

DIST. 2 REGION _____

W.P. No. 817-93-01
818-93-01

~~818-93-0~~

CONT. No. _____

W. O. No. _____

STR. SITE No. 23-117

HWY. No. 401

LOCATION Hwy 401 @ Horner Creek
 (EBL & WBL)

(EBL & WBL)

No of PAGES - 1

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____



Ministry
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- LAB REPORT NO.
WP 317/318-93-01

FILE No. _____ DATE _____

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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

**ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION**

WP 817/818-93-01 DIST 31
HWY 401 STR SITE 23-117/1/2

Hwy. 401 and Horner Creek Bridge EBL/WBL

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GEOCRES 40P2-55

DATE

AUG 22 1995

FOUNDATION INVESTIGATION REPORT
FOR
Hwy. 401 and Horner Creek Bridge EBL/WBL
WP 817/818-93-01, Site 23-117/1/2
Hwy. 401, District 31, London

Introduction

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed that the existing twin structure located at Hwy. 401 and Horner Creek be widened in the median due to traffic staging. This report contains factual information obtained from this investigation pertaining to structural foundations and earthworks.

Site Description

The site is located along Hwy. 401 approximately 35 km west of Woodstock at the intersection of Horner Creek within Blenheim Township, County of Oxford.

The topography of the area consists of undulating or gently rolling landscapes with the highway lined with rows of maple trees. While not extensive, there are some marshes in the area. The region is known for its favourable soil conditions for agriculture. In the immediate vicinity of the Horner Creek bridge is a golf course to the south and a marsh to the north. A gas pipeline crosses Horner Creek 50 m south of the existing bridge location. At this location Hwy. 401 has two lanes with wide paved shoulders along the bridge. The natural ground level is approximately 288.5 m, with the ground surface of the highway at an elevation of 291.3 m whilst the stream bed is 288.8 metres. Water levels in the creek were at an approximate elevation of 289.3 metres at the time of the investigation. This is expected to rise considerably during spring thaw periods.

Physiographically the site is located in the geological domain known as the Oxford Till Plain. The area is characterized with gentle slopes, good drainage, medium texture, and lack of extreme stoniness, making this favourable soil. It consists of London and Berrien Sandy Loams. With the morainic soils being clay loams while the alluvial soils in the hollows are silt loams and sandy or gravelly loams. The tills are a pale brown, calcareous loam in which Middle Devonian limestone is the dominant material, although grey or pale brown dolostone is also abundant.

Field Investigation

The fieldwork for the investigation was carried out on 95 01 10 and 95 01 11 and consisted of two boreholes located within the median of Hwy. 401 at the abutment location of the existing structures. All boreholes were advanced to depths of 34 metres.

The boreholes were advanced using conventional solid stem augering techniques. Track and truck mounted continuous flight auger drilling rigs were employed for the operation.

In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 6 m and at 1.5 m intervals thereafter. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). A Cone Penetration Test was conducted at each borehole location.

All subsoil samples were identified in the field and returned to the laboratory for further examination and applicable testing.

Water levels were monitored throughout the duration of the investigation in open boreholes. All boreholes were backfilled upon completion of the fieldwork.

Survey information related to the location and elevation of boreholes was provided by the Surveys and Plans Office in Southwestern Region, London.

Laboratory Analysis

The following laboratory tests were carried out on select soil samples:

1. Atterberg Size Distributions
2. Grain Size Distributions
3. Unit Weights
4. Natural Moisture Contents

Laboratory test results are given in the following section of this report and are illustrated on figures and borehole logs included in the Appendix.

Subsurface Conditions

General

The current investigation confirmed the results of the borings conducted in 1958, however boreholes were terminated at greater depths to reach end bearing material. A borehole was placed on each side of the creek at the abutment locations within the median. As a consequence 2.9 m of a sand and gravel, trace silt with occasional cobbles and boulders of fill was encountered from the surface. This depth corresponds with the elevation of Horner Creek. Underneath the fill was 0.8 m thick organics containing some wood particles. Sandwiched between another 1.5 m - 1.9 m thick organic layer and the previously described organic layer is a 1.5 - 1.9 m non-cohesive sandy silt, trace clay. In turn lying beneath the above layers was encountered a 15.2 - 16.8 m thick fine to medium coarse silty sand which had random layers of gravel. Underlying the above is a cohesive clayey silt, trace sand glacial till containing layers of a trace to some gravel. This deposit

was found to extend beyond the termination depths of the investigation and had a stiff to hard consistency. In one borehole an additional layer of silty sand, some gravel, trace clay was encountered within the clayey silt glacial till with a thickness of 3 m and a very dense state. An end bearing stratum was confirmed within the last cohesive deposit.

Cone penetration tests conducted from the natural ground surface beyond the fill at both abutment locations were found to extend down to elevations of 278 m to the west and 276 m to the east. Zones of gravel within the Silty Sand stratum made driving difficult.

The plan and location of borings and the stratigraphical profile are shown on drawing No. 817/8189301-A in the attached appendix. The field and laboratory test results are plotted on the record of borehole sheets and in the appendix of this report. A brief description of the different soil types are given below.

Sand and Gravel, trace Silt (Fill)

As the two boreholes are located within the median a 2.9 m thick fill was encountered. This material consisted of Sand and Gravel, trace Silt with occasional cobbles and boulders. It was of a denser nature than that of the native material below.

Organics, trace Sand

Encountered below the fill were two layers of Organics, being separated by a Sandy Silt. The first layer was 0.8 m thick in both borings and 1.5 - 1.9 m thick underneath the Sandy Silt deposit. Wood particles were encountered within the second organic layer.

Sandy Silt, trace Clay

Sandwiched between the Organic layers was a non-cohesive Sandy Silt, trace Clay which was found to be 1.5 - 1.9 m thick.

Laboratory results of grain size distribution indicated it comprised primarily of 0 % gravel, 43 % sand, 49 % silt and 8 % clay. The standard penetration resistance 'N' values ranged between 3 - 13 blows/0.3 m with a very loose to compact state of denseness.

Fine to Medium Coarse Silty Sand

Underlying the above surficial deposits is a Fine to Medium Coarse Silty Sand which contained pockets of trace to some Gravel and Clay. The layer extended 15.2 - 16.8 m deep.

Results of grain size distribution tests carried out on select samples are shown on Figure 1 in the appendix, in an envelope form. The results summarize Grain Size Distribution Tests carried out on this material throughout the site. The deposit is comprised primarily

of 0 - 50 % Gravel, 14 - 97 % Sand, 0 - 79 % Silt and 0 - 7 % Clay.

In this stratum the Standard Penetration Resistance 'N' values ranged from 1 blow/0.3 m to 74 blows/0.3 m. While the layer had a Very Loose to Dense state of denseness, sampling within this deposit was difficult due to blow up conditions within the sand causing the samples to be disturbed. Thus blow counts are questionable within this layer.

Clayey Silt, trace Gravel, trace Sand (Glacial Till)

A Clayey Silt, trace Gravel, trace Sand (Glacial Till) deposit was found to underlie the non-cohesive layer above. It contained varying proportions of gravel and sand. To the east this layer is split into two by a layer of Silty Sand, some Gravel, trace Clay. This stratum extends beyond the scope of the investigation, however at the east abutment the first Clayey Silt layer was 4.6 m and the Sandy Silt layer was 3 m in thickness.

Results of grain size distribution tests carried out on select samples are shown on Figure 2 in the Appendix. The deposit comprised primarily of 0 - 10 % gravel, 1 - 21 % sand, 58 - 70 % silt and 5 - 37 % clay.

The results from the Atterberg Limit Tests performed on the fine fraction of this deposit is summarized as follows:

	<u>Range</u>	<u>No. of Tests</u>
Natural Moisture Content (w)	12 - 15	4
Liquid Limit (W_L)	18 - 34	3
Plastic Limit (W_P)	11 - 17	3
Plastic Index (I_P)	3 - 17	3

From the plasticity chart (figure 3), the layer can be classified as a Clayey Silt of medium plasticity.

In this stratum the Standard Penetration Test Resistance 'N' values ranged from 27 Blows/0.3 m to > 120 Blows/0.3 m indicating a Stiff to Hard consistency.

Groundwater Conditions

Ground water levels obtained at the time of the investigation revealed that the groundwater table is generally at the elevation of the water in the creek. This corresponds to depths ranging from 2 to 3 metres below the highway ground surface. However, after the completion of the investigation during a period of a large amount of rain and surface runoff the creek level rose considerably overflowing the surrounding banks to an approximate elevation of 289 metres.

Groundwater levels are subject to seasonal fluctuations with conditions varying from those in this report.

Discussion and Recommendations

It is proposed that the existing twin structure located at Hwy 401 and Horner Creek be widened in the median due to traffic staging. The twin bridge is a two span (13.7 m, 13.7 m) reinforced concrete tee beam structure. The approach fills and existing retaining walls are in the order of magnitude of 2.5 to 3 m.

The existing twin bridge reinforced concrete tee beam structures appears to be performing satisfactorily. According to plans provided by the structural section, southwestern region the bridge foundations consist of steel H piles driven to depths of 16.7 metres to an elevation of approximately 272.2 metres. These piles do not appear to be driven all the way down to end bearing material. The pile caps are located below the natural ground surface at an elevation of 285.5 metres placed 3.5 m below ground surface. Concerns noted in the foundation report for the existing bridge included the presence of 3 - 4.5 m of organics, the necessity of protection against scour and the extent of settlements and soil displacement which would be caused by the placement of fill.

A review of the general plans for the existing two structures (Drawing No. D-4192-2) indicate that at both the east and west abutments the pile caps have been extended beyond the width of the two structures. They join at the centre of the median leaving no gap. The presence and state of the extended pile caps should be verified by the structural section. If found feasible they could be utilized and incorporated into the design of the proposed widenings. In any case the foundation recommendations are provided below.

To facilitate the design and construction of the proposed structure foundations the following foundation and geotechnical recommendations are provided in the scope of this report.

1. Structure Foundation
2. Slope Stability
3. Lateral Earth Pressure
4. Construction Consideration

Structural Foundations

Abutment

The presence of a pile foundation for the existing twin structures precludes the use of conventional spread footings. Therefore it is recommended that the widening be supported utilizing steel H bearing piles down to end bearing material.

For the purposes of the O.H.B.D.C., the steel H-piles can be designed using the axial capacities below:

Axial Capacities - Driven Steel H-Piles

<u>Pile Type</u>	<u>Structure</u>	<u>Factored Axial Capacity at ULS (kN)</u>	<u>Factored Axial Capacity at SLS (kN)</u>	<u>Tip El. (m)</u>
HP 310x110	E. Abut.	1600	1150	+/- 262
	W. Abut.	1600	1150	+/- 260
HP 310x79	E. Abut.	1150	890	+/- 262
	W. Abut.	1150	890	+/- 260

As an alternative to end bearing piles, shorter piles can be utilized at the reduced capacities below:

Axial Capacities - Driven Steel H-Piles

<u>Pile Type</u>	<u>Structure</u>	<u>Factored Axial Capacity at ULS (kN)</u>	<u>Factored Axial Capacity at SLS (kN)</u>	<u>Tip El. (m)</u>
HP 310x110	E. Abut.	450	300	+/- 274
	W. Abut.	450	300	+/- 274
HP 310x79	E. Abut.	350	250	+/- 274
	W. Abut.	350	250	+/- 274

Provided below are two schemes for the construction of foundations at the abutments, we believe the first is the most economical.

OPTION 1

In order to minimize any disturbance of the adjacent pile caps it is recommended that the widening be supported utilizing driven bearing piles inside caisson units augered to the bottom of the existing pile cap elevation. This scheme, previously utilized for Cambellville Rd. and Hwy. 40 (WP 95-90-01, District 4) would eliminate the need for road protection since the caissons could be augered from the existing median level. In addition to eliminating the need for any shoring scheme the use of caissons will minimize any disturbance to the existing structural foundations and avoid any serious dewatering problems.

It is proposed to use 508 mm outside diameter metal tube caissons augered to the bottom of the pile cap level at an elevation of approximately 285.5 m with steel H 310 X 110 bearing piles driven inside the caisson units to a suitable capacity using the Hiley Pile Driving Formula. This scheme assumes settlement would be low and that the new central structure would be dowelled into the sides of the existing decks. The steel tube caissons would be filled with tremie concrete below the water table and 30 MPa concrete above. The steel H piles should be equipped with driving shoes and driven with an energy not less than 40 Kilojoules to toe elevations of about 285.5 m.

OPTION 2

An alternative scheme would be to excavate and drive piles at the elevation of the adjacent pile caps. As this is below the level of the creek a major dewatering scheme will be required. This could be accomplished with the use of sheet piles to form a coffer dam and sump pumping the inside. For dewatering purposes, the minimum depth of penetration of the sheet pile below the base of the excavation should be equal to the depth of the excavation below the prevailing groundwater table, in order to prevent piping. Due to the presence of sands the disturbance of the pile cap base will be a concern. Tremie concrete may be required at the pile cap base if the pile cap cannot be constructed in the dry. The sheet piling would also serve as road protection being placed inside the excavation adjacent to the travelled highway, retaining the existing approach embankments during construction.

PIERS

It is also recommended to utilize steel H-piles driven to end bearing material at the pier location. For the purposes of the O.H.B.D.C., the steel H-piles can be designed using the axial capacities below:

<u>Axial Capacities - Driven Steel H-Piles</u>				
<u>Pile Type</u>	<u>Structure</u>	<u>Factored Axial Capacity at ULS (kN)</u>	<u>Factored Axial Capacity at SLS Type (kN)</u>	<u>Estimated pile Tip El. (m)</u>
HP 310x110	Pier	1600	1150	+/- 260
HP 310x79	Pier	1150	890	+/- 260

As an alternative to end bearing piles, shorter piles can be utilized at the reduced capacities below:

<u>Axial Capacities - Driven Steel H-Piles</u>				
<u>Pile Type</u>	<u>Structure</u>	<u>Factored Axial Capacity at ULS (kN)</u>	<u>Factored Axial Capacity at SLS (kN)</u>	<u>Tip El. (m)</u>
HP 310x110	E. Abut.	450	300	+/- 274
	W. Abut.	450	300	+/- 274
HP 310x79	E. Abut.	350	250	+/- 274
	W. Abut.	350	250	+/- 274

The piers would be constructed within the creek thus requiring a dewatering scheme. This can be accomplished by utilizing sheet piles as described in option 2 above. The pile cap would be placed at an elevation of 286 metres requiring an excavation of approximately 2-3 metres. The pile cap should be constructed in the dry. If required tremie concrete could be utilized to make a suitable founding base.

Based on experience with the existing structure, the presence of pockets of gravel and driving cone tests during the course of the investigation, the native material may present some problems for the installation of piles. It is therefore recommended that pile installations be carefully controlled and monitored employing the hiley dynamic formula in accordance with MTO standards SS103-10 or SS103-11 and assuming an ultimate capacities shown below.

Ultimate Capacity Employing Hiley Dynamic Formula	
<u>Pile Type</u>	<u>Ultimate Capacity (kN)</u>
End Bearing - HP310x110	3450
End Bearing - HP310X79	2670
Short Piles - HP310x110	900
Short Piles - HP310x79	750

To facilitate pile penetration through the fill, it is recommended that the steel H-piles be equipped with reinforced tips.

The pile spacing shall conform with Section 6-11.1 of the O.H.B.D.C. for centrally loaded piles equal load sharing of the deep foundation units can be assumed.

Lateral Earth Pressure on Structure

Free draining material such as Granular 'A' or Granular 'B' is recommended as appropriate backfill to the abutments to prevent hydrostatic pressure build-up. Design parameters of the soil are given below:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (phi)	35°	30°
Unit Weight (kN/m ³)	22.8	21.2
Coefficient of Active Earth Pressure (k _a)		
S.L.S.	0.27	0.33
U.L.S.	0.33	0.4
Coefficient of Earth Pressure at Rest (k _o)		
S.L.S.	0.43	0.5
U.L.S.	0.5	0.58

The earth pressure coefficient at rest is to be used in design if the abutment walls are rigid and unyielding. Weep holes in the abutment walls should be designed to drain any accumulation of water in the backfill.

Slope Stability

The existing forward and side slopes constructed to a grade of 2H:1V appear to be

stable. In consideration of the proposed widening in the median no slope stability problems are anticipated.

Construction Considerations

Longitudinal temporary excavations of depths of 3 m or less below grade may be carried out using 1V:1H slopes provided the excavation materials are not stockpiled near the crest of the slopes.

Pile caps should be constructed in the dry with any loose or organic material at the base removed and replaced with a granular fill.

No dewatering scheme will be required for option 1 at the abutment location. Any local dewatering could be handled by sump pumps. Excavations into the native sands would require an extensive dewatering scheme and should be avoided.

Scour protection should be provided for the central pier and any retaining wall foundations. Due to the underlying sandy deposits care should be taken to prevent any washing out of matter beneath the pile caps and any footings.

The pile caps shall be protected against frost by providing a minimum of 1.2 m earth cover or equivalent frost protection.

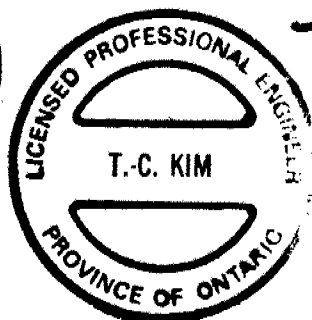
Miscellaneous

The field work for this investigation was carried out under the supervision of M. Michalek, Jr. Foundation Engineer and T. Hickey, Construction Inspector, utilizing equipment owned and operated by London Soils Investigations.

The project was carried out under the general supervision of T. C. Kim, Sr. Foundation Engineer. The report was written by M. Michalek, reviewed and approved by T. C. Kim, Sr. Foundation Engineer.

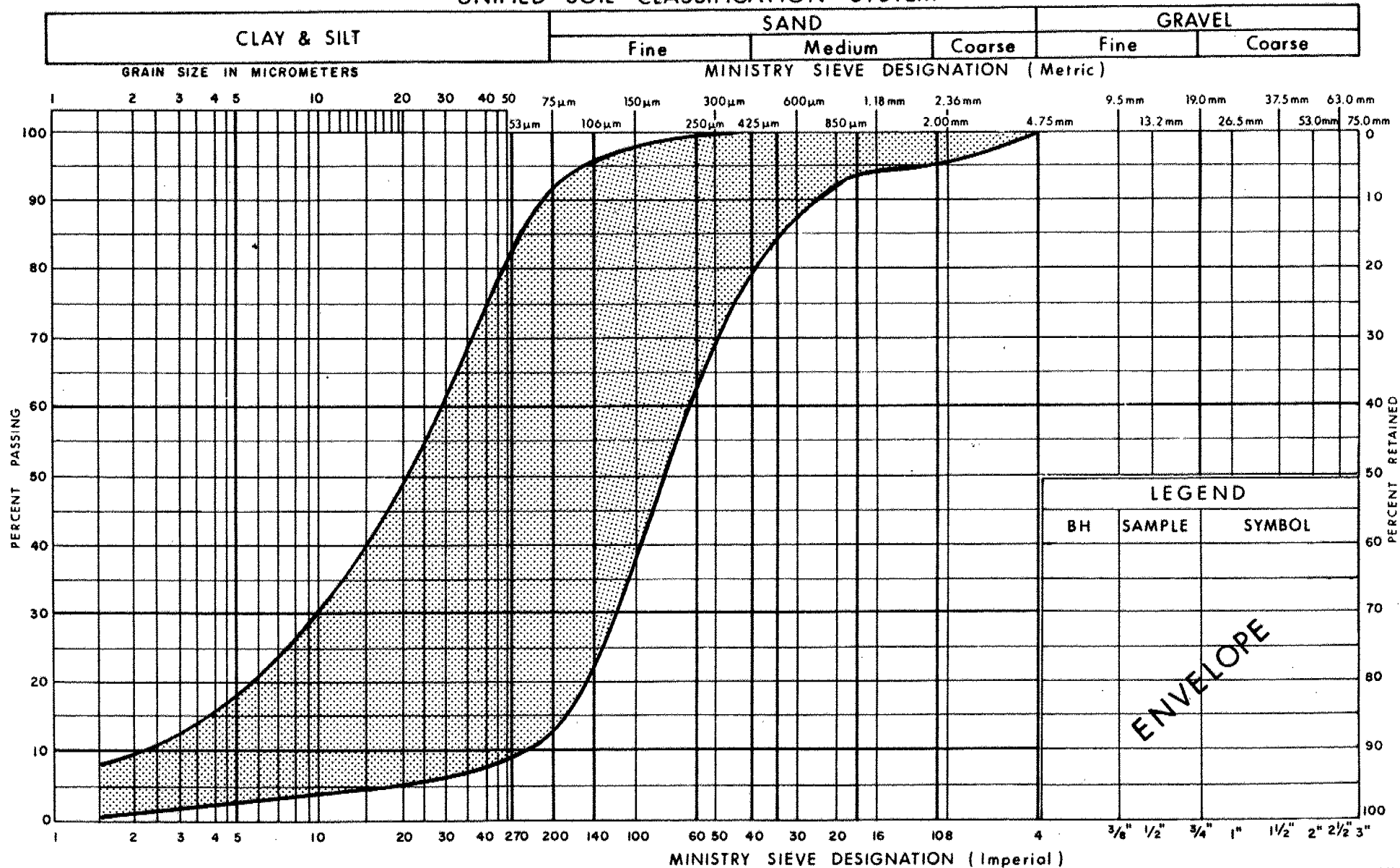


M. Michalek, P. Eng.
Jr. Foundation Engineer



T. C. Kim, P. Eng.
Sr. Foundation Engineer

UNIFIED SOIL CLASSIFICATION SYSTEM



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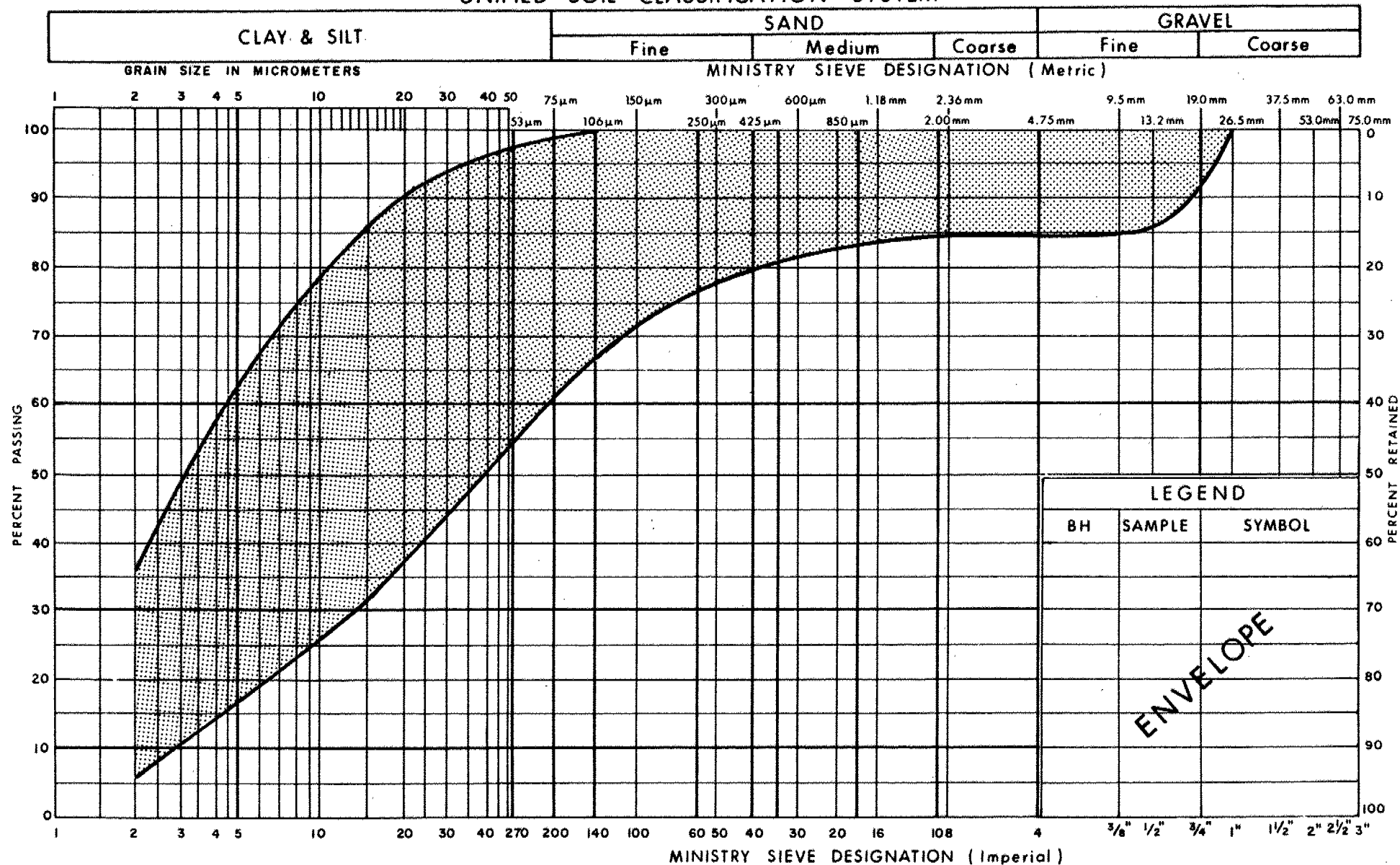
GRAIN SIZE DISTRIBUTION

SILTY SAND

FIG No 1

W P 817/818-93-01

UNIFIED SOIL CLASSIFICATION SYSTEM

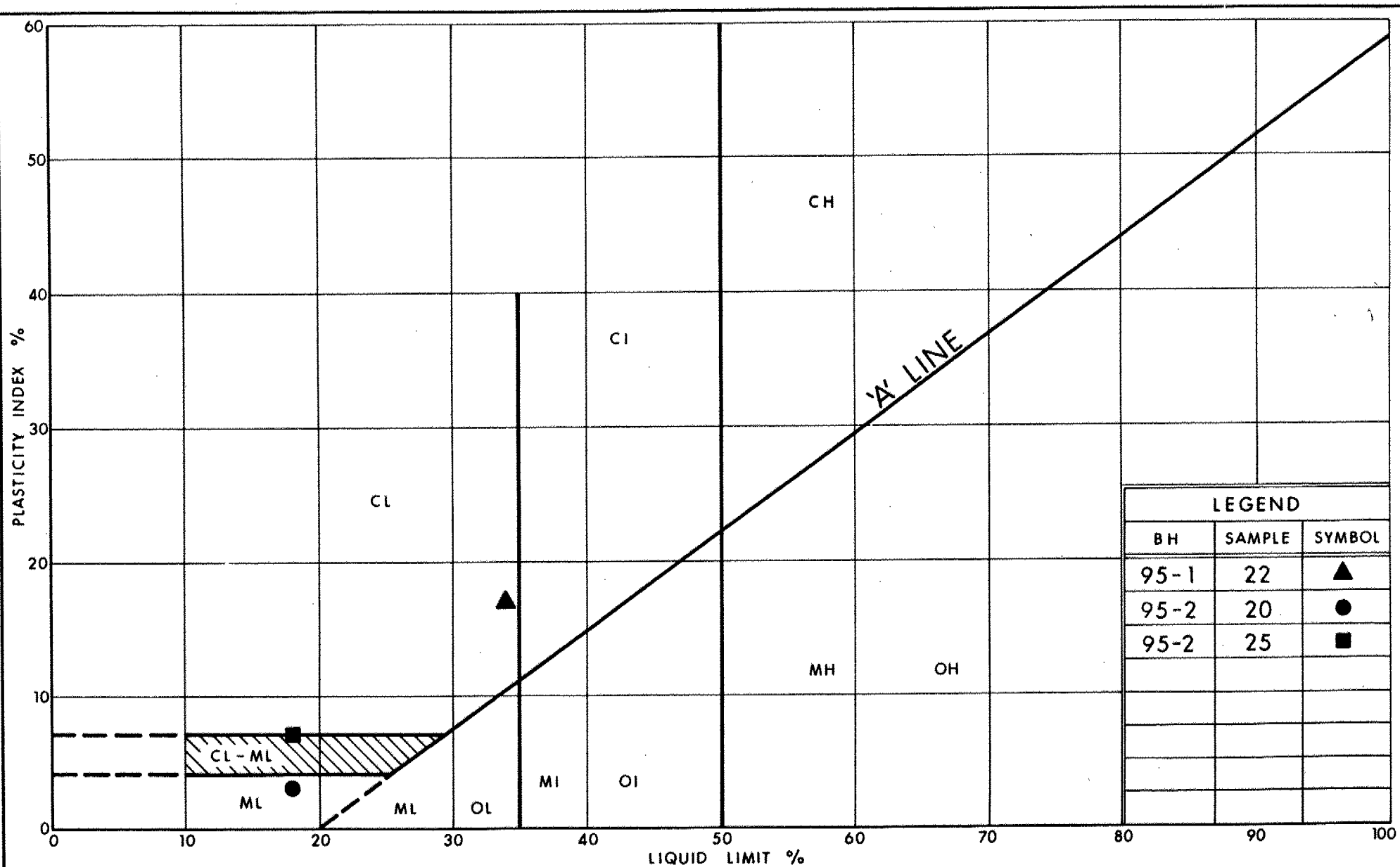


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GRAIN SIZE DISTRIBUTION
CLAYEY SILT, TRACE SAND, TRACE GRAVEL
(Glacial Till)

FIG No 2

W P 817/818-93-01



Ministry of
Transportation

PLASTICITY CHART CLAYEY SILT, TRACE SAND, TRACE GRAVEL (Glacial Till)

FIG No 3

W P 817/818 - 93 - 01

RECORD OF BOREHOLE No 95-1 1 OF 2 METRIC

W.P. 817/818-93-01 LOCATION Co-ords: N 4 785 305.4; E 213 336.7 ORIGINATED BY M.M.
 DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY T.H.
 DATUM Geodetic DATE 95 01 11 CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								20 40 60 80 100							20 40 60		
							• UNCONFINED + FIELD VANE • QUICK TRIAXIAL x LAB VANE										
291.4	Ground Surface																
0.0	Sand and Gravel Trace Silt Occasional Cobbles and boulders (Fill)		1	SS	42												
			2	SS	43												
288.5			3	SS	28												
2.9	Organics, Trace Sand		4	SS	7												
287.7	Dark Brown																
3.7	Sandy Silt trace Clay Very Loose		5	SS	5												
			6	SS	3												
285.8														0 43 49 8			
5.6	Organics, Dark Brown Wood Particles		7	SS	5												
284.3		Brown Gray															
7.1			8	SS	13												
	Trace to Some Gravel Coarse Grained		9	SS	22									3 97 0 0			
			10	SS	14												
	Fine Grained		11	SS	18												
			12	SS	3									0 89 9 2			
	Fine to Medium coarse Silty Sand Very Loose to Dense		13	SS	32												
			14	SS	13												
	With Gravel Coarse Grained		15	SS	28												
			16	SS	23												
	Trace to Some Clay Fine Grained		17	SS	25												
			18	SS	23									0 14 79 7			
267.5																	
23.9	Very Stiff		19	SS	27												
	Clayey Silt, Trace Sand, Trace to Some Gravel (Glacial Till) Hard		20	SS	49									0 1 82 37			
260.9																	

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 95-1 2 OF 2 METRIC

W.P. 817/818-93-01 LOCATION Co-ords: N 4 785 305.4; E 213 336.7 ORIGINATED BY M.M.
DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY T.H.
DATUM Geodetic DATE 95 01 11 CHECKED BY T.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W _p	W	W _L		
260.9	Continued															
30.5	Clayey Silt, Trace Sand, Trace to Some Gravel [Glacial Till] Hard		21	SS	115											
			22	SS	120											16 21 58 5
257.4	Trace Sand		23	SS	120											
34.0	End of Borehole 'N' values within the Silty Sand deposit may be questionable due to blow up and disturbance during sampling.															

RECORD OF BOREHOLE No 95-2 1 OF 2 METRIC

W.P. 817/818-93-01 LOCATION Co-ords: N 4 785 326.5; E 213 371.0 ORIGINATED BY M.M.
 DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY M.M.
 DATUM Geodetic DATE 95 01 10 CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT			UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
291.3	Ground Surface													
0.0	Sand and Gravel Trace Silt Occasional Cobbles and Boulders [Fill]		1	SS	64	/20cm	290	Cone Penetration Test commenced at 31 m from the surface beyond existing fill						
288.4			2	SS	35		288							
287.8	Organics, Trace Sand Dark Brown, Wood Particles		3	SS	41		286							
3.7	Sandy Silt trace Clay Loose to Compact		4	SS	17		284							
286.1			5	SS	13		282							
5.2	Organics, Dark Brown Wood Particles		6	SS	5		280							
284.2			7	SS	7		278							
7.1			8	SS	5		276							
	Fine to Medium Coarse Silty Sand Very Loose to Very Dense		9	SS	1		274							
			10	SS	10		272							
			11	SS	5		270							
			12	SS	8		268							
			13	SS	32		266							
			14	SS	56		264							
			15	SS	74		262							
			16	SS	31		260							
			17	SS	20		258							
			18	SS	21		256							
269.0			19	SS	34		254							
22.3	Clayey Silt Trace Sand Trace Gravel [Glacial Till] Hard		20	SS	72		252							
			21	SS	130		250							
264.4			22	SS	120	/13cm	248							
26.9	Silty Sand Some Gravel Trace Clay Very Dense		23	SS	50	/8cm	246							
261.4							244							
29.9							242							
30.5							240							

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 95-2 2 OF 2 METRIC

W.P. 817/818-93-01 LOCATION Co-ords: N 4 785 326.5; E 213 371.0 ORIGINATED BY M.M.
 DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY M.M.
 DATUM Geodetic DATE 95 01 10 CHECKED BY I.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W _p	W	W _L		
260.8	Continued															
30.5	Clayey Silt Trace Sand Trace Gravel [Glacial Till] Hard		25	SS	135	260										
257.3			26	SS	60	258										
34.0	End of Borehole 'N' Value within the Silty Sand deposit may be questionable due to blow up and disturbance during sampling.															

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

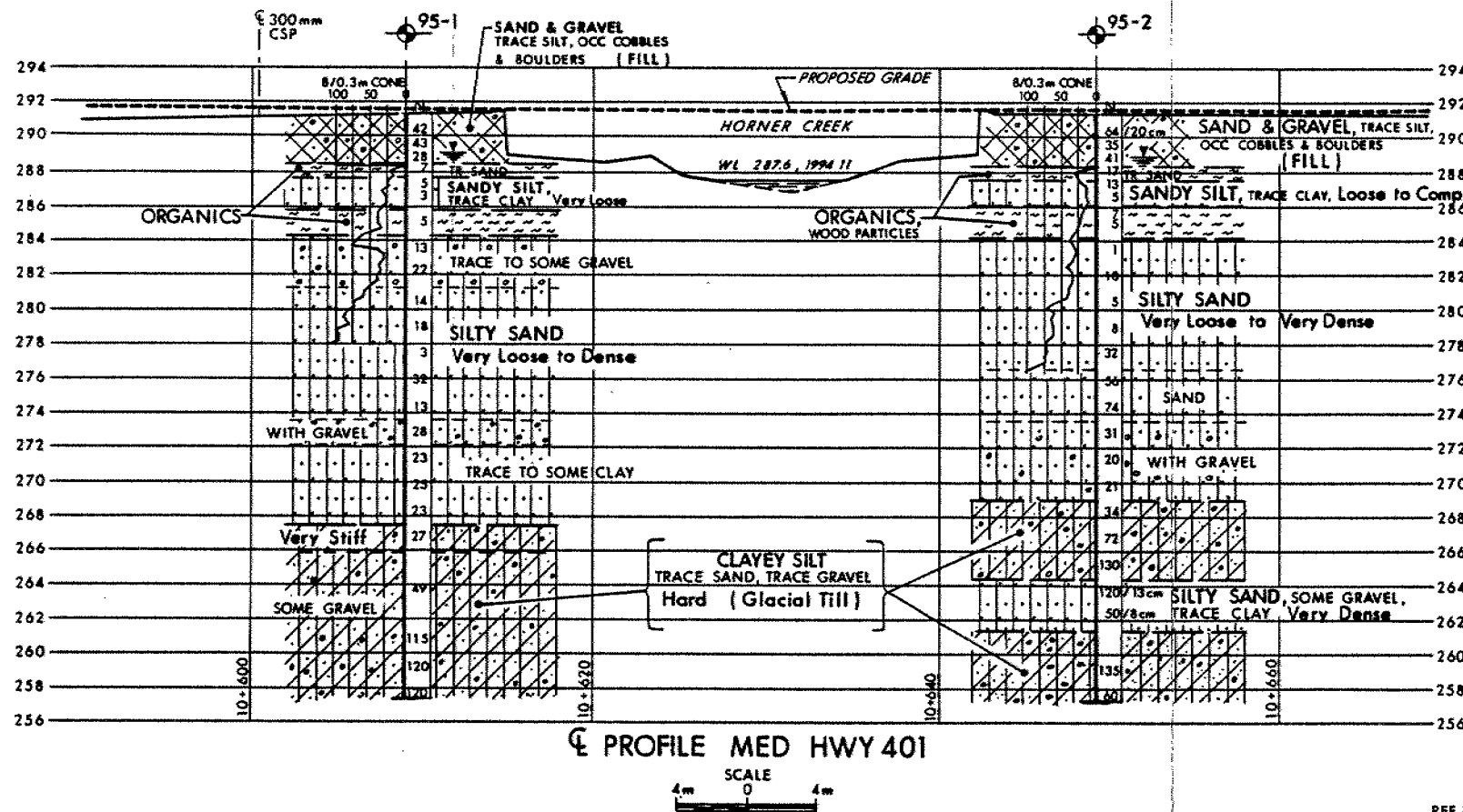
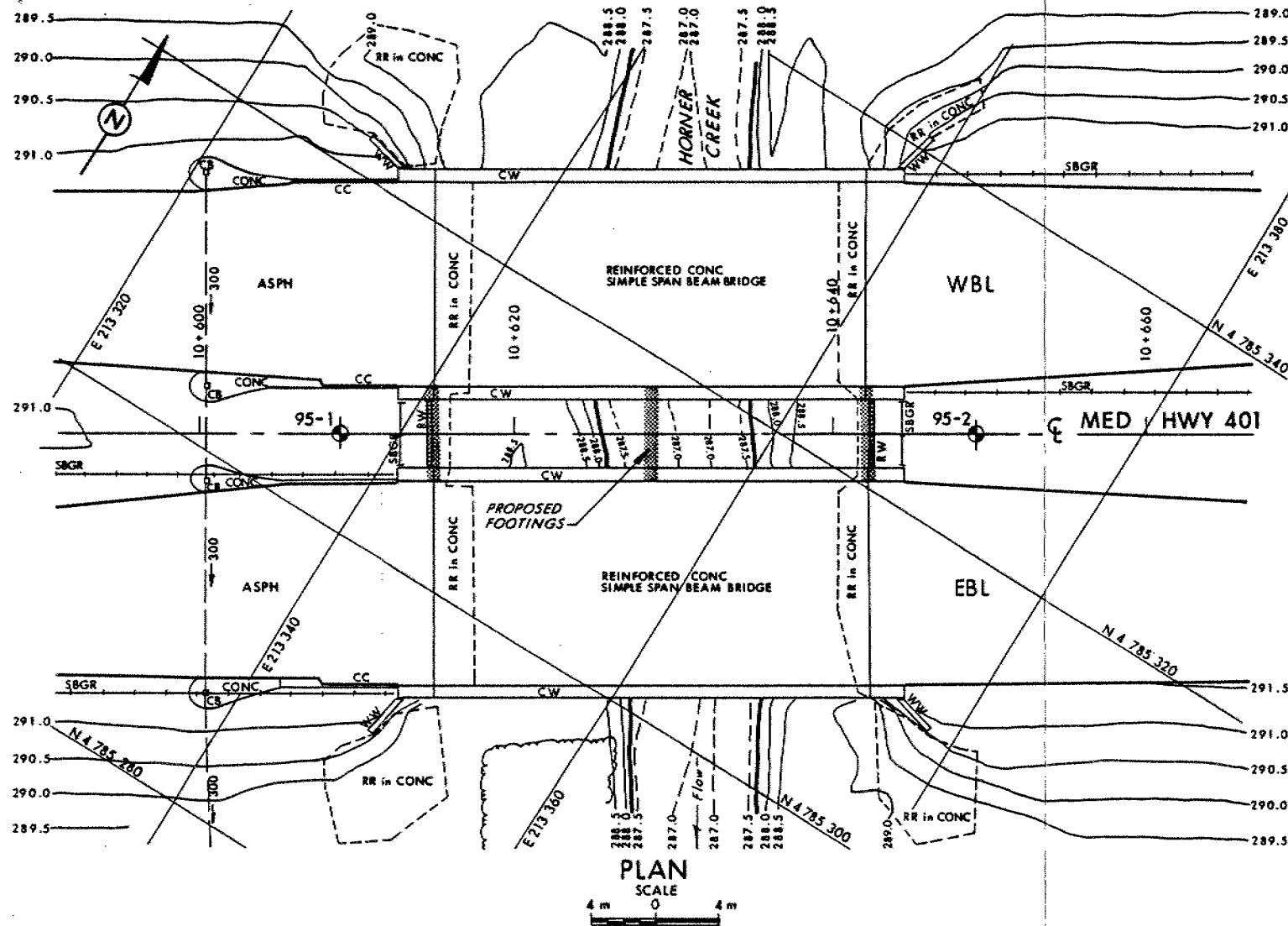
m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

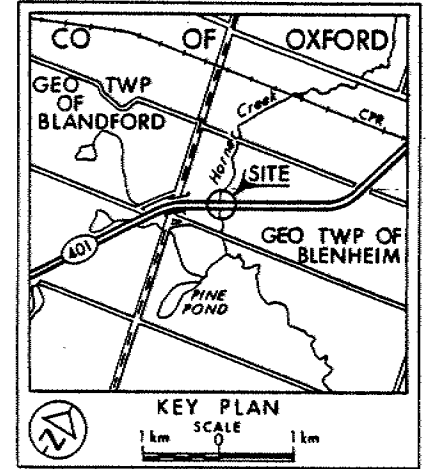
PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 817/818-93-01
HORNER CREEK
BORE HOLE LOCATIONS & SOIL STRATA



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 1995 01

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
95-1	291.4	4785 305.4	213 336.7
95-2	291.3	4785 326.5	213 371.0

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.



REV	DATE	BY	DESCRIPTION
1			
Geocres No 40P2-55			
HWY No 401 EBL & WBL		DIST 31	
SUBWD MM CHECKED M.A. DATE 1995 07 07		SITE 23-11771 & 72	
DRAWN RS CHECKED M.A.		DWG 817/818-9301-A	