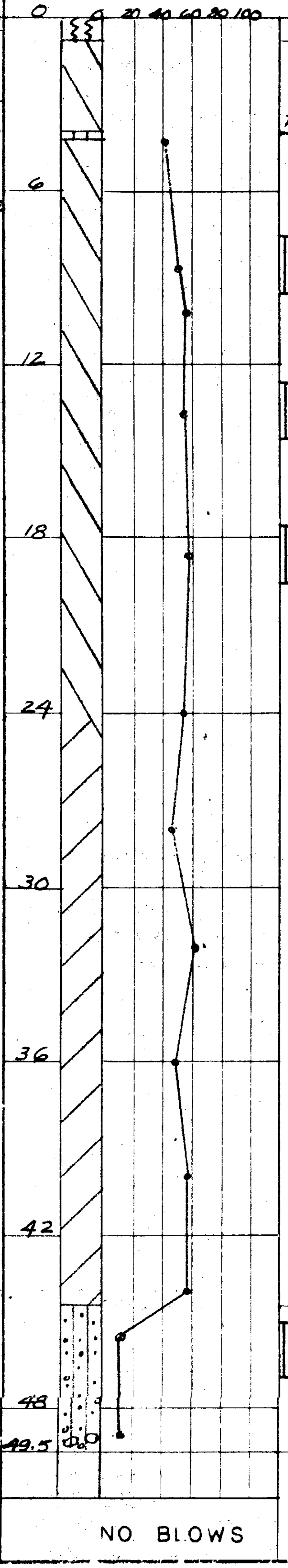
FREE WATER

AUGER REFUSAL AT D=58'

TEST HOLE NO. 4

LOCATION SEE DRAWING 1
HOLE ADVANCED BY 4 IN. POWER AUGER
LOGGED BY OM DATE JULY 4/60

DEPTH ELEV FT. | % MOISTURE | DESCRIPTION



BLACK ORGANIC CLAYEY SILT

STIFF

THIN LAYER ORANGE SILT

BROWN CLAY, VARVED,

q_u=1730 γ_m=105 S=100

MEDIUM STIFFNESS

q_u=2150 γ_m=109 S=100

GRADUAL CHANGE TO GREY CLAY, MEDIUM STIFFNESS, BECOMING SOFTER WITH DEPTH

q_u=396 γ_m=148 S=100

PINK MIXTURE OF SAND, SILT ROCKFLOUR & SCATTERED GRAVEL SOFT, PUTTYLIKE

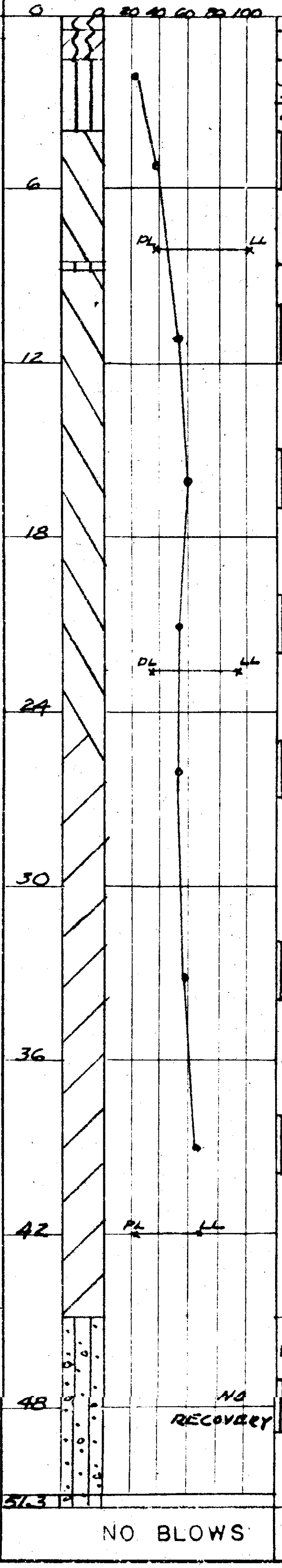
AUGER REFUSAL

HOLE DRY

TEST HOLE NO. 2

LOCATION SEE DRAWING 1
HOLE ADVANCED BY 4 IN. POWER AUGER
LOGGED BY OM DATE JULY 5/60

DEPTH ELEV FT. | % MOISTURE | DESCRIPTION



BLACK ORGANIC CLAYEY SILT

GREY " " "

SAND - 18 % SILT - 63 CLAY - 19 SOFT, TAN SILT

q_u=2320 γ_m=114 S=95

STIFF

THIN LAYER ORANGE SILT

q_u=1480 γ_m=104 S=95

BROWN CLAY, VARVED,

MEDIUM STIFFNESS

q_u=1240 γ_m=103 S=99

q_u=1440 γ_m=106 S=100

q_u=1650 γ_m=106 S=100

GRADUAL CHANGE TO GREY CLAY, MEDIUM STIFFNESS, BECOMING SOFTER WITH DEPTH

q_u=2010 γ_m=104 S=100

q_u=1420 γ_m=101 S=100

WATER REACHED THIS LEVEL IMMEDIATELY AFTER AUGERING

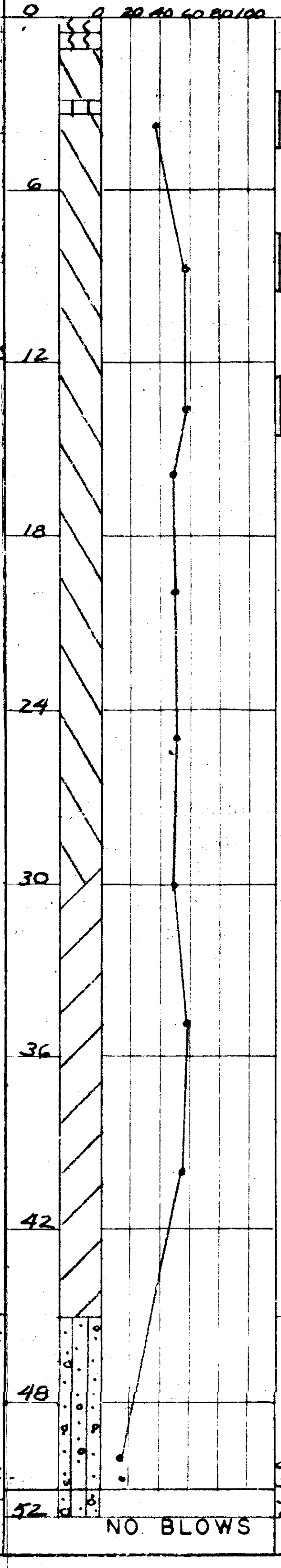
PINK MIXTURE OF SAND, SILT, ROCK-FLOUR & SCATTERED GRAVEL SOFT, PUTTYLIKE

AUGER REFUSAL

TEST HOLE NO. 1

LOCATION SEE DRAWING 1
HOLE ADVANCED BY 4 IN. POWER AUGER
LOGGED BY OM DATE JULY 6/60

DEPTH ELEV FT. | % MOISTURE | DESCRIPTION



BLACK ORGANIC CLAYEY SILT

GREY " " "

TAN SILT LAYER

BROWN CLAY, VARVED SCATTERED GYPSUM POCKETS

q_u=1470 γ_m=105 S=100

q_u=1810 γ_m=110 S=100

STIFF

MEDIUM STIFFNESS

GRADUAL CHANGE TO GREY CLAY, MEDIUM STIFFNESS, BECOMING SOFTER WITH DEPTH.

PINK MIXTURE OF SAND, SILT, ROCKFLOUR, SCATTERED GRAVEL, SOFT, PUTTYLIKE

GRAVEL - 13 %
SAND - 33
SILT - 41
CLAY - 13

FREE WATER

AUGER REFUSAL

INDEX

- ORGANIC CLAY
- CLAY
- ORGANIC SILT
- SILT
- SAND
- GRAVEL
- BED ROCK
- MOISTURE CONTENT %
- NO BLOWS STANDARD PENETRATION TEST
- PL LL PLASTIC, LIQUID LIMITS
- WATER TABLE

CODE

C = COHESION, LB/ SQ FT

ϕ = INTERNAL FRICTION, DEGREES

S = SATURATION, %

γ_d = UNDISTURBED DENSITY, LB/CU FT. (DRY BASIS)

γ_m = (WET BASIS)

q_u = UNCONFINED COMPRESSION STRENGTH, LB/SQ FT

] = UNDISTURBED SAMPLE

BARACOS AND MARANTZ
CONSULTING ENGINEERS
WINNIPEG MANITOBA

DRAWING NO. 2 JOB NO. SM 421

DRAWN BY OM DATE JULY, 1960

CHECKED BY DATE

File No. SM 421

LABORATORY TEST SUMMARY SHEET

Test Hole No	Depth ft	Sample No	Moisture Content %	Degree of Saturation %	Specific gravity	Moist Density lb/cu ft	Dry Density lb/cu ft	Strength Tests: Lateral Confining Pressure psi	Strength Tests: Deviator Stress at Failure psf	Strength Tests: Angle of Internal Friction Degrees	Strength Tests: Cohesion lb/sq ft	MIT Grain size Distribution: Clay %	MIT Grain size Distribution: Silt %	MIT Grain size Distribution: Sand %	MIT Grain size Distribution: Gravel %	Liquid Limit	Plastic Limit	Plasticity Index	Description Comments
1	2.5		37.8																
	7.5		57.3																SEE TEST HOLE
	12.5		57.2																LOG SHEET
	16		49.3																
	20		49.8																
	25		51.4																
	30		49.2																
	35		57.4																
	40		54.6																
	50		13.0																
	47-52											13	41	33	13				
2	1.5-4											19	63	18					
	2		23.4																
	4		36.3	95		114	83	0	2320										
	5-10															105	38	65	
	10		51.8	95		104	69	0	1480										
	15		58.7	99		103	65	0	1240										
	20		53.7	100		106	69	0	1440										
	20-25															93	34	59	
	25		53.7	100		106	69	0	1650										
	32		58.0	100		104	66	0	2010										
	38		65.3	100		101	61	0	1420										
	40-45															67	23	44	

Project EAST RESERVOIR SITE, ST BONIFACE, MANITOBA

Tests Requested by GREATER WINNIPEG WATER DISTRICT

Address WINNIPEG, MANITOBA

Date Submitted JULY, 1960 Checked by

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

File No. SM 421

LABORATORY TEST SUMMAPY SHEET

Test Hole No	Depth ft	Sample No	Moisture Content %	Degree of Saturation %	Specific gravity	Moist Density lb/cu ft	Dry Density lb/cu ft	Strength Tests: Lateral Confining Pressure psi	Strength Tests: Deviator Stress at Failure psf	Strength Tests: Angle of Internal Friction Degrees	Strength Tests: Cohesion lb/sq ft	MIT Grain size Distribution: Clay %	MIT Grain size Distribution: Silt %	MIT Grain size Distribution: Sand %	MIT Grain size Distribution: Gravel %	Liquid Limit	Plastic Limit	Plasticity Index	Description Comments
3	5		36.2	100		117	86	0	2900										SEE TEST HOLE LOG SHEET
	15		47.3	100		110	74	0	2210										
	25		54.3	100		106	69	0	2620										
	31		51.7	100		107	71	0	2040										
	37		40.7	100		113	80	0	1110										
4	4		41.5																
	7.6		52.3																
	10		56.7																
	12.6		54.0																
	17.5		56.8																
	24		54.2																
	28		46.6																
	32		60.7																
	36		49.7																
	40		56.9																
	44		57.4																

Project EAST RESERVOIR SITE, ST. BONIFACE, MANITOBA

Tests Requested by GREATER WINNIPEG WATER DISTRICT

Address WINNIPEG, MANITOBA

Date Submitted JULY, 1960 Checked by

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

File No. SM 421

LABORATORY TEST SUMMARY SHEET

Test Hole No	Depth ft	Sample No	Moisture Content %	Degree of Saturation %	Specific gravity	Moist Density lb/cu ft	Dry Density lb/cu ft	Strength Tests - Lateral Confining Pressure psi	Strength Tests - Deviator Stress at Failure psf	Strength Tests - Angle of Internal Friction Degrees	Strength Tests - Cohesian lb/sq ft	M I T Grain size Distribution - Clay %	M I T Grain size Distribution - Silt %	M I T Grain size Distribution - Sand %	M I T Grain size Distribution - Gravel %	Liquid Limit	Plastic Limit	Plasticity Index	Description Comments
SUPPLEMENTARY					TESTS														
1	7.5		53.3	100		105	69	0	1470										
1	12.5		46.2	100		110	75	0	1810										
3	45		11.9	100		151	136	0	320										
4	12.5		50.6	100		105	70	0	1730										
4	17.5		47.0	100		109	74	0	2150										
4	45		11.9	100		148	132	0	396										

Project: EAST RESERVOIR SITE, ST BONIFACE, MANITOBA

Tests Requested by: GREATER WINNIPEG WATER DISTRICT

Address: WINNIPEG MANITOBA

Date Submitted: AUG 1969 Checked by

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

SM 421

CONSOLIDATION TEST RESULTS

Specific Gravity of Soil Solids G_s = 2.75

Height of Soil Solids $H_s = \frac{\text{Wt Soil}}{G_s \times \text{Area} \times 2.54}$ ins. = 0.244"

Void Ratio e (Start Dimensions) = $\frac{.572-.244}{.244}$ = 1.34

e = previous e ± $\frac{\text{Defl.}}{H_s}$

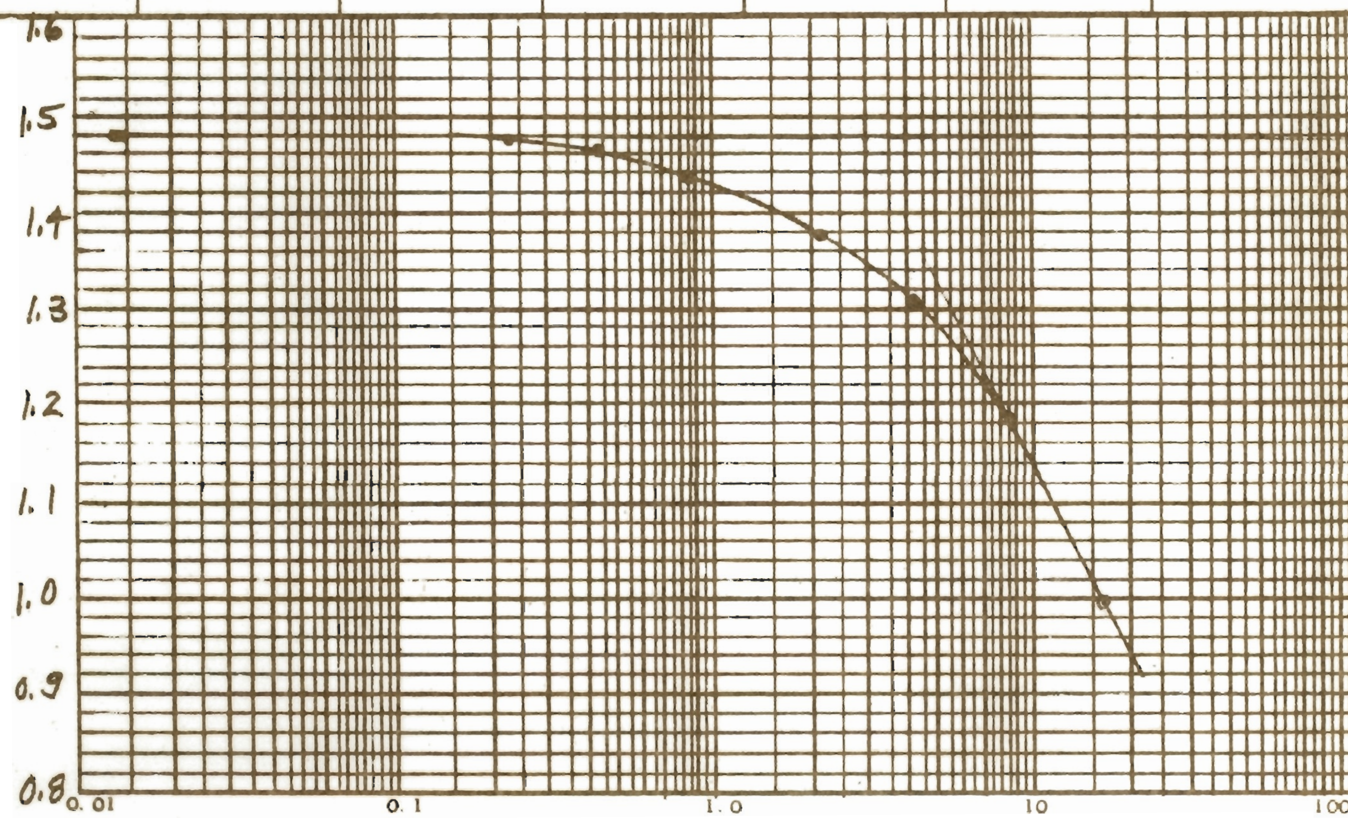
e (End) = W% (End) x G_s = 0.995

Compression Index 0.64

Swelling Pressure 0 Ton/ft²

Pre Consolidation Load — Ton/ft²

Time Interval (hrs)	Load on Pan (lbs.)	Corr Dial Reading (ins)	Deflection (ins)	Deflection / H_s	Void Ratio e	Pressure Ton/ft²
—	0	0.5000	+ .1180	0.485	1.48	0.14
46	½	.4993	+ .1173	.48	1.475	0.23
4	1	.4962	+ .1142	.47	1.465	0.44
20	2	.4896	+ .1076	.44	1.435	0.85
71	5	.4743	+ .0923	.38	1.375	2.1
24	10	.4585	+ .0765	.315	1.31	4.2
22½	20	.4288	+ .0468	.19	1.185	8.3
28¼	40	0.3820	0.0000	0.00	0.995	16.6



Pressure Ton/ft²

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

Project ST. BONIFACE RESERVOIR
Site
Sample T103 Hole 2 Depth 15'
Technician RJM Date 3-8-60

CONSOLIDATION TEST RESULTS

Specific Gravity of Soil Solids G_s = 2.75 assumed

Height of Soil Solids $H_s = \frac{\text{Wt Soil}}{G_s \times \text{Area} \times 2.54}$ ins. = 0.193"

Void Ratio e (Start Dimensions) = $\frac{.579-.193}{.193}$ = 2.00

e = previous e $\pm \frac{\text{Defl.}}{H_s}$

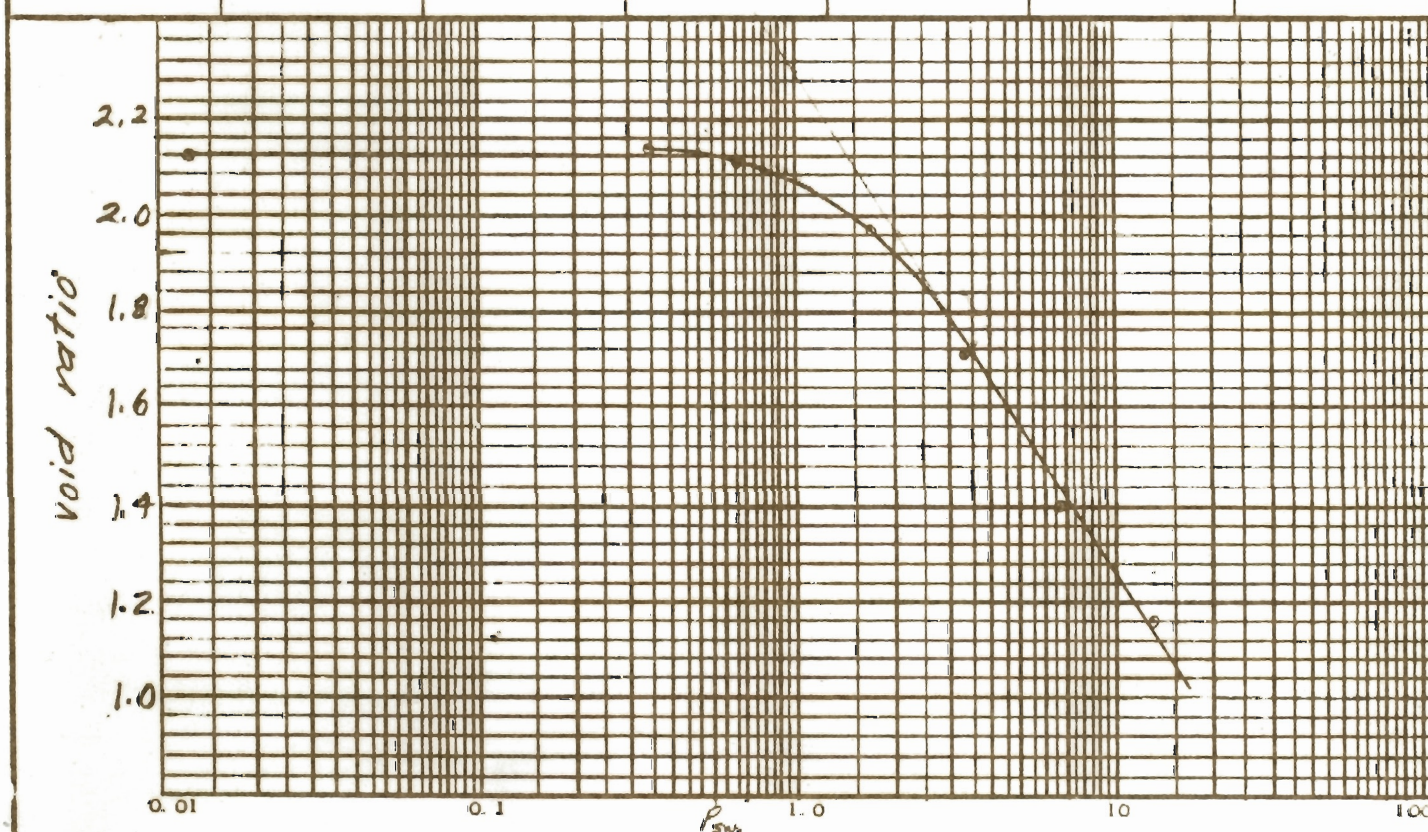
e (End) = W% (End) x G_s = 1.16

Compression Index 1.04

Swelling Pressure .... 0.55 Ton/ft²

Pre Consolidation Load .... Ton/ft²

Time Interval (hrs.)	Load on Pan (lbs.)	Corr Dial Reading (ins)	Deflection (ins)	Deflection / H_s	Void Ratio e	Pressure Ton/ft²
—	0	0.5000	+ 0.1854	0.96	2.12	0.012
44	1	0.5027	+ .1881	.975	2.135	0.34
24	2	0.4974	+ .1828	.945	2.105	0.67
24	5	.4707	+ .1561	.81	1.97	1.67
23	10	.4213	+ .1067	.55	1.71	3.30
22½	20	.3613	+ .0467	.24	1.40	6.65
28½	40	0.3146	0.0000	0.000	1.16	13.2



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

Project St. Boniface Reservoir, SN 421

Site

Sample T48 Hole 2 Depth 38'

Technician RTM Date 3-8-60