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W.P. No. 48-71-09

CONT. No.

W. O. No. 72-11001

STR. SITE No.

HWY. No.

LOCATION PROPOSED SUBWAY STRUCTURE

PROPOSED GOREWAY DRIVE

No OF PAGES - ~

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

30M12-60

TO: Mr. G.C.E. Burkhardt, (2)
Regional Bridge Planning Engineer,
Central Region,
90 Floral Parkway, Downsview.

FROM: Foundations Office,
Design Services Branch,
Central Bldg., Downsview.

ATTENTION:

DATE: March 9, 1972.

OUR FILE REF.

IN REPLY TO MAR 13 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Subway Structure at the Crossing
of the C.N.R. and Proposed Goreway Drive
County of Peel, Town of Mississauga
District No. 6 (Toronto)
W.O. 72-11001 -- W.P. 48-71-09

Attached, we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER.

AGS/ao
Attach.
cc: Messrs. D. W. Farren
B. R. Davis
A. Rutka
G. K. Hunter
H. Greenland
B. J. Giroux
T. J. Kovich
G. A. Wrong
B. A. Singh
McCormick & Rankin & Assoc. Ltd.
Foundations Files
Documents

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
For
Proposed Subway Structure at the Crossing
of the C.N.R. and Proposed Goreway Drive
County of Peel, Town of Mississauga
District No. 6 (Toronto)
W.O. 72-11001 -- W.P. 48-71-09

1. INTRODUCTION:

The Foundation Office was requested to carry out a subsurface investigation for the subway structure to be constructed at the crossing of the C.N.R. and the proposed Goreway Drive, in the Town of Mississauga, County of Peel. The request was contained in a memo from the Bridge Office (Mr. G. C. E. Burkhardt, Regional Bridge Planning Engineer) dated December 29, 1971. Subsequently, an investigation was carried out by this Office to determine the subsoil, bedrock and groundwater conditions at the site.

The results of the investigation are presented in this report, together with our recommendations for the design of the structure foundations as well as the stability considerations associated with the approach cuts.

2. DESCRIPTION OF SITE AND GEOLOGY:

The site is located along the east-west trending C.N.R. line, immediately west of Indian Line and approximately 1 mile south of Rexdale Blvd., in the Town of Mississauga. The terrain is gently undulating in relief between elevations 536 and 546. The area has been developed for small industrial enterprises; many one and two storey factories and warehouses are located here. The grade of the existing C.N.R. line is at about the level of the surrounding ground. Shallow (approximately 2 to 3 feet deep)

.....2

drainage ditches are located on either side of the track.

The site is located in the physiographic region known as the "Peel Plain." The characteristic deposit in this region is a ground moraine laid down during the Wisconsinan Glacial Age. In the vicinity of the area under investigation, the moraine is primarily composed of a cohesive glacial till whose thickness generally ranges from 75 to 90 feet. In this region the Humber River, Etobicoke and Mimico Creeks have cut deep valleys into the overburden. There is, therefore, no large undrained depression, swamp or bog, although in many of the instream areas drainage is still imperfect. The overburden is underlain by grey shale bedrock of the Meaford-Dundas formation, Ordovician Period. Available geological information indicates that the surface of the bedrock varies somewhere between 458 and 473.

3. FIELD AND LABORATORY WORK:

A total of thirteen boreholes, all of which were accompanied by a dynamic cone penetration test, was carried out at the site during the course of the field investigation. The boreholes and the cone penetration tests were advanced by means of a continuous flight auger machine (Penn Drill) or a diamond drill rig, both of which were adapted for soil sampling purposes.

Samples were obtained at required depths in a 2-inch O.D. split spoon sampler which was hammered into the soil. The method of driving the split-spoon conformed to the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. Bedrock was proven at three of the boring locations by obtaining BX size rock core samples.

During sampling and drilling operations, detailed logs of the borings were made. These logs contain a record of the drilling and sampling techniques used, together with the soil types and bedrock encountered. The location and elevation of

.....3

all the boreholes are shown on Drawing No. W.O. 72-11001A, together with estimated stratigraphical sections across the site. Surveying at the site was carried out by the personnel from Central Region Engineering Survey Section. The elevations given in this report are referred to a Geodetic datum.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples to determine the following physical properties of the overburden:

Natural Moisture Content
Atterberg Limits
Grain-Size Distribution

The results of these tests are plotted on the Record of Borelog sheets as well as the figures located in Appendix I of this report.

4. SUBSOIL AND BEDROCK CONDITIONS:

4.1) General:

The predominant stratum across the site is a cohesive glacial till, the thickness of which varies from 77 feet to 87 feet. This cohesive deposit is underlain by shale bedrock.

The boundaries of the various deposits, as determined in the boreholes, are shown on the accompanying Record of Borehole sheets. The stratigraphical sections, shown on Drawing No. 72-11001A, have been inferred from this data. From ground surface downward, the soil types and bedrock encountered are as follows:

4.2) Glacial Till (Heterogeneous Mixture of Clayey Silt, Sand and Gravel):

Directly beneath a nominal topsoil cover (1 foot or less) is the predominant stratum across the site, which is composed of a heterogeneous mixture of clayey silt with sand and gravel.

.....4

The thickness of this glacial till varies from 77 feet (B.H. #2) to 88 feet (B.H. #9). Localized pockets of silty clay were encountered in the upper portion of the stratum at B.H.'s #6, 9 and 10. Occasional seams of silt up to 2 feet thick are present randomly throughout the deposit. Grain-size distribution curves, for samples of the cohesive stratum obtained with 2" O.D. sampling equipment, are shown on Figure No. 2 in Appendix I.

Atterberg limit tests were performed on samples of the glacial till. The results are tabulated below:

			Range	(Average)
Liquid Limit	(W_L)	(%)	15.5 - 40	(26)
Plastic Limit	(W_p)	(%)	12 - 19	(15)
Natural Moisture Content	(W)	(%)	7 - 17	(9)

Based on these values it is estimated that the cohesive deposit has a matrix, which is inorganic and of low to intermediate plasticity. The natural moisture content was typically 2 to 5 percent below the plastic limit.

The Standard Penetration Tests, carried out within this glacial till stratum, are plotted on the Record of Borehole sheets. This testing gave 'N' values which ranged from 4 blows/ft. to 100 blows for 1 inch. The lower 'N' values occurred in the upper few feet of the deposit. This would indicate that this zone has been softened by weathering. Based on this testing it is estimated that the consistency of this cohesive deposit varies from very stiff to hard, being generally in the hard range. The upper softened zone, however, has a firm consistency.

4.3) Shale Bedrock:

The cohesive glacial till stratum is directly underlain by bedrock which was proven in three of the boreholes by obtaining up to 5 feet of BX size rock core samples. In addition, the surface of the bedrock, at a number of other boring locations, was inferred to exist at the level where the hammer driven casing

.....5

met practical refusal. Over the site the bedrock surface was found to vary randomly between elevations 460 and 470. The bedrock is composed of a grey shale, which is in a sound state, as evidenced by the high percentage of core recovered.

5. GROUNDWATER CONDITIONS:

The groundwater level conditions across the site, during the period of the investigation (January 1972), were observed by taking readings in the open boreholes. The results of the readings are shown on the Borelog sheets, as well as on Drawing No. 72-11001A.

The observations indicate that the groundwater level is located between elevations 534.5 and 540.5, which corresponds to levels which range from 1 to 6 feet below existing ground surface.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to extend the present airport expressway northerly toward Finch Ave. following closely to the existing Indian Line Rd. This expressway will be designated as Hwy. #427. Interchanges are contemplated at the crossings of Dixon Rd., Belfield Expressway (Hwy. #409), Rexdale Blvd. and Finch Ave. In addition, structures will be required at the crossing of the Mimico Creek complex, American Drive, proposed Goreway Drive, Canadian National Railway, the Woodbine Racetrack Entrance and Morningstar Drive. At present the northern limit of this project has not been decided upon.

This report will deal with the subway structure to be constructed at the crossing of proposed Goreway Drive and the C.N.R. This crossing will be located immediately west of Indian Line approximately 1 mile south of Rexdale Blvd. in the Town of Mississauga, County of Peel. Foundation reports, for the other structure crossings along the expressway, will be presented under separate cover.

.....6

The profile grade of proposed Goreway Drive, in the vicinity of it's crossing with the C.N.R., is to range between elevations 520 and 521. At this grade Goreway Drive will be located in a cut section, whose maximum depth extends up to 22 feet below the existing ground surface. The profile grade of the existing C.N.R. track is similar to that of the surrounding terrain; this grade is to be maintained. It is understood that the C.N.R., in the vicinity of the proposed structure, will be detoured during the construction period.

Two possible schemes are being considered for the subway structure. One is to employ a two span (45' - 45'), 70 feet wide structure constructed with closed-type of abutments. Alternatively the subway may be designed as a three span (40' - 70' - 40'), 70 feet wide structure with the abutments 'perched' within the approach cuts.

The predominant stratum across the site is a cohesive glacial till, whose thickness ranges from 77 to 87 feet. This deposit is underlain by shale bedrock.

Recommendations pertaining to the foundation design for the two alternate schemes are presented separately in the subsections to follow. Also included are comments with regard to the stability of the cut section.

6.2) Two Span Structure (Closed-Type of Abutments):

6.2.1) Goreway Drive Cut Section:

As mentioned elsewhere the profile grade of Goreway Drive is such that cuts up to 22 feet in depth will be required in the vicinity of the structure. The excavation will extend through the basically competent glacial till, therefore, no stability problems are anticipated, provided standard 2:1 slopes are employed.

The cut will extend some 15 to 20 feet below the groundwater level recorded during the period of the investigation. The cohesive glacial till is relatively impervious; therefore, no major dewatering complications are anticipated. Some water

.....7

bearing granular layers are, however, present within the glacial deposit. If any of these are intersected by the excavation some groundwater seepage may occur. This seepage could be controlled using conventional techniques such as pumping from sumps.

The cut slopes should be protected against the erosional effects of uncontrolled surface runoff or groundwater seepage. This could be accomplished by either seeding and mulching or alternatively sodding the slopes.

6.2.2) Structure Foundations:

As mentioned elsewhere the profile grade of proposed Goreway Drive, in the vicinity of the structure, is to be at about elevation 520. At least 4 feet of earth cover should be provided above the base of the foundations in order to satisfy the frost protection requirements in this area. Taking this into consideration the centre pier and closed-type abutment foundations could be founded at or below elevation 516. Below this elevation the cohesive subsoil is competent, therefore, the structure elements can be founded on spread footings. These footings can be designed using an allowable bearing pressure of 3.0 t.s.f.

The base of the footings will be located well below the groundwater level recorded during the period of the investigation. In this regard it is recommended that the foundation elements be constructed only after the Goreway Drive cut has been made and the permanent drainage system installed. This provision would lower the prevailing groundwater level at these locations. The excavations will be carried out within the relatively impervious cohesive glacial till. No major dewatering problems are, therefore, anticipated. As mentioned previously, there are some water bearing granular seams and layers throughout the deposit. If these are encountered during the construction period some groundwater may flow into the excavations. This seepage could be handled using conventional techniques such as pumping from sumps. A complication would arise, however, if such a thin granular layer is present at the base of the excavation. In

.....8

this instance, the material may boil(and thus be loosened) due to the unbalanced hydrostatic water pressure head existing. If this occurs the granular layer should be sub-excavated to its full depth; the excavation so formed should be brought up to footing foundation level using either mass concrete or granular 'A' material.

Settlement will be induced in the cohesive glacial till by the imposed footing pressure. The cohesive till is highly preconsolidated, therefore, the settlement will be of a recompression nature; i.e., take place during or immediately following the construction period. Computations carried out indicate that this settlement should not exceed 3/4 inch, provided the foundation subsoil is not softened by uncontrolled surface runoff or groundwater seepage. In this regard it is recommended that a lean concrete working slab should be placed at the footing formation level as soon as it is reached.

The following paragraphs pertain to the foundation design of the closed-type abutment walls and footings.

If the structure is designed as a rigid frame, then a coefficient of earth pressure at rest (K_o) of 0.5 should be assumed for the granular fill placed behind the wall, when designing the abutments. However, if some movement of the wall is permitted, then a coefficient of active earth pressure (K_a) of 0.33 can be used.

The granular backfill behind the wall should be allowed to drain in order to prevent the buildup of excess hydrostatic groundwater pressures in this area. This can be accomplished by providing weep holes at the base of the walls. The location and spacing of these weep holes should be determined in accordance with current D.T.C. practices.

It is recommended that a value of 2,500 p.s.f. be used in the computations to determine the sliding resistance between the concrete base of the footing and the underlying cohesive till stratum.

.....9

The closed-type vertical wall sections will be inherently stable with respect to a deep-seated rotational type of failure in the cohesive subsoil located beneath the spread footings.

6.3) Three-Span Structure (Abutments 'Perched' in the Approach Cuts):

6.3.1) Goreway Drive Cut Section:

Comments and recommendations made with regard to this cut section were discussed in Subsection 6.2.1).

6.3.2) Structure Foundations:

i) East and West Pier:

As discussed in detail in Subsection 6.2.2) these piers can be founded on spread footings located at or below elevation 516 in the competent cohesive glacial till deposit. These footings can be designed using an allowable bearing value of 3.0 t.s.f.

Comments and recommendations made with regard to i) dewatering requirements and ii) expected settlement of the foundation subsoil, were presented as well in Subsection 6.2.2).

ii) Abutments:

If this scheme is adopted the abutments will be 'perched' within the approach cuts. No stability problems are envisaged, provided i) the approach cuts are trimmed no steeper than 2:1 and ii) the finalized slopes are protected against the erosional effect of uncontrolled surface runoff or groundwater seepage.

The abutments will be located in the competent portion of the glacial till deposit; they, therefore, can be founded on spread footings. At least 4 feet of earth cover should be provided above the base of the footings in order to satisfy the frost protection requirements. Taking the aforementioned into consideration it is inferred that the base of the footings would be located somewhere between elevations 530 and 532.

.....10

Footings founded between these elevations could be designed using the following bearing values.

Abutment	Allowable Bearing Pressure	Refer to
West	2.5 t.s.f.	B.H.'s #1 and 2
East	3.0 t.s.f.	B.H.'s #9 and 10

The spread footings will be located below the groundwater level recorded during the period of the investigation. This level will be lowered following the excavation of the Goreway Drive cut. Further, the footing excavations will be carried out within the relatively impervious cohesive glacial till deposit. No major dewatering problems are, therefore, anticipated. Any minor seepage into the excavations could be handled using conventional techniques such as pumping from sumps.

Settlement will be induced in the cohesive till by the imposed footing pressure. The cohesive till is highly preconsolidated, therefore, the settlement will be of a recompression nature - i.e., take place during or immediately following the construction period. Computations carried out indicate that this settlement should not exceed 3/4 inch, provided the foundation subsoil is not softened by uncontrolled surface runoff or groundwater seepage. In this regard it is recommended that a lean concrete working slab should be placed at the footing formation level as soon as it is reached.

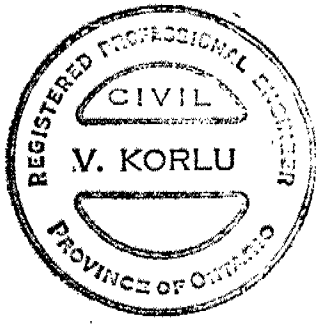
7. MISCELLANEOUS:

The field work for this project was carried out during the period of January 7 to 24, 1972, under the supervision of Mr. V. Korlu, Project Foundation Engineer, and Mr. H. Szymanski, Foundation Technician.

The equipment used was owned and operated by Master Soil Investigation Ltd., Toronto.

....11

This report was written by Mr. V. Korlu and reviewed by Mr. M. Devata, Supervising Foundation Engineer.



V. Korlu, P. Eng.

M. Devata, P. Eng.

VK/ao
March 8, 1972.

APPENDIX I

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

72-11001

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,474 N; 969,519 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 24, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
540.5	Ground Level					
0.0	Het.mix.of clayey silt, sand & gravel					540
	Glacial Till		1	SS	31	
	occ.seams of silt.		2	SS	25	
	Brown		3	SS	44	530
	Grey		4	SS	14	
	Stiff to Hard		5	SS	20	
			6	SS	21	
			7	SS	19	520
			8	SS	22	
			9	SS	53	510
			10	SS	46	
			11	SS	63	500
			12	SS	41	
			13	SS	37	490
			14	SS	51	480
			15	SS	100/5"	470
460.7						
79.8	End of Borehole Probable Bedrock					460

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT — 20 40 60 80 100

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_p
WATER CONTENT —— w
w_p w w_L
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS GR.SA.SI.CL.

▼ 534.5 in open BH Jan.26/72
6 17 51 26
8 29 41 22
8 34 40 18

20
15 ⟡ 5 % STRAIN AT FAILURE
10

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION 2

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,540 N; 969,505 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 21 & 25, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE
539.4	Ground Level					
0.0	Het.mix.of clayey silt,sand & gravel Glacial Till occ. seams of silt Brown		1	SS	25	
			2	SS	20	
			3	SS	39	530
	Grey		4	SS	15	
	Very Stiff to Hard		5	SS	17	
			6	SS	17	520
			7	SS	29	
			8	SS	27	
			9	SS	37	510
			10	SS	52	
			11	SS	30	500
			12	SS	53	
			13	SS	55	490
						480
			14	SS	152/6"	470
462.4						
77.0	Bedrock Shale					
457.4	Grey		15	RC	85%	460
82.0	End of Borehole					450

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
w_P w w_L
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR. SA. SI. CL.

▼ 537.8 in open BH Jan.26/72

4 24 51 21

5 22 50 23

4 34 47 15

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD–MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

3

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,477 N; 969,557 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 19, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FOOT	ELEV. SCALE
540.0	Ground Level					
0.0	Het.mix.of clayey silt, and & gravel. Glacial Till		1	SS	27	
			2	SS	18	
			3	SS	34	530
	Brown		4	SS	35	
	Grey		5	SS	33	
	Very Stiff to Hard		6	SS	28	
			7	SS	18	520
			8	SS	29	
			9	SS	46	
			10	SS	135	510
			11	SS	107	
			12	SS	52	500
488.5			13	SS	46	490
51.5	End of Borehole					480

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
w_P w w_L
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR.SA.SI.CL.

▼538.0 in open BH Jan.26/72

3 23 53 21

3 24 45 28

3 25 44 28

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION 4

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,544 N; 962,543 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 20, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
540.0	Ground Level					
0.0	Het.mix.of clayey silt,sand & gravel, seams of silt. Glacial Till Brown		1	SS	29	
			2	SS	28	
			3	SS	42	530
	Grey		4	SS	37	
	Very Stiff to Hard		5	SS	27	
			6	SS	36	520
			7	SS	31	
			8	SS	33	
			9	SS	54	510
			10	SS	45	
			11	SS	32	500
			12	SS	40	
			13	SS	20	490
			14	SS	22	
			15	SS	100	480
			16	SS	120/6"	
						470
465.0			17	SS	150/5"	
75.0	End of Borehole Probable Bedrock					460

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

538.5 in open BH Jan.26/72

3 22 52 23

2 28 50 20

6 31 54 9

20
15 5 % STRAIN AT FAILURE
10

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,476 N; 969,594 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 20, 21, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE
544.0	Ground Level					
0.0	Het.mix.of clayey silt,sand & gravel Glacial Till Brown Grey Very Stiff to Hard		1	SS	21	540
			2	SS	16	
			3	SS	46	
			4	SS	34	530
			5	SS	28	
			6	SS	25	
			7	SS	28	520
			8	SS	42	
			9	SS	52	
			10	SS	162	510
			11	SS	103	
			12	SS	81	500
			13	SS	79	
			14	SS	57	490
			~~15~~	~~SS~~	~~100/1"~~	480
						470
464.0			16	SS	100/0"	
80.0	End of Borehole Probable Bedrock					460

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

▼ 541.7 in open BH Jan.26/72

GR.SA.SI.CL.

13 22 44 21

15 27 37 21

7 33 47 13

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD–MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION 6

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,541 N; 969,582 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 19, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE
542.7	Ground Level					
0.0	Het.mix.of clayey silt,sand & gravel, occ.seams of silty clay. Glacial Till — Brown / Grey — Stiff to Hard		1	SS	14	540
			2	SS	37	
			3	SS	36	
			4	SS	21	530
			5	SS	23	
			6	SS	30	
			7	SS	71	
			8	SS	36	520
			9	SS	38	
			10	SS	116	510
			11	SS	80	
			12	SS	86	500
			13	SS	52	
			14	SS	24	490
481.2			15	SS	152	
61.5	End of Borehole					480

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT — 20 40 60 80 100
SHEAR STRENGTH P.S.F. — ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
WATER CONTENT % — 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR.SA.SI.CL.

▼ 538.7 in open BH Jan.26, 1972

1 30 63 6

8 39 31 22

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,478 N, 969,636 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 18 & 19, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Penetration CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FOOT	ELEV. SCALE
541.0	Ground Level					
0.0	Het.mix.of clayey silt, sand & gravel					540
			1	SS	17	
	Glacial Till		2	SS	5	
			3	SS	39	
	Brown		4	SS	23	530
	Grey		5	SS	24	
	Firm to Hard		6	SS	26	
			7	SS	29	520
			8	SS	29	
			9	SS	61	510
			10	SS	67	
			11	SS	63	500
			12	SS	109	
489.5			13	SS	107	490
51.5	End of Borehole					480

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
w_P w w_L
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR.SA.SI.CL.

▼ 539.0 in open BH Jan.26/72

2 14 61 23

13 32 41 14

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD–MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,547 N; 969,625 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 17, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE
540.2	Ground Level					
0.0	Het.mix.of clayey silt, sand & gravel. Glacial Till Brown		1	SS	42	
			2	SS	47	
			3	SS	44	530
	Grey		4	SS	27	
			5	SS	16	
	Very Stiff to Hard		6	SS	32	
			7	SS	26	520
			8	SS	30	
			9	SS	104	510
			10	SS	105/6"	
			11	SS	65	500
			12	SS	37	
			13	SS	16	490
			14	SS	130	480
			15	SS	100/6"	470
460.1			16	SS	100/1"	
80.1	End of Borehole Probable Bedrock					450

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
w_P w w_L
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR.SA.SI.CL.

539.2 in open BH Jan.26/72

7 31 41 21

11 32 42 15

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 9

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,484 N; 969,670 E. ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan. 14, 17 & 18, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FOOT	ELEV. SCALE
542.0	Ground Level					
0.0	Het.mix.of clayey silt,sand & gravel, occ.seams of silty clay. Glacial Till Brown Grey Firm to Hard		1	SS	5	540
			2	SS	4	
			3	SS	49	
			4	SS	37	530
			5	SS	30	
			6	SS	34	
			7	SS	28	520
			8	SS	20	
			9	SS	50	510
			10	SS	100/6"	
			11	SS	68	500
			12	SS	30	
			13	SS	49	490
			14	SS	87	480
			15	SS	100/1½"	470
			16	SS	100/1"	
459.5						460
82.5	Shale Bedrock Sound		17	BXL	100%	
454.5						
87.5	End of Borehole					450

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS	GR.	SA.	SI.	CL.
▼ 539.7 in open BH Jan.26/72				
	2	34	48	16
	10	28	44	18
	18	30	43	9

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD–MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 10

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,545 N; 969,660 E ORIGINATED BY HS
W.P. 48-71-09 BORING DATE Jan 18 & 19, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FOOT	ELEV. SCALE
543.6	Ground Level					
0.0	Het.mix.of clayey silt,sand and gravel, occ.seams of silty clay Glacial Till Brown Grey Hard		1	SS	33	540
			2	SS	42	
			3	SS	53	
			4	SS	45	530
			5	SS	26	
			6	SS	43	
			7	SS	41	520
			8	SS	34	
			9	SS	43	
			10	SS	165	510
			11	SS	115	
497.1			12	SS	90	500
46.5	End of Borehole					490

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

▼ 539.5 in open BH Jan.26/72

7 17 48 28

5 35 44 16

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD–MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 12

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,448 N; 969,638 E. ORIGINATED BY VK
W.P. 48-71-09 BORING DATE Jan 12 & 13, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE
540.8	Ground Level					
0.0	Het.mix.of clayey silt,sand & gravel, occ. seam of silt					540
	Glacial Till		1	SS	17	
			2	SS	10	
			3	SS	51	530
	Brown		4	SS	34	
	Grey		5	SS	22	
	Stiff to Hard		6	SS	18	
			7	SS	31	520
			8	SS	28	
			9	SS	45	510
			10	SS	102	
			11	SS	62	500
			12	SS	61	
			13	SS	88	490
			14	SS	100/6"	480
469.8			15	SS	100/5"	470
71.0	End of Borehole Probable Bedrock					460

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT — 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL x LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR. SA. SI. CL.

▽ 539.6 in open BH Jan.26/72

4 22 54 20

0 0 90 10

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

13

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 13

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,607 N; 969,537 E. ORIGINATED BY VK

W.P. 48-71-09 BORING DATE Jan. 11 & 12, 1972 COMPILED BY TST

DATUM Geodetic BOREHOLE TYPE Pendrill & ConeTest CHECKED BY

SOIL PROFILE ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
539.6	Ground Level					
0.0	Het.mix.of clayey silt sand and gravel.		1	SS	6	
			2	SS	8	
	Glacial Till		3	SS	14	530
			4	SS	25	
	Brown / Grey		5	SS	21	
			6	SS	25	520
			7	SS	31	
			8	SS	21	
			9	SS	34	510
			10	SS	52	
			11	SS	49	500
			12	SS	44	
			13	SS	27	490
			14	SS	157	480
			15	SS	125/5"	470
465.5			16	SS	100/1"	
74.1	End of Borehole Probable Bedrock					460

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT — 20 40 60 80 100

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

w_P w w_L

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR.SA.SI.CL.

▼ 538.6 in open BH Jan.26/72

3 21 50 26

3 14 42 41

15 20 5 10 % STRAIN AT FAILURE

FORM OD–MT 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 14

FOUNDATION SECTION

JOB 72-11001 LOCATION Co-ords. 15,880,594 N; 969,609 E. ORIGINATED BY VK
W.P. 48-71-09 BORING DATE Jan. 7 & 10, 1972 COMPILED BY TST
DATUM Geodetic BOREHOLE TYPE Pendrill & Cone Test CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
541.0	Ground Elevation					
0.0	Het.mix.of clayey silt sand and gravel.		1	SS	29	540
			2	SS	24	
	Glacial Till		3	SS	35	
	Brown		4	SS	28	530
	Grey		5	SS	31	
	Very Stiff to Stiff		6	SS	29	
			7	SS	27	520
			8	SS	26	
			9	SS	58	510
			10	SS	73	
			11	SS	74	500
			12	SS	47	
			13	SS	40	490
			14	SS	120	480
			15	SS	100/4"	470
462.4			16	SS	100/1½"	
78.6	Shale Bedrock					
457.4	Sound		17	BXL	100%	460
83.6	End of Borehole					450

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT — 20 40 60 80 100

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

WATER CONTENT % — 10 20 30

BULK DENSITY γ P.C.F.

REMARKS

GR. SA. SI. CL.

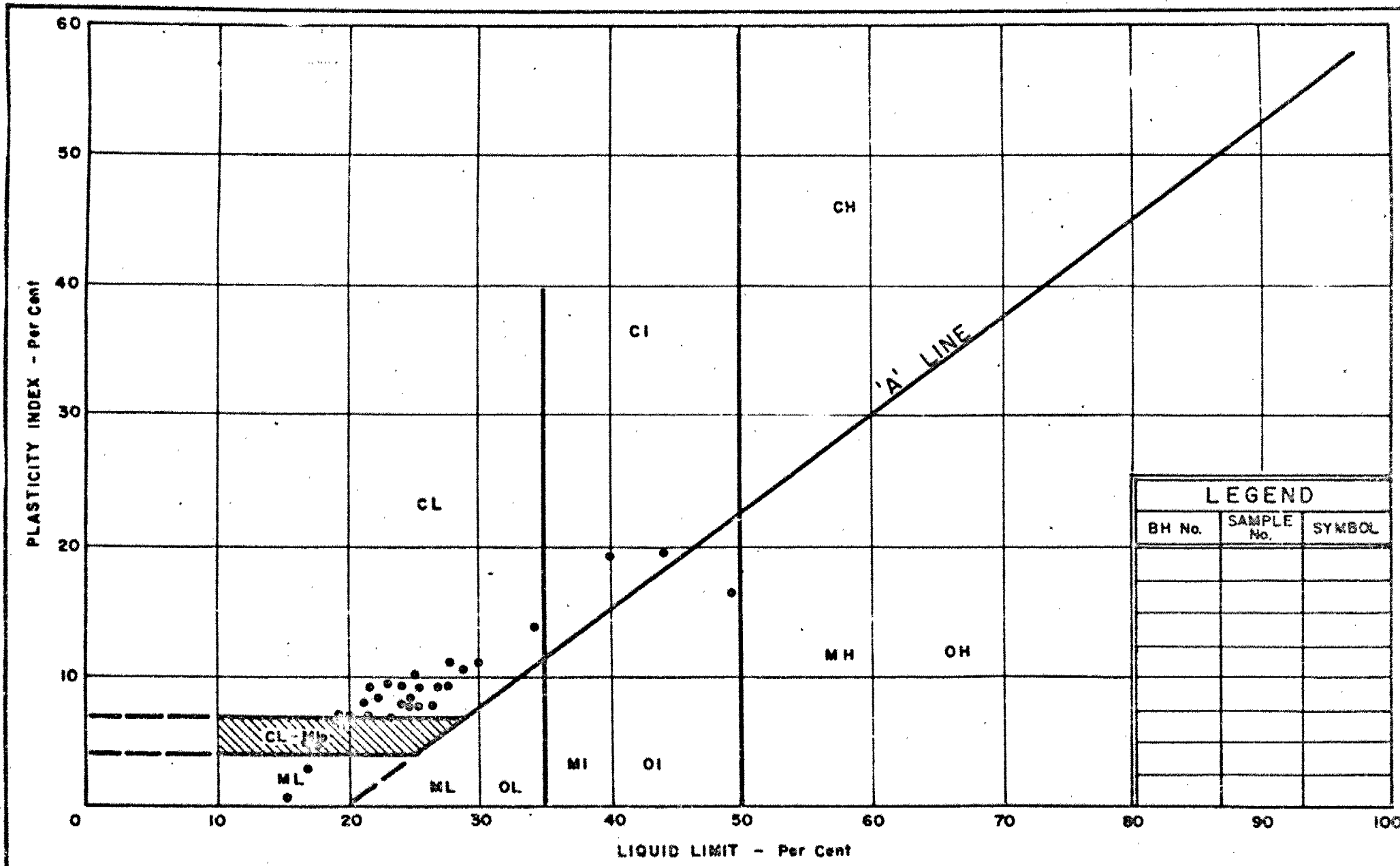
▽ 540.5 in open BH Jan.26/72

4 32 44 20

3 25 40 32

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OB–MT–252
REVISED JUL[illegible] 1966

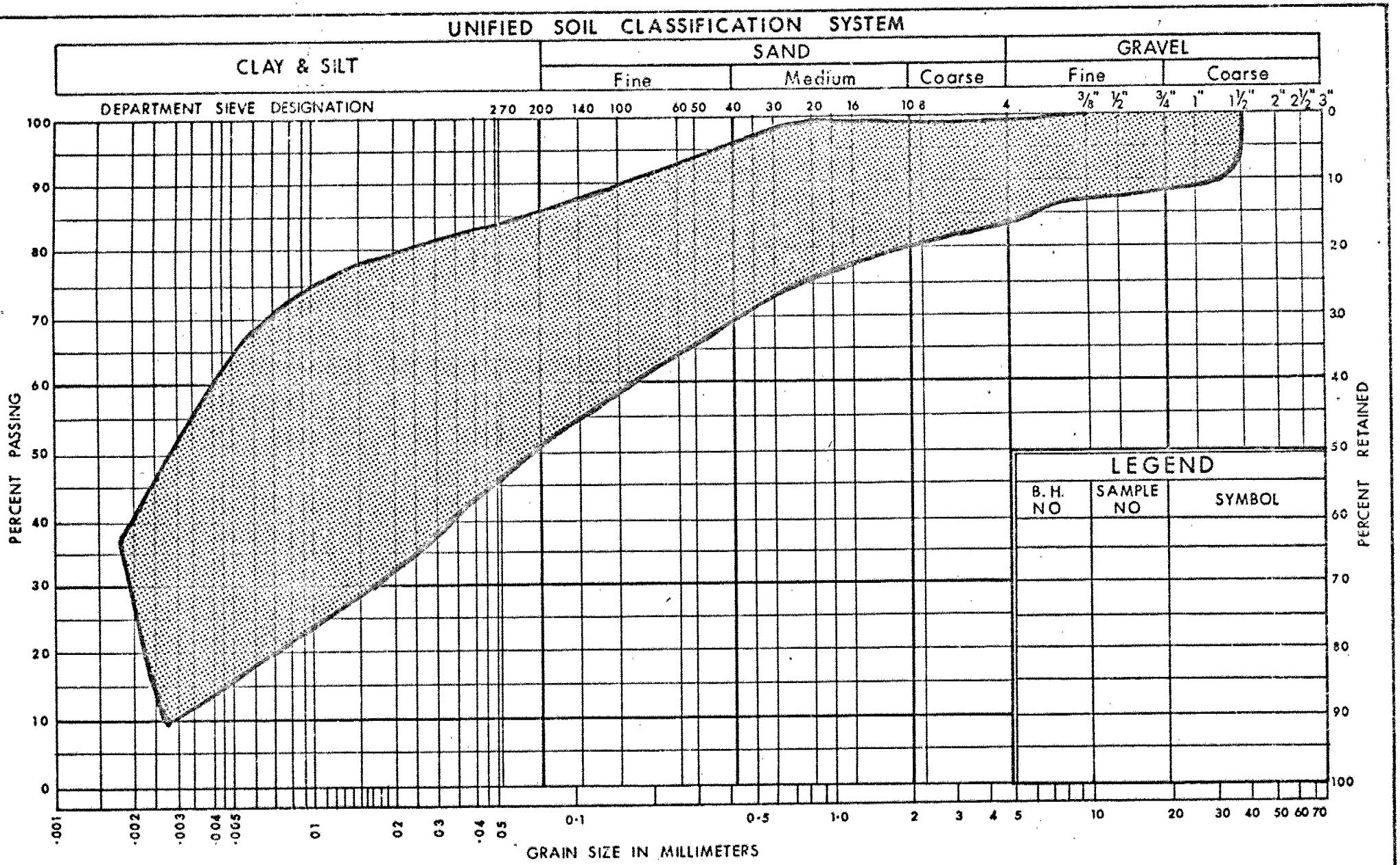


DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART
GLACIAL TILL
HET. MIXTURE CLAYEY SILT, SAND, & GRAVEL

W.P. No. 48–71–09
JOB No. 72–11001
FIG. No. 1

FORM OD-MT-
REV. AUG. 1971



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

ONTARIO

DESIGN SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

GLACIAL TILL

HET. MIXTURE CLAYEY SILT, SAND, & GRAVEL

W.P. No. 48-71-09

JOB No. 72-11001

FIG. No. 2

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta \sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
	IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
	IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3·1416
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. M. Devata,
Sup. Foundation Engr.

FROM: K. W. Ingham

ATTENTION:

DATE: March 10, 1972

OUR FILE REF.

IN REPLY TO

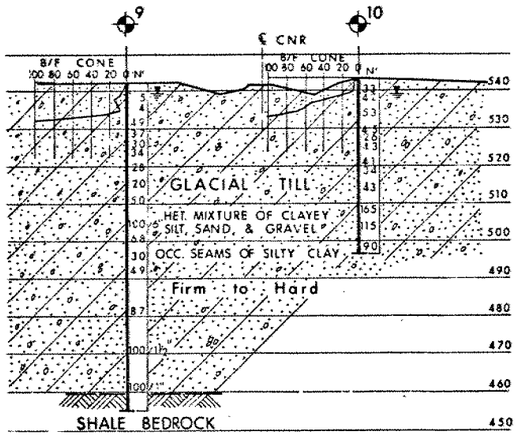
SUBJECT: Foundation Investigation 72-11001;
Bridge Footing; Highway 427

Three boreholes, Nos. 2, 9 and 14 intersected bedrock at the site. The rock is a dark grey shale with thin bands of calcareous shale and beds of limestone and siltstone. A small amount of till was recovered at the top of each hole and this appears to be underlain by 0.5 to 3.0 ft. of moderately fractured bedrock and then the undisturbed shale. The depth to bedrock and corresponding elevation for each hole is given below.

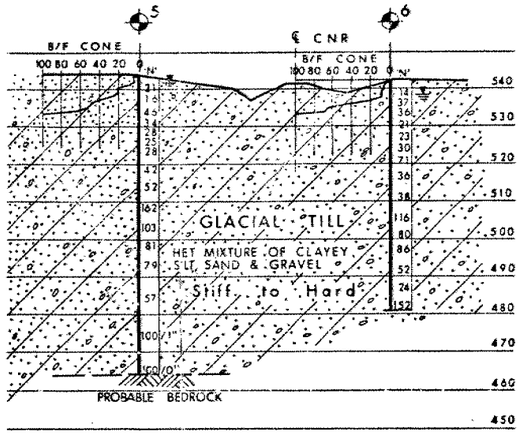
Hole No.	Depth to Bedrock - ft.	Bedrock Elevation - ft.
2	77.0	462.4
9	82.5	459.5
14	78.6	462.4

K. W. Ingham,
Geologist.

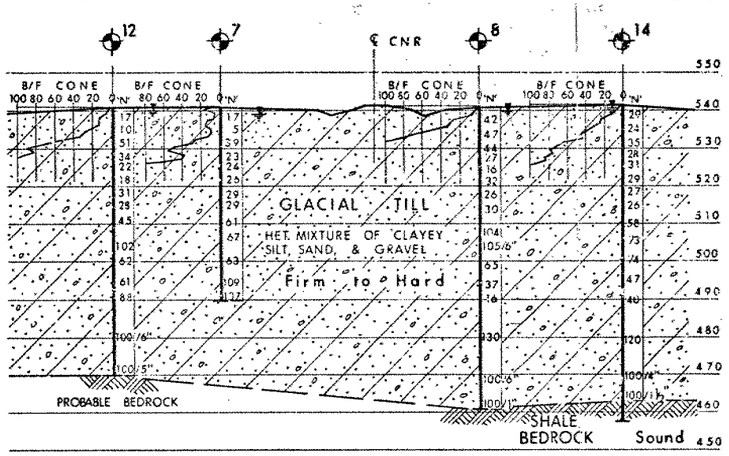
KWI:mv



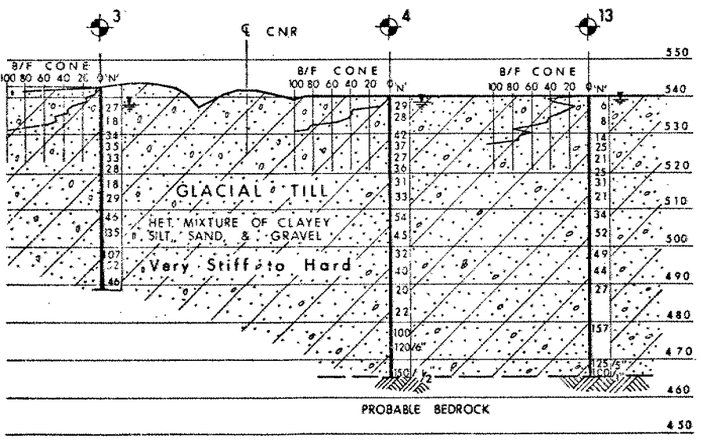
A-A



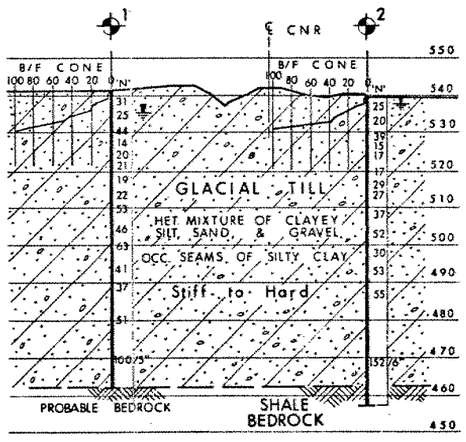
C-C



B-B

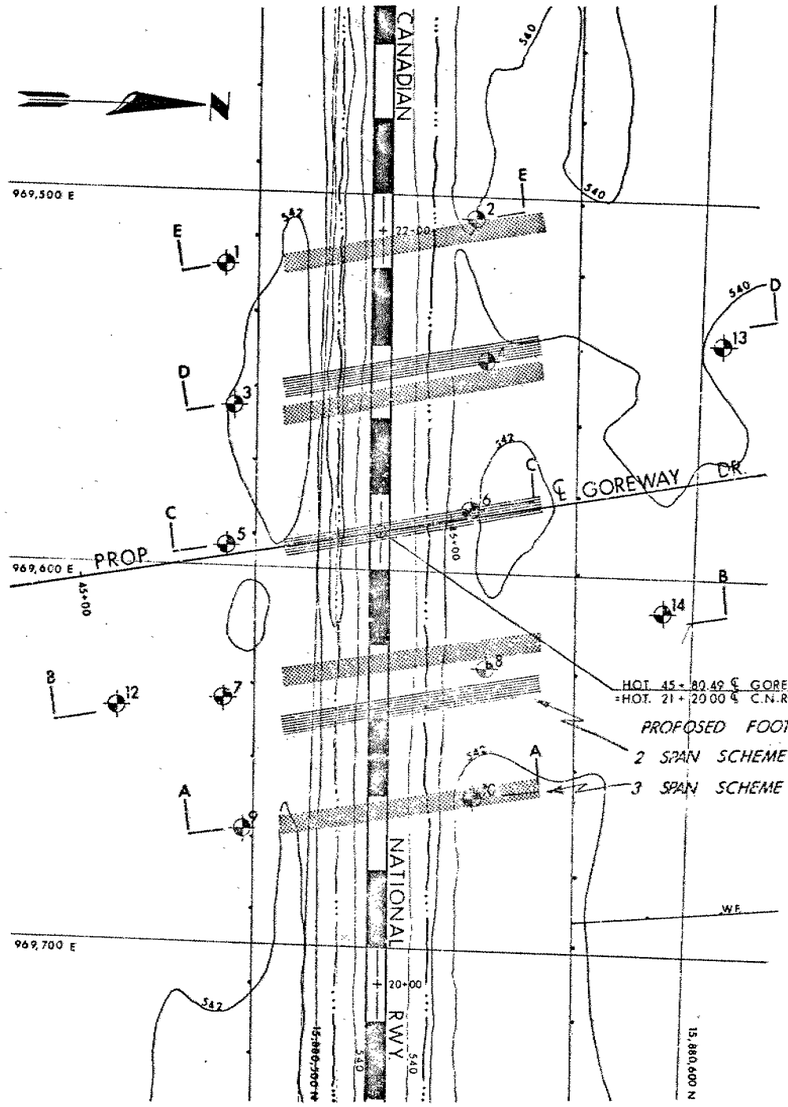


D-D

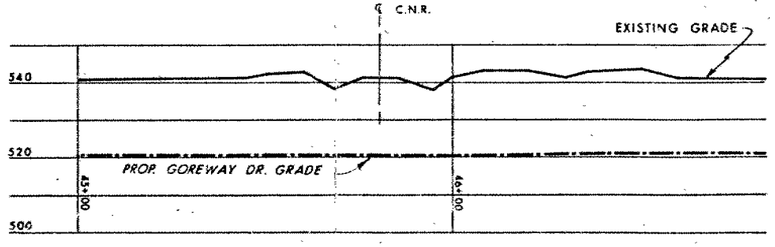


E-E

SECTIONS
SCALE
0 20 40 60 FT



PLAN
SCALE
0 20 40 60 FT



PROFILE
SCALE
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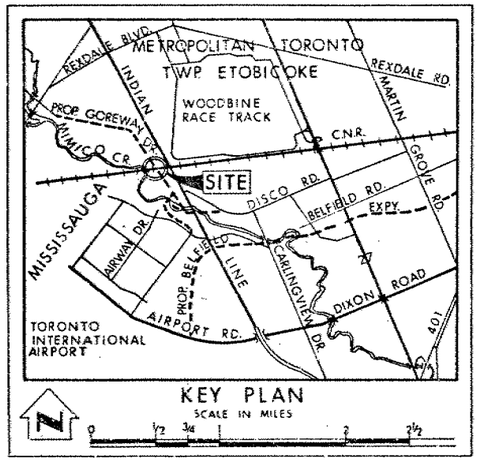

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. 26 JAN., 1972

NO.	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	540.5	15,880,474	969,519
2	539.4	15,880,540	969,505
3	540.0	15,880,477	969,557
4	540.0	15,880,544	969,543
5	544.0	15,880,476	969,594
6	542.7	15,880,541	969,582
7	541.0	15,880,478	969,636
8	540.2	15,880,547	969,625
9	542.0	15,880,484	969,670
10	543.6	15,880,545	969,660
11	540.8	15,880,485	969,638
12	539.6	15,880,607	969,537
14	541.0	15,880,594	969,609

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS			
DATE	BY	DESCRIPTION	

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH FOUNDATION OFFICE

CANADIAN NATIONAL RAILWAY

~~HIGHWAY NO.~~ PROP. GOREWAY DRIVE DIST NO. 6
CO. PEEL TOWN OF MISSISSAUGA
TWP. LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D V.W.	CHECKED	W.P NO. 48-71-09	M&T DRAWING NO. 72-11001A
DRAWN TSS	CHECKED	JOB NO. 72-11001	
DATE 29 FEB. 1972		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
West Bldg.

FROM: G.C.E. Burkhardt,
Bridge Planning Office,
90 Floral Parkway.

ATTENTION: Mr. M. Devata
Supervising Foundation Eng.

DATE: December 29, 1971.

OUR FILE REF.

IN REPLY TO

SUBJECT: Hwy. 427 O'Pass at Campus Road, W.P. 273-66,
Hwy. 427 O'Pass at Disco Road, W.P. 280-65
Hwy. 427 N to E & W ramps, over Goreway Drive,,W.P. 387-65,
Mimico Creek crossing at Hwy. 427, W.P. 48-71-11,
Mimico Creek crossing at American Drive, W.P. 48-71-08,
72-11001 C.N.R. grade-separation at Goreway Drive, W.P. 48-71-09,
Proposed Hwy. 427, District 6, Toronto.

Attached please find two prints each of the 1" = 40' scale plans covering above sites.

Shown in different colours are probable footing locations for the structures involved. Related profiles are also enclosed.

At Campus Road a reinforced concrete rigid frame has been agreed upon.

Hwy. 427 will overpass Disco Road by means of a twin three span spillthrough type of bridge. This design being intended to avoid at the initial stage of construction the high cost of a wide closed median. The gap thus originated will be ultimately closed when higher volumes of traffic warrant the implementation of the 8 lanes scheme planned for future Hwy. 427.

Traffic from Hwy. 427 N to E & W ramps will be carried over Goreway Drive by a separate structure having span arrangement similar to the one previously mentioned.

Mimico Creek is planned to cross Hwy. 427 and associated N to E & W ramps through a structure situated under Disco Road and Goreway Drive and which will basically follow their proposed alignment. Hydrology studies are now being conducted to evaluate the most convenient sort of water crossing to be provided at this place. Some of the alternatives being contemplated are: twin concrete box culvert, single concrete box culvert and single concrete rigid frame.

...2

RE: Proposed Hwy. 427,
District 6, Toronto

As shown on the enclosed plan, partial conflict can arise between the foundation work required for the construction of the south end piers of above two bridges and the proposed Mimico Creek structure. To overcome such problem a shift of these piers will be taken in account if present conditions still prevail by the time detailed plans and profiles of these grade-separations become available.

American Drive will cross Mimico Creek either by using a concrete box culvert or a single span concrete rigid frame.

At Goreway Drive a subway condition (2 or 3 span bridge) appears to be desirable. However, due to anticipated severe flooding conditions in the vicinity of this road an overhead solution should also be considered and, therefore, it hasbeen included as a second alternative.

Would you kindly arrange to have a foundation investigation performed at the mentioned sites.

M.D. Bendayan,
REG. BRIDGE LOCATION ENGINEER,
for:
G.C.E. Burkhardt,
REG. BRIDGE PLANNING ENGINEER.

MDB/co
Attach.

c.c. R. Fitzgibbon
J. Anderson