

GEOCRES No. 30MS-187

DIST. 4 REGION

W.P. No. 199-77-11

CONT. No.

W. O. No.

STR. SITE No. 10-482

HWY. No. 403

LOCATION RAMP Hwy 403/E
-REW/S OVER North
SERVICE RD (BRIDGE #36)

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 199-77-11 DIST 4
HWY 403 STR SITE 10-482

Proposed Bridge No. 36 at
North Service Road-Hwy. 403/QEW Interchange

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FOUNDATION INVESTIGATION REPORT

For

Proposed Bridge No. 36 at
North Service Road-Hwy. 403/QEW Interchange
W.P. 199-77-11, Site 10-482
District 4, Burlington

INTRODUCTION

A foundation investigation was carried out at the above-captioned site, between 91 09 17 and 91 09 23, for proposed Bridge No. 36 to be constructed across the existing, recently constructed, North Service Road, immediately to the west of the Highway 403/Brant Street interchange.

During this investigation, four boreholes were advanced to depths of 5.4 to 18.2 m.

This report contains the factual information obtained from this investigation.

SITE DESCRIPTION

The existing North Service Road, is located to the north of the QEW, within an open cut, at an elevation approximately 5 m lower than the prevailing ground surface. South of the cut, the prevailing ground surface slopes rapidly to the south towards the QEW and Lake Ontario.

PROCEDURES

The fieldwork, for this project, was carried out by this office between 91 09 17 and 91 09 23. A total of four (4) boreholes, were advanced to depths of 2.3 to 10.3 m, using continuous-flight hollow stem augers driven by a truck-mounted drilling rig equipped with standard soil sampling equipment.

The boreholes were then extended to depths of up to 18.2 m, using conventional diamond drilling (BXL and NQ) techniques in order to penetrate through boulders and to prove bedrock.

Upon completion of coring, piezometers were installed in three of the boreholes, in order to measure the long term groundwater conditions.

The locations of the boreholes were staked out in the field, and their elevations determined by McCormick Rankin Consulting Engineers. However, due to difficulties in having the drilling rig gain access to most of these locations (ie. the boreholes were located on a slope), nearly all of the boreholes had to be moved. Slight changes in the location and elevation of the boreholes were determined by representatives from this office by simple methods (ie. tape measure, compass and hand level).

The soil samples, which were obtained in the field, were examined in the laboratory by visual and tactile methods. Moisture content, Atterberg limits and grain size distribution tests were conducted on several select soil samples. The results of this laboratory testing are included on the Borehole log sheets and on Figures 1 to 3.

SUBSURFACE CONDITIONS

Beneath a thin layer of fill, which was encountered at some locations, the subsoils at the site generally consist of a deposit of a cohesive, heterogeneous mixture of clayey silt with some sand and gravel (glacial till) which is, in turn, underlain by a deposit of a non-cohesive, heterogeneous mixture of sandy silt with some gravel and clay (glacial till). The glacial till directly overlies slightly to moderately weathered, greyish red, Queenston shale bedrock. The groundwater table ranges from a relatively high elevation of 108.8 m on the south side of the bridge to an elevation of 106.2 m at the north end of the bridge.

Details of the soil and groundwater conditions are given on the Borehole log sheets. The following paragraphs are intended to augment these data.

Silty Clay to Clayey silt, trace of Organics (Fill or Possible Fill)

A thin surficial layer of dark brown, silty clay to clayey silt was encountered to a depth of approximately 0.6 m within the ditch, directly adjacent to the

south side of the North Service Road. This material, which was found to contain traces of root fibres and/or occasional topsoil enclosures, has been referred to as 'Fill'.

At the south abutment and above the cut, similar material was found to a depth of 1.4 m. However, since this material had only a slight trace of organics, it has been referred to as 'Possible Fill'.

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

A deposit consisting of a cohesive, heterogeneous mixture of clayey silt containing some sand and gravel was contacted at the ground surface or beneath a thin layer of fill (and/or 'possible fill') to depths of up to 1.4 m at three of the boreholes. At these locations, this deposit was found to extend to depths of up to 12.4 m or elevations of 105.0 to 107.5 m.

Atterberg Limits tests, carried out on four samples of this deposit which were obtained from the boreholes, had liquid limits of 25 to 26 (average of 25) percent and plasticity indices of 11 to 12 (average of 11) percent. These results, which have been plotted on Figure 1, indicate that this soil can be classified as CL or Clayey Silt of low plasticity. Moisture contents from samples obtained from this deposit ranged from 7.5 to 14 (average of 10.5) percent.

A grain size distribution test, carried out on a sample of soil obtained from this deposit, and shown on Figure 2, indicates a relatively well-graded soil with 37 percent silt, 31 percent clay, 24 percent sand and 8 percent gravel-sized particles. These results, coupled with a visual examination indicate that this soil is likely to be of glacial origin and, therefore, may be referred to as glacial till.

Standard Penetration resistances (ie. 'N' values) of 17 to more than 104 blows/0.3 m, which were measured in this deposit, indicate that the clayey silt till is of generally very stiff to hard consistency.

Heterogeneous Mixture of Sandy Silt, some Gravel and Clay (Glacial Till)

At BH 36-C, a non-cohesive, heterogeneous mixture of sandy silt containing some gravel and clay, was found to underlie the cohesive clayey silt till referred to in the previous section, at a depth of 1.4 m. At BH 36-B, similar material was found beneath the thin layer of fill. At the two boreholes, in which it was encountered, this deposit was found to extend to depths of 2.1 m or elevations of 106.3 and 106.9 m.

A moisture content of 9 percent was measured in a sample of soil obtained from this deposit.

A grain size distribution test carried out on a sample of this soil, and shown on Figure 3, indicates a generally well-graded soil with 20 percent gravel, 27 percent sand, 40 percent silt and 13 percent clay-sized particles. These results, coupled with a visual examination, indicate that this soil is of glacial origin and, therefore, may be referred to as glacial till.

Since the 'N' values, which were measured in this deposit, were all greater than 50 blows/0.3 m, these soils are considered to be in a very dense state.

Shale Bedrock

All boreholes reached bedrock (or the assumed bedrock surface) at depths ranging from 2.1 to 12.4 m, or elevations of 105.0 to 107.5 m. The bedrock surface was found to be generally slope towards the north. Recoveries and RQD's generally ranged from 53 to 100 percent and 0 to 80 percent, respectively.

The bedrock consists of greyish red, weak to very weak, Queenston shale containing relatively hard, interbedded, greenish grey siltstone bands. Additional details of the bedrock core samples, which were obtained during this investigation, are included in the Appendix entitled "Rock Core Description".

GROUNDWATER CONDITIONS

Measurements taken in the open boreholes, immediately prior to coring, were generally found to either be dry or with a slight amount of water at the bottom of the hole.

Piezometers were installed in three of the boreholes, immediately after coring, in order to measure the long term groundwater conditions. The water levels, measured in the piezometers, at least 24 hours after their installations, were found to be at elevations of 106.0 to 108.8 m.

It should be noted that the groundwater table is subject to seasonal fluctuations, and is expected to rise during the spring freshet as well as during and immediately following any periods of prolonged heavy rainfall.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to construct Bridge No. 36 across the open cut which carries the newly-constructed, single lane North Service Road. Another bridge (Bridge No. 31 - Ref. No. W.P. 199-77-09), which was also investigated at the same time, will be located immediately to the east of Bridge No. 36.

Bridge No. 36 will consist of a three-span structure, with the innermost span being supported by piers.

On both sides of the bridge, it is proposed to lower the grade by up to 2.0 m, within 50 m of its abutments.

Design Considerations

Abutments

The loads for Bridge No. 36, at the abutment areas, may be supported by spread footings placed on undisturbed clayey silt till. The foundations must be taken below any fill, organic or otherwise unsuitable soil to bear on the undisturbed, very stiff, glacial till. Assuming that such footings are less than 3 m wide, a design value of 600 kPa may be used for the factored bearing capacity at U.L.S. or 300 kPa for the bearing capacity at S.L.S. Type II at the following elevations:

<u>Structure</u>	<u>North Abutment</u>	<u>South Abutment</u>
Bridge No. 36	116.8	116.2

It should be noted, however, that at the north abutment, the soils appear to weaken somewhat below a depth of 5.3 m (elevation of 112.2 m). Therefore, in order to prevent overstressing this underlying zone (which would result in a reduction of the above-stated bearing capacities), footings for the north abutment should not be placed any lower than an elevation of about 113.3 m (based on an assumed 3 m wide square footing).

In areas where subexcavation will be required, the excavated soil should be replaced by well-compacted Granular 'A'.

Piers

It appears that the proposed piers for Bridge No. 36 (and particularly the south pier) will be located on the existing cut slopes adjacent to the North Service Road. Spread footings placed on the very dense sandy silt till may also be considered here. However, in order to reach suitably competent till, it may be necessary to significantly cut back the existing excavated slopes. Therefore, caissons socketed into the underlying bedrock should also be considered.

Recommendations for both spread footings and caissons are given below.

Piers Placed on Spread Footings

The loads from the piers may be supported by spread footings placed below any organic or otherwise unsuitable soils to bear on the very dense sandy silt till. For such footings, a design value of 750 kPa may be used for the factored bearing capacity at U.L.S. This value may be assumed to be at or below the following elevations.

<u>Structure</u>	<u>North Pier (m)</u>	<u>South Pier (m)</u>
Bridge No. 36	109.8	110.1

The bearing capacity at S.L.S. Type II will not govern the design, in this case.

Placement of the footings on the underlying shale should not be considered here, since the footing excavations would extend well below the elevation of the existing roadway and the groundwater table.

Piers Placed on Caissons

The structural loadings for the piers may also be transferred to the underlying sound shale bedrock by means of bored cast-in-place concrete caissons founded on sound, competent bedrock.

For the north and south piers, it is recommended that the following design parameters be used for caissons founded on sound shale bedrock at or below elevations of 106.0 and 105.5 m, respectively.

	<u>Caisson Diameter</u>	
	760 mm	915 mm
Factored Axial Capacity at U.L.S.	4400 kN	6300 kN

In both of these cases, the allowable capacities at S.L.S. Type II would not govern the design.

Caissons should be a minimum diameter of 760 mm to allow for both the clean out of any basal debris and final evaluation of the rock surface in order to confirm the above-stated capacities.

The caissons must be fully cased and socketed at least 0.3 m into the underlying sound shale bedrock. However, depending upon the desired degree of lateral resistance, the length of the socket may have to be increased.

Some groundwater infiltration should be expected, particularly when augering through the sandy silt till, below the groundwater table. Although it is likely that seepage can be controlled by properly filtered sumps, if saturated sand deposits or layers are encountered and seepage becomes excessive, it may be necessary to control groundwater infiltration by using drilling mud or other methods.

Resistance to Lateral Forces

At the pier locations, for design purposes, an unfactored coefficient of friction of 0.45 may be assumed to apply between the base of the footing and the hard clayey silt till or the very dense sandy silt till. However, at the abutments, the unfactored coefficient of friction should be reduced to 0.35 between the base of the footing and the somewhat weaker clayey silt till, which would be encountered.

Approach Areas

Embankment slopes, for approaches less than 8 m high, may be designed at 2H:1V, assuming they are comprised of borrow materials as per MTO specifications.

Backfill Against Abutments

Free-draining granular fill, such as Granular 'A' or Granular 'B', must be used against the abutment walls to prevent the buildup of hydrostatic pressure behind them. The following design parameters, for these granular fills, may be used in accordance with the O.H.B.D.C.:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m^3), γ	22.8	21.2
Coefficient of Active Earth Pressure (K_a)	0.27	0.33
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.5

If the abutment walls are rigid and unyielding, then the earth pressures acting on them should be computed using the earth pressure coefficients at rest. However, if some movement of the abutment walls can be tolerated, then the earth pressures acting on them may be computed using the active pressure coefficients.

Care should be taken to avoid the development of large horizontal pressures, when compacting the backfill behind the abutments. Vibratory compaction equipment, for use behind retaining structures, must be restricted in size, as per current M.T.O. specifications.

Frost Protection

All foundations should have a minimum cover of 1.2 m for frost protection.

Construction Considerations

Excavations and Dewatering

In general, temporary excavations through the clayey silt till to depths of up to 4.0 m will be temporarily stable at slopes of 1:1.

However, at the pier locations, temporary excavations for spread footings, from elevations of 108.3 to 108.6 m, will be required. Such excavations would have to be carried out within the existing cut slopes. The design and construction methodology, proposed by the Contractor, should be submitted to this office for review.

It is expected that any surface water entering the excavation or perched water in any sandy zones within the cohesive (ie. clayey silt till) may be controlled by gravity drainage and/or properly-filtered sumps. If, however, the excavations must extend through the coarser underlying till (ie. sandy silt till) below the groundwater table, and seepage becomes excessive, more extensive groundwater control measures may be required, for even temporary excavations.

Construction of Approaches and Abutment Areas

All of the existing organic-stained fill or other unsuitable soils must be stripped throughout the full-width of the proposed abutment and approach areas. It should be noted, that in our experience at many sites, the thickness and extent of fill can vary significantly. Allowance should, therefore, be made for such variations when estimating stripping quantities etc.

The subgrade preparation, the selection of the fill material and its placement and compaction should be carried out according to OPSS Standards and MTO practice.

MISCELLANEOUS

The field investigation was supervised by A. Hildebrand and J. Blair using equipment owned and operated by Master Soil Investigation Inc.

The project was carried out by J. Blair, Project Foundation Engineer, under the general supervision of B. Iyer, Senior Foundation Engineer.

This report was written by J. Blair, reviewed by B. Iyer, and approved by M. Devata, Chief Foundation Engineer.



John A. Blair

J. Blair, P.Eng.
Project Foundation Engineer

M. Devata

M. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

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 31mm 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 (31mm 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

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

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 = $\frac{c_u}{\tau_r}$

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^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

ROCK CORE DESCRIPTION

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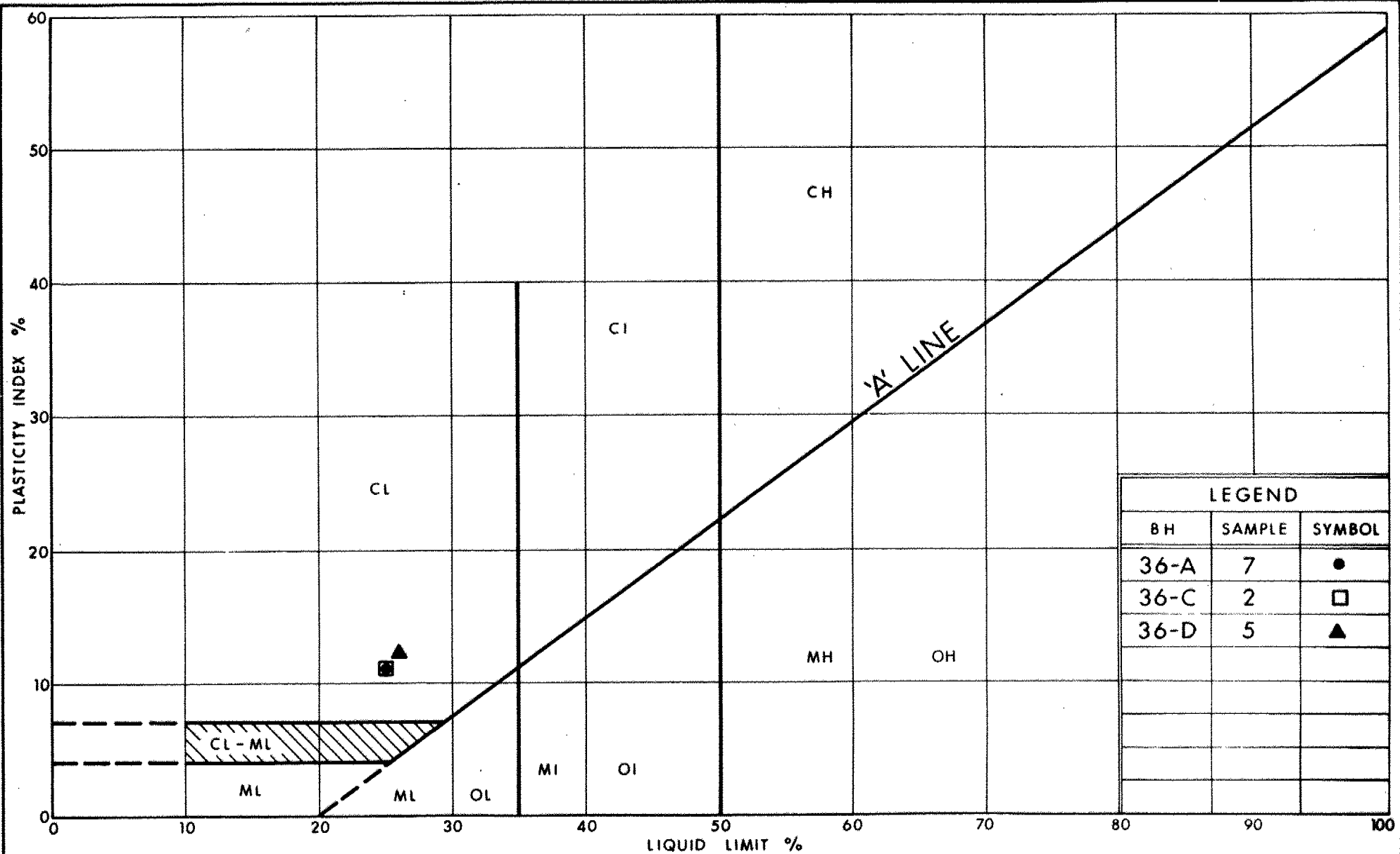
CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
36-A	13	10.54-12.09	57	7	10.54-14.99	SHALE , greyish red, with interbedded greenish grey SILTSTONE (13%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 10.54-10.62 m); fractures close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	14	12.09-13.56	66	21		
	15	13.56-14.99	82	10		
36-B	5	2.41-3.86	77	21	2.41-5.39	SHALE , greyish red, with interbedded greenish grey SILTSTONE (14%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 2.41-2.54 m); fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	6	3.86-5.39	99	60		
36-C	5	2.41-3.94	92	61	2.41-5.49	SHALE , greyish red, with interbedded greenish grey SILTSTONE (23%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 2.41-2.49 m); fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	6	3.94-4.47	86	33		
	7	4.47-5.49	100	80		
36-D	13	10.34-11.84	15	0	10.34-12.42	OVERBURDEN (boulder till).
	14	11.84-12.32	73	0	12.42-18.16	SHALE , greyish red, with interbedded greenish grey SILTSTONE (14%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 12.42-12.47 m and 13.16-13.59 m); fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	15	12.32-13.23	53	28		
	16	13.23-13.59	0	0		
	17	13.59-15.11	93	14		
	18	15.11-16.64	71	13		
	19	16.64-18.16	97	50		

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section



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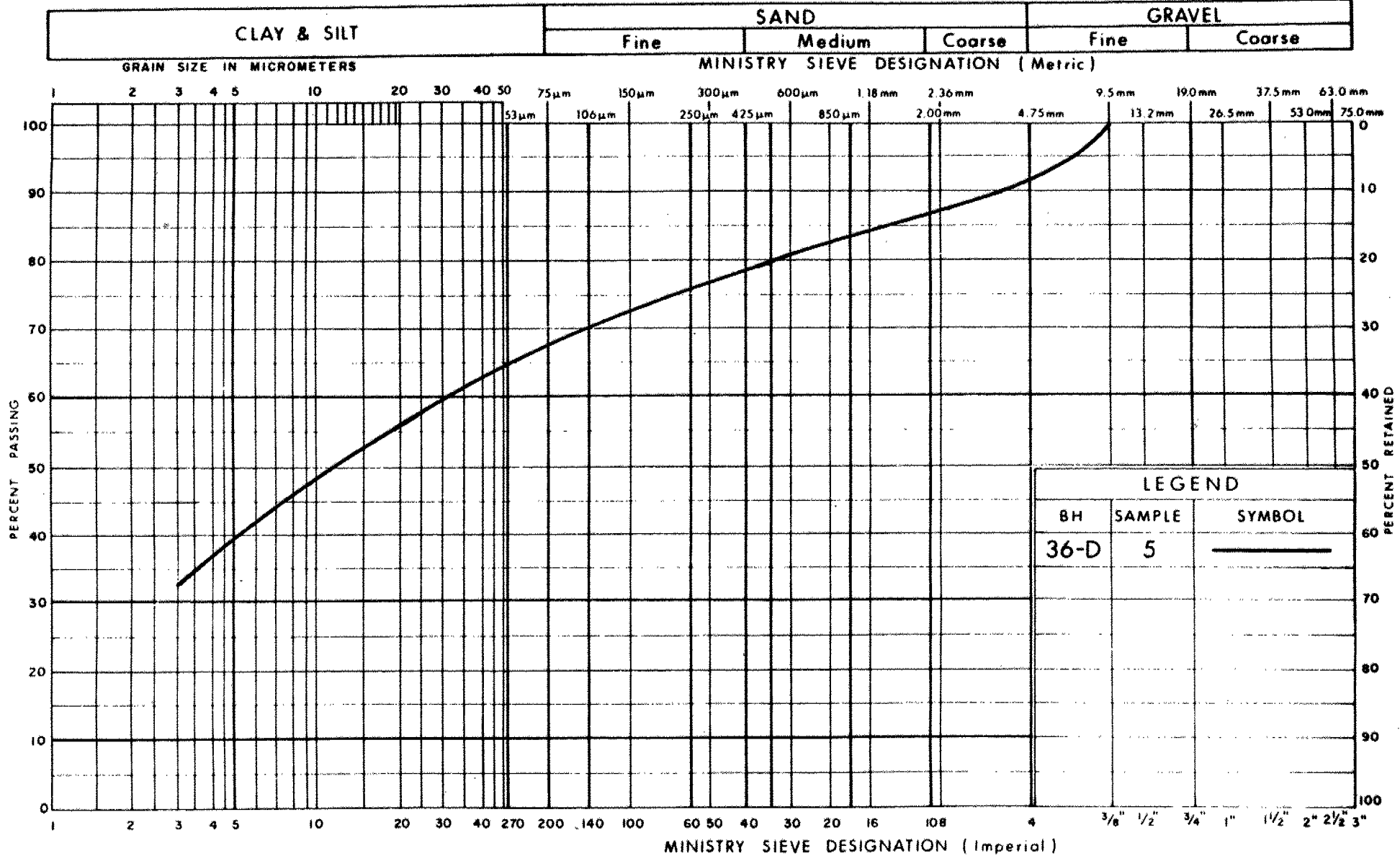
Ontario

PLASTICITY CHART
HETEROGENEOUS MIXTURE OF CLAYEY SILT
SOME SAND & GRAVEL (GLACIAL TILL)

FIG No 1

W P 199-77-11

UNIFIED SOIL CLASSIFICATION SYSTEM



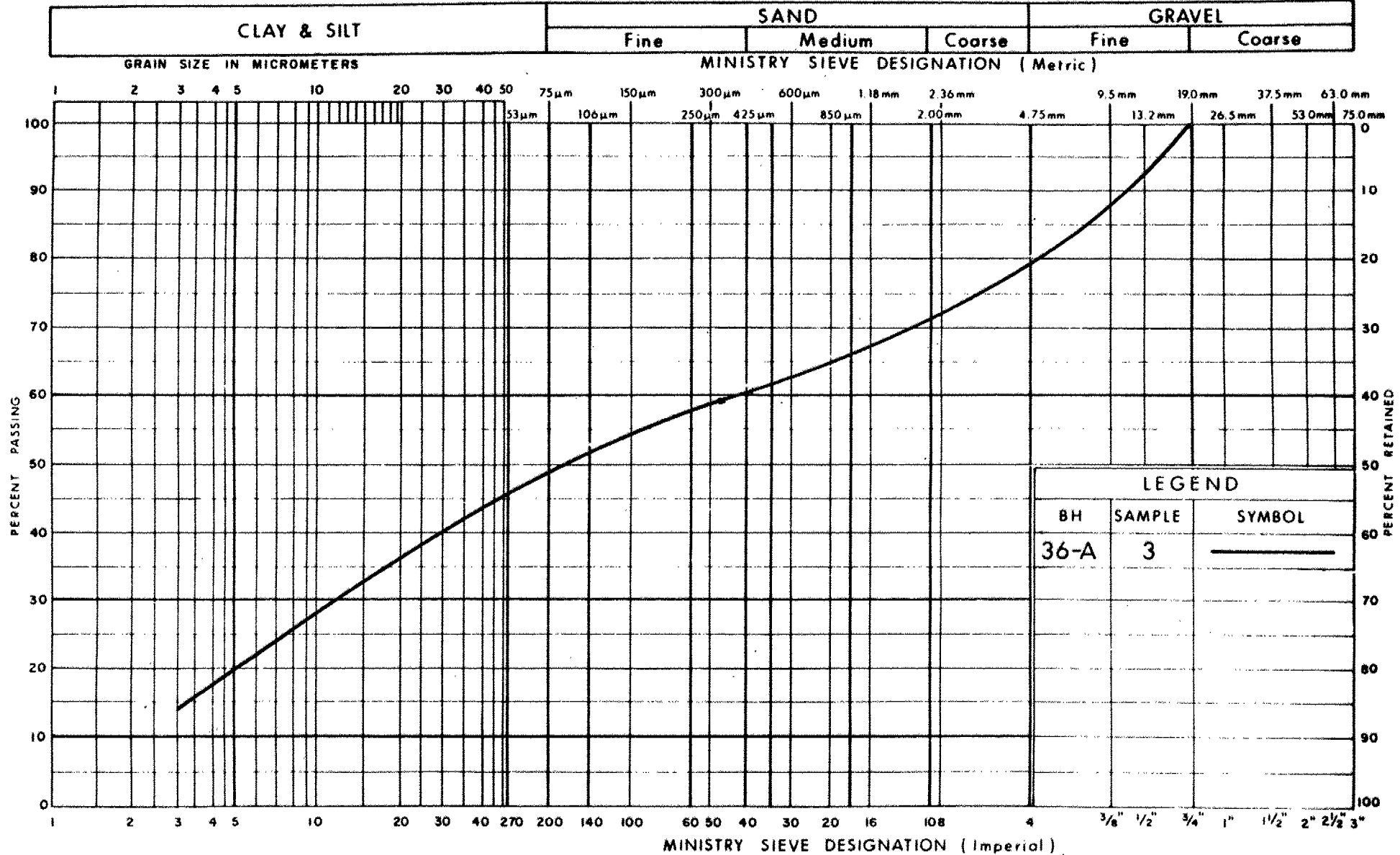
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GRAIN SIZE DISTRIBUTION
HETEROGENEOUS MIXTURE OF SANDY SILT
SOME GRAVEL & CLAY (GLACIAL TILL)

FIG No 2

W P 199-77-11

UNIFIED SOIL CLASSIFICATION SYSTEM



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GRAIN SIZE DISTRIBUTION
HETEROGENEOUS MIXTURE OF CLAYEY SILT
SOME SAND & GRAVEL (GLACIAL TILL)

FIG No 3

W P 199-77-11

RECORD OF BOREHOLE No 36-A

1 OF 1







METRIC

W.P. 199-77-11 LOCATION Co-ords: N 4 799 828; E 277 885 ORIGINATED BY JB
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger / NQ Coring COMPILED BY JB
 DATUM Geodetic DATE September 17, 1991 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							WATER CONTENT (%) 10 20 30
117.6	Ground Surface							20 40 60 80 100							
0.0	150mm Clayey Silt Fill		1	SS	34										
116.2	Clayey Silt, Some Sand (Possible Fill) Brown		2	SS	17										
1.4	Brownish Grey to Gray		3	SS	18										
	Heterogeneous Mixture of Clayey Silt Some Sand and Gravel (Glacial Till)		4	SS	26										
			5	SS	23										
			6	SS	19										
			7	SS	24										
			8	SS	21										
			9	SS	68										
			10	SS	71/23cm										
			11	SS	77/19cm										
			12	SS	30/10cm										
			13	RC	REC 57%										
	14		RC	REC 66%											
107.5			15	RC	REC 82%										
10.1	Weathered Sound														
	Bedrock Shale														
	Containing Siltstone Interbeds														
102.6															
15.0	End of Borehole														
	1991 09 24 • GROUND WATER CONDITIONS														
	PIEZO. NO.														
	GROUND WATER ELEVATION (Metres)														
	1														
	108.8														

RECORD OF BOREHOLE No 36-B 1 OF 1 METRIC

W.P. 199-77-11 LOCATION Co-ords: N 4 799 858; E 277 891 ORIGINATED BY AH
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger / BXL Coring COMPILED BY JB
 DATUM Geodetic DATE September 18, 1991 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	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										10 20 30		
108.4	Ground Surface																			
107.8	0.6m Dark Brown Fill		1	SS	45	*	108													
0.6	Heterogeneous Mixture of Sandy Silt, Some Gravel and Clay Grayish Red		2	SS	60/15 cm											20 27 40 13				
106.3	Glacial Till, Very Dense		3	SS	60/15 cm															
2.1	Weathered Sound		4	SS	60/30 cm		106													
	Bedrock Shale		5	RC	REC 77%											RQD 21%				
	Containing Siltstone Interbeds		6	RC	REC 99%		104									RQD 60%				
103.0																				
3.4	End of Borehole • Groundwater Elevation not Established.																			

RECORD OF BOREHOLE No 36-C

1 OF 1

METRIC

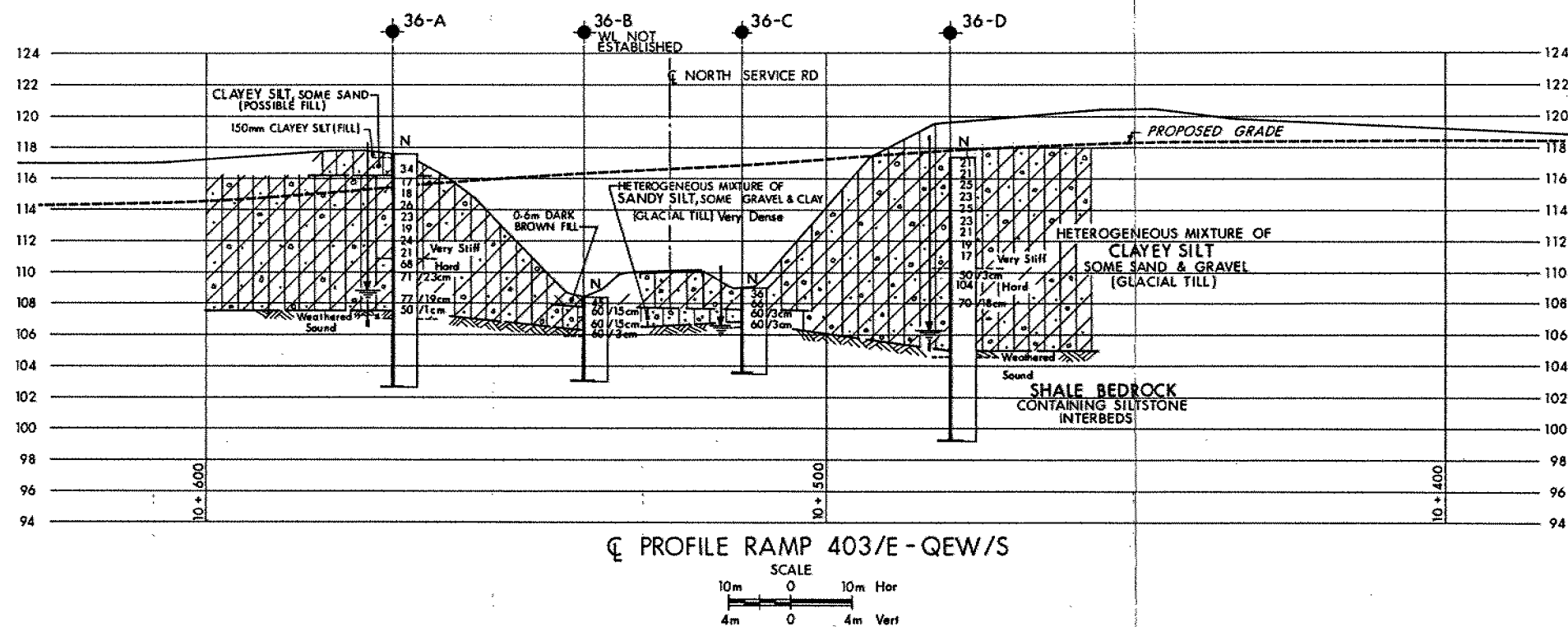
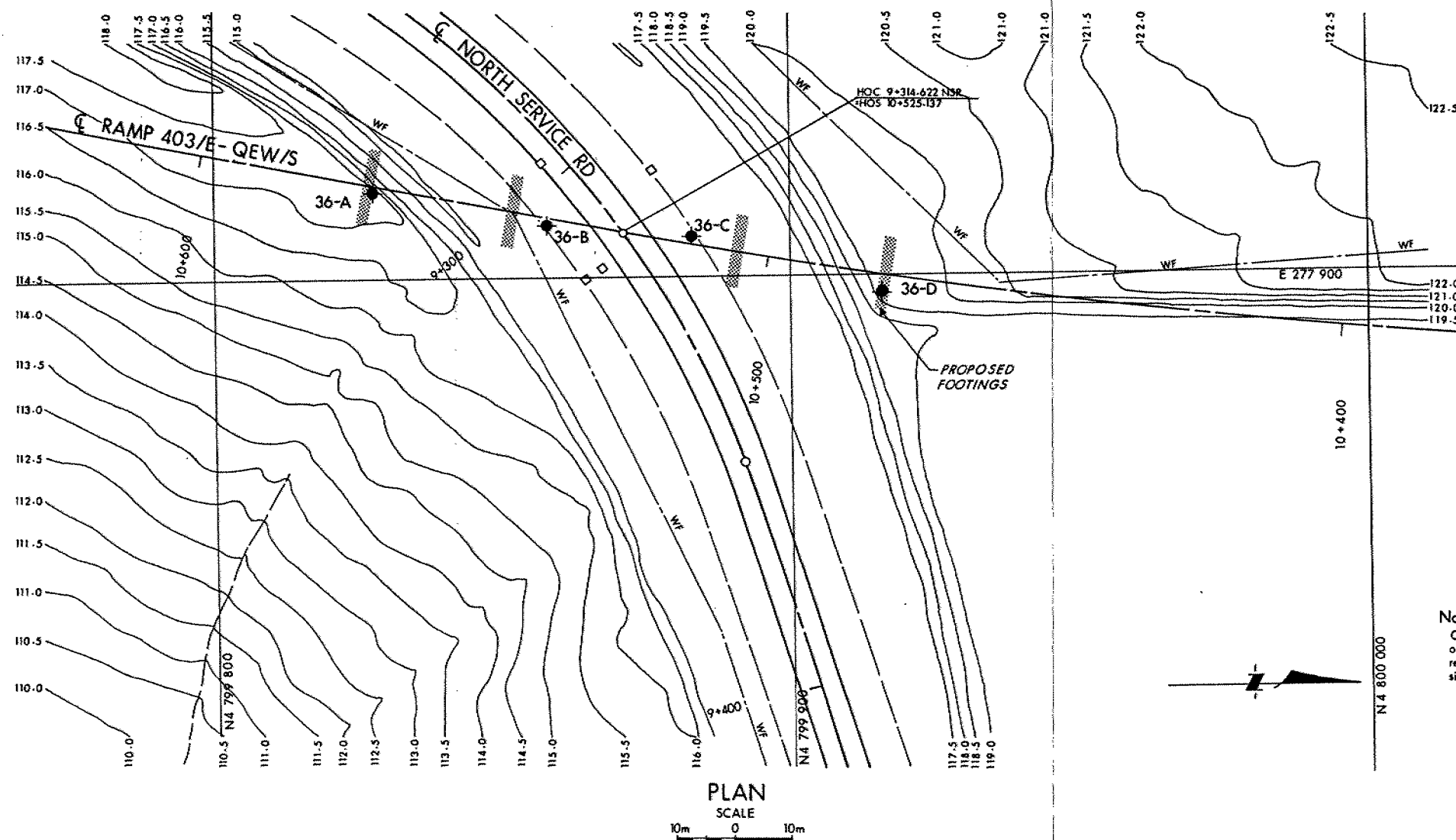
W.P. 199-77-11 LOCATION Co-ords: N 4 799 883; E 277 893 ORIGINATED BY AH
DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger / BXL Coring COMPILED BY JB
DATUM Geodetic DATE September 19, 1991 CHECKED BY BI

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
109.0	Ground Surface												
0.0	Heterogeneous Mixture of Clayey Silt		1	SS	36								
107.6	Some Sand and Gravel (Glacial Till) Hard Brownish Grey		2	SS	66								
1.4	Heterogeneous Mixture of Sandy Silt, Some Gravel and Clay **		3	SS	60/30m								
106.9	Weathered Sound		4	SS	60/30m								
2.1	Bedrock Shale		5	RC	REC 92%								RQD 61%
	Containing Siltstone Interbeds		6	RC	REC 86%								RQD 33%
103.5			7	RC	REC 100%								RQD 80%
5.5	End of Borehole												
	1991 09 24												
	* GROUND WATER CONDITIONS												
	PIEZO. NO.												
	GROUND WATER ELEVATION (Metres)												
	1												
	** Very Dense (Glacial Till)												

RECORD OF BOREHOLE No 36-D 1 OF 1 METRIC

W.P. 199-77-11 LOCATION Co-ords: N 4 799 918; E 277 903 ORIGINATED BY JB/AH
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger / NQ Coring COMPILED BY JB
 DATUM Geodetic DATE September 19 to 23, 1991 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 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							10 20 30		
117.4	Ground Surface																
0.0	Heterogeneous Mixture of Clayey Silt Some Sand and Gravel (Glacial Till)		1	SS	21										8 24 37 31		
			2	SS	21												
			3	SS	25												
			4	SS	23												
			5	SS	25												
			6	SS	23												
			7	SS	21												
			8	SS	19												
			9	SS	17												
			10	SS	50 / 5 cm												
			11	SS	104												
			12	SS	70												
105.0	Brownish Grey		13	RC	REC 15%									RQD 0%			
12.4	Weathered Sand		14	RC	REC 7%									RQD 0%			
	Greyish Red		15	RC	REC 53%									RQD 28%			
	Bedrock Shale		16	RC	REC 0%										RQD 0%		
			17	RC	REC 93%											RQD 14%	
			18	RC	REC 71%											RQD 13%	
			19	RC	REC 95%											RQD 49%	
99.2			Containing Siltstone Interbeds														
18.2	End of Borehole																
1991 09 24 * GROUND WATER CONDITIONS																	
PIEZO. NO.		GROUND WATER ELEVATION (Metres)															
1		106.2															
* The water level, which was measured in the piezometer, may not have stabilized, due to the use of water during the coring operation.																	



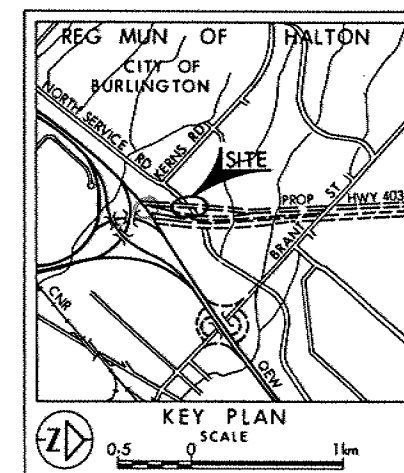
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 199-77-11

NORTH SERVICE RD
HWY 403/QEW INTERCHANGE
(BRIDGE-36)
BORE HOLE LOCATIONS & SOIL STRATA



SHEET



Note:
Contour Lines on the North side
of the proposed bridge do not
represent current conditions at
site due to excavation for Hwy 403

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 91 09
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
36-A	117.6	4 799 828.0	277 885.0
36-B	108.4	4 799 858.0	277 891.0
36-C	109.0	4 799 883.0	277 893.0
36-D	117.4	4 799 916.0	277 903.0

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 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION
1			
Geocres No 30M5-187			
HWY No 403		DIST 4	
SUBWD JB	CHECKED	DATE 911216	SITE 10-482
DRAWN DT	CHECKED	APPROVED	DWG 1997711-A

MINUTES OF MEETING

Date : Tuesday, September 29, 1992

Time : 10:00 a.m.

Location : Foundation Section

Present : Foundation - M.Devata, B.Iyer
Structural Office - G.Al-Bazi, E.Chan

Item

Action By

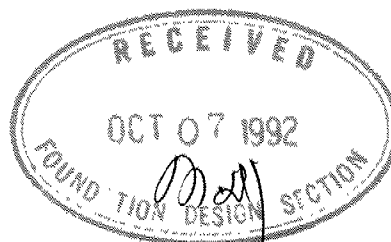
1. Review of Bridge #36 Preliminary G.A.

- a. Foundation agreed both caisson and spread footing were equally feasible for pier foundations. Spread footings can be constructed by either open cut or roadway protection.
- b. The cost comparison of both schemes prepared by the Structural Office are reviewed. Foundation confirmed the prices of caissons and roadway protection assumed were reasonable. The cost of the caisson and footing scheme are \$ 39,000 and \$ 83,000 resp. In view of cheaper cost and minimal disturbance to existing slope, Foundation recommended to use caissons. GAB/EC
- c. Foundation confirmed that a single 1.8 m diameter caisson embedded 3 m into sound shale would be able to resist a lateral load of 1800 kN. GAB/EC
- d. Subsequent discussion between B.Iyer and G.Al-Bazi on Oct. 2, 1992 agreed that caisson design should be based on a factored bearing capacity at ULS of 3500 kPa and a factored frictional resistance of 525 kPa at ULS between rock and concrete. Servicability will not govern.

E. Chan

cc : all attendees

Eric Chan
Project Engineer



B.I. → for files

MINUTES OF MEETING

Date : Tuesday, September 29, 1992

Time : 10:00 a.m.

Location : Foundation Section

Present : Foundation - M.Devata, B.Iyer
Structural Office - G.Al-Bazi, E.Chan

Item

Action By

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E. Chan

cc : all attendees

Eric Chan
Project Engineer



MINUTES OF MEETING

RE: Foundation Design Recommendations
Caisson Loading

DATE: January 25/94

ATTENDING:

George Al-Bazi
Nicholas Chatzitheodorou
Eric Chan

Structural Office, Design Section

Paul Payer
Tae Kim
Dave Dundas

Foundation Design Section

The purpose of the meeting was to discuss caisson loadings in general and specific caisson recommendations for the following projects:

16 Mile Creek
WP 406-85-01/02, Site 10-490
Hwy 403, District 4, Burlington

Bridge #36
North Service Road - Hwy 403/QEW Interchange
WP 199-77-11, Site 10-482
Hwy 403, District 4, Burlington

Bronte Creek
WP 410-85-01/02, Site 10-220
Hwy 403, District 4, Burlington

It was agreed:

- All of the above sites are in Queenston Shale
- Caisson capacities may be calculated assuming

end-bearing

Factored Bearing Capacity at ULS = 3500 kPa
Bearing Capacity at SLS Type II = 3500 kPa

bond stress

Factored Bond Stress at ULS = 350 kPa
Bond Stress at SLS Type II = 350 kPa

- Capacity from bond stress shall not exceed capacity from end bearing
- Bond stress in weathered bedrock zone shall be excluded
- 16 Mile Creek will be designed using above-noted recommendations:
- Bridge #36 will be revised using above-noted recommendations
- Structural Office will advise design consultant for Bronte Creek to verify design is in accordance with above-noted recommendations

prepared by D. Dundas

A handwritten signature in cursive script, appearing to read "D. Dundas", written in dark ink.

If there are any errors or omissions, please advise.

1992 10 02

G. Al-Bazri

Bridge 36

Gave him the following design parameters for design.

End bearing = 3500 kPa
(factored ULS)

Shaft friction = 525 kPa.
(concrete & bedrock)

memorandum



To: Mr. V.F. Boehnke
Head, Structural Section
Central Region

Date: 1991 10 06

Attn: Mr. M.D. Bendayan

From: Foundation Design Section
Room 315, Central Building

Re: Foundation Investigation
W. P. 199-77-09/10/11
Highway 403, Sites 10-482/10-483/10-484
North Service Road and Kerns Road, Burlington, Ontario
District 4, Burlington

INTRODUCTION

A foundation investigation was carried out at the above-captioned site, between 1991 09 17 and 1991 09 23, 1991 for three bridges to be constructed across the existing, recently-constructed North Service Road, immediately to the west of the Highway 403/Brant Street interchange.

During this investigation, a total of 15 boreholes were advanced to depths of 2.6 to 16.2 m.

This memorandum contains the factual information obtained from the fieldwork, and provides interim recommendations for the three proposed bridges. Final reports for each bridge will follow shortly.

SITE DESCRIPTION

The existing North Service Road, is located to the north of the QEW within a cut, at an elevation approximately 5 m lower than the prevailing ground surface. South of the cut, the prevailing ground slopes rapidly to the south towards the QEW and Lake Ontario.

It is proposed to construct Bridge #'s 31, 32 and 36 (Site No.'s 10-483, 10-484, 10-482) across the cut carrying the newly-constructed North Service Road. The three bridges are to be located immediately to the west of Bridge #40 (Contract No. 91-22), which is presently under construction.

PROCEDURES

The fieldwork, for this project, was carried out by this office between 1991 09 17 and 1991 09 23. A total of 15 boreholes, were advanced to depths of 2.6 to 12.2 m using continuous-flight hollow stem augers driven by a truck-mounted drilling rig equipped with standard soil sampling equipment.

Most of the boreholes were then extended to depths of up to 16.2 m using conventional diamond drilling (BXL and NQ) techniques to prove bedrock and, in some cases, to penetrate boulders and/or pieces of rafted bedrock.

Groundwater levels were measured in most of the open boreholes immediately upon completion of sampling (or prior to coring). Upon completion of coring, piezometers were then installed in several of the boreholes, in order to measure the long term groundwater conditions.

The locations of the boreholes were staked out in the field, and their elevations determined by McCormick Rankin Consulting Engineers. However, due to difficulties in having the drilling rig gain access to most of these locations (ie. the boreholes were located on a slope), nearly all of the boreholes had to be moved. Slight changes in location and elevation of the boreholes were determined by representatives from this office by simple methods (ie tape measure, compass and hand level).

The soil samples, which were obtained in the field, were examined in the laboratory by visual and tactile methods. Moisture content, Atterberg Limits and grain size distribution tests were conducted on several select soil samples. The results of this laboratory testing will be included in the final report for each site.

SUBSURFACE CONDITIONS

The stratigraphy at the site generally consists of a cohesive, heterogeneous mixture of clayey silt with some sand and gravel (glacial till) which is, in turn, underlain by a non-cohesive, heterogeneous mixture of sandy silt with some gravel and clay (glacial till). The cohesionless till directly overlies the bedrock surface. A thin surficial layer of fill was also encountered at some locations above cut on the south side of North Service Road and, at several places, within the ditches directly adjacent to the road.

All boreholes reached bedrock (or the assumed bedrock surface) at depths ranging from 2.0 to 12.4 m, or elevations of 105 to 107.5 m. The bedrock surface was found to generally slope from north to south.

Although, the water table was found to range from elevation 106.2 to 109.4 m, it is likely that some of the higher water levels recorded in the boreholes may not have represented the stabilized groundwater table, since water was used during coring operations. It is believed that the groundwater table varies from about elevation 106.2 to 108.2 m and generally slopes from north to south towards the QEW and Lake Ontario.

Detailed descriptions of both the soil and groundwater conditions which were encountered at the various borehole locations will be given in the final report for each site.

DISCUSSIONS and RECOMMENDATIONS

General

The existing North Service Road consists of a single lane road running in a generally east/west direction throughout the area of investigation.

Initially, Bridges 31 and 32 will be used to accomodate two Highway 403 Westbound and Eastbound Lanes, respectively. Ultimately, however, they will both be widened to accomodate an additional lane.

Bridge 36 will be used to accomodate two lanes for the ramp from the Highway 403 Eastbound lane to the QEW Southbound lane. However, we understand that, in this case, there will not be a need to widen this bridge.

All three bridges will consist of three-span structures, with the inner span being supported by piers.

It is proposed that, on the south side of Bridges 31 and 32, the grade will be raised by up to 5.0 and 7.5 m, respectively, within 50 m of their respective south abutments. However, the grade on the north side of both bridges will be slightly lowered.

At Bridge 36, the existing grade will be lowered by up to 2.0 m within 50 m of the abutments on both sides of the bridge.

Design Considerations

Abutments

The loads at the abutment areas, for the three bridges, may be supported by spread footings placed either on undisturbed clayey silt till or on granular fill. The recommendations for the north and south abutments for the three bridges will be dealt with below.

South Abutments

The loads from the south abutment areas may be supported by conventional shallow spread footing foundations. The foundations must be taken below any fill, organic or otherwise unsuitable soil to bear on the undisturbed, very stiff, glacial till. For such footings, a design value of 600 kPa may be used for the factored bearing capacity at U.L.S. or 300 kPa for the bearing capacity at S.L.S. Type II at the following elevations:

<u>Structure</u>	<u>South Abutment</u> <u>(m)</u>
Bridge 31	114.5
Bridge 32	114.4
Bridge 36	116.2

Where subexcavation is required, the excavated soil should be replaced by well-compacted Granular 'A'.

North Abutments

Bridge 31 and 32

The clayey silt till encountered beneath the north abutment areas for both Bridges 31 and 32 appears to be somewhat weaker than similar material encountered beneath either the south abutment areas of those bridges or both abutment areas of Bridge 36. In addition, it should be noted, that a particularly weak zone was found at a depth of about 4 m (ie. an elevation of about 113.2 m).

Conventional shallow spread footing foundations placed directly on the stiff to very stiff clayey silt till may be considered. However, in view of the soil conditions, encountered, a design value of only 400 kPa U.L.S. and 200 kPa S.L.S. Type II may be used for footings placed below any organic or otherwise unsuitable soils at an elevation of 116.6 m. In addition, in order to prevent overstressing of the weak zone referred to above, the footings should not be extended any lower than about 0.6 times the footing width of the footing (if the footing is square) above the weak zone or elevation 115.0 m (assuming a three m wide square footing).

Should additional allowable bearing pressures be required, it is recommended that the loads from the abutment areas of these two bridges be supported by conventional shallow spread footing foundations placed on a well-compacted pad of Granular 'A'. If this scheme is adopted, footings may be designed for factored bearing pressures of 850 kPa at U.L.S. and 300 kPa at S.L.S. Type

II. Based on a 3 m wide square footing, calculations indicate that the granular pad beneath the underside of the footings must be at least 0.4 times the footing width or a minimum of 1.2 m thick.

Once again, in order to prevent overstressing of the weak zone referred to above, the footings must be kept as high as possible and should not extend below elevation 116.8 m (assuming a 3 m wide square footing). However, should the geometry require that the footings must be extended below this elevation, then either the allowable bearing pressure must be reduced or, alternatively, pile foundations may have to be considered.

Bridge 36

The soils encountered at the north abutment area of Bridge 36 are somewhat more competent than those encountered for the two previous bridges. Therefore, in this case, it is recommended that the loads from the north abutment be supported by conventional shallow spread footing foundations. The foundations must be taken below any organic-stained or otherwise unsuitable soil to bear on undisturbed clayey silt till. For footings, placed at or below an elevation of 116.8 m, a design value of 600 kPa may be used for the factored bearing capacity at U.L.S. or 300 kPa for the bearing capacity at S.L.S. Type II.

It should be noted, however, that the soils appear to weaken somewhat below a depth of 5.2 m (elevation of 112.2). Therefore, in order to prevent overstressing this underlying zone (which would result in a reduction in the above-stated bearing capacities) the footings should not be placed lower than an elevation about 113.3 m (assuming a three m wide square footing).

Piers

From the drawings provided to us, it appears that the proposed piers for Bridges 31 and 32 will be located close to the bottom of the existing cut (ie. immediately adjacent to the North Service Road). In this case, spread footings should be placed on either the very hard clayey silt till or the very dense sandy silt till.

For Bridge 36, however, it appears that the proposed piers (and particularly the one to the south) will be located somewhat further up the existing slope. Spread footings may also be considered here. However, excavations to reach a suitably competent till will be somewhat greater and the excavated slopes may have to be cut back significantly. In this case, caissons socketed into the underlying bedrock may also be considered.

Recommendations for spread footings are given for the proposed piers at all three bridges and caissons for the piers at Bridge 36.

Spread Footings on Glacial Till

The piers for all of the bridges may be founded on shallow spread footings placed on either very hard clayey silt till or very dense sandy silt till. For such footings, a design value of 750 kPa may be used for the factored bearing capacity at U.L.S. This value may be assumed at or below the following elevations:

<u>Structure</u>	<u>North Pier</u> <u>(m)</u>	<u>South Pier</u> <u>(m)</u>
Bridge 31	108.1	108.3
Bridge 32	108.6	108.3
Bridge 36	109.8	110.1

The bearing capacity at S.L.S. Type II will not govern the design, in this case.

Placement of the footings on the underlying shale was not considered since the footing excavations would extend well below the elevation of the existing roadway and the groundwater table.

Piers Placed on Caissons

The structural loadings for the piers at Bridge 36, may be transferred to the underlying sound shale bedrock by means of bored cast-in-place concrete caissons. All caissons should be socketed at least 0.3 m into the underlying sound competent bedrock.

For the north and south piers, it is recommended that the following design parameters be used for caissons founded on sound shale bedrock at or below elevations of 106.0 and 105.5 m, respectively:

	Caisson Diameter	
	760 mm	915 mm
Factored Axial Capacity at U.L.S.	4400 kN	6300 kN

In both cases, the allowable capacities at S.L.S. Type II would not govern the design.

Caissons should be a minimum diameter of 760 mm to allow for both the clean out of any basal debris and final evaluation of the rock surface in order to confirm the above-stated capacities.

The caissons must be fully cased and socketed at least 0.3 m into the underlying sound bedrock. However, depending upon the desired degree of lateral resistance, the length of the socket may have to be increased.

Some groundwater infiltration should be expected, particularly when augering through the sandy silt till, below the groundwater table. It may be necessary to control groundwater by using drilling mud or other methods.

Resistance to Lateral Forces

For design purposes, an unfactored coefficient of friction of 0.45 may be assumed to apply between the base of the footing and the hard clayey silt till or very dense sandy silt till at the pier locations. At the abutments, however, the unfactored coefficient of friction should be reduced to 0.35 between the base of the footing and the very stiff clayey silt till.

Approach Areas

Slopes for the approaches and abutment areas may be designed at 2H:1V assuming they are comprised of borrow materials as per MTO specifications.

Frost Protection

All foundations should have a minimum cover of 1.2 m for frost protection.

Construction Considerations

Excavations and Dewatering

Temporary excavations through the clayey silt till to depths of up to 4.0 m will be temporarily stable at slopes of 1:1.

It is expected that any surface water entering the excavation or perched water in any sandy zones within the cohesive (ie. clayey silt till) may be controlled by gravity drainage and/or properly-filtered sumps. If, however, the excavations must extend through the coarser underlying till (ie. 8s sandy silt till) below the

groundwater table, more extensive groundwater control measures may be required.

Construction of Approaches and Abutment Areas

All of the existing organic-stained fill or other unsuitable soils must be stripped throughout the full-width of the proposed abutment and approach areas.

The subgrade preparation, the selection of the fill material and its placement and compaction should be carried out according to OPSS Standards and MTO practice.

MISCELLANEOUS

The field investigation was supervised by Mr. Arthur Hildebrand and Mr. John Blair using equipment owned and operated by Master Soil Investigation Inc.

This memorandum was written by Mr. J. Blair, Project Foundation Engineer, reviewed by Messrs. B. Iyer, Senior Foundation Engineer and M. Devata, Chief Foundation Engineer.

We believe that this memorandum meets with your present requirements. However, should you have any questions regarding it, please do not hesitate to contact this office.

John A. Blair

John A. Blair, P.Eng.
Project Foundation Engineer

for

Balu Iyer, P.Eng.
Senior Foundation Engineer

WATER LEVELS (m ground surface)

SEPT 24 / 91 09:00

APPROXIMATELY 25 hrs after last rain

	<u>BH</u>	<u>WATER LEVEL (m)</u>	<u>BOTTOM OF PIEZOMETR (m)</u>
P	32-B	$10.50 - 1.92 = 8.58$	$11.39 - 1.92 = 9.47$
O	31-B	8.95	—
P	36-A	$10.75 - 1.92 = 8.83$	$13.00 - 1.92 = 11.08$
O	31-A	DESTROYED & FILLED	
O	32-A		
O	31-D	0.92	—
P	32-D	$4.52 - 1.92 = 2.60$	$4.62 - 1.92 = 2.70$
P	36-C	$\sim 4.33 - 1.92 = 2.41$ (MUD)	$4.33 - 1.92 = 2.41$
P	31-G	$3.04 - 1.92 = 1.12$	$4.72 - 1.92 = 2.80$
O	36-B	CAVED	
O	(32-G, 31-C)	"	
O	32-C	$4.52 - 1.92 = 2.60$	—

SEPT 26 / 91 10:00

P	36-D	$13.08 - 1.92 = 11.16$	$14.35 - 1.92 = 12.43$
P	32-E	$9.90 - 1.92 = 7.98$	$11.95 - 1.92 = 10.03$
P	32-D	MOIST AT BOTTOM (MUD)	
P	36-C	"	
P	31-G	$2.95 - 1.92 = 1.03$	
O	32-B	$10.30 - 1.92 = 8.38$	
	36-A	$10.75 - 1.92 = 8.83$	

P- PIEZOMETER

O- OPEN HOLE

Structural Section
Central Region
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario, M3M 1J8
Telephone: 235-5508

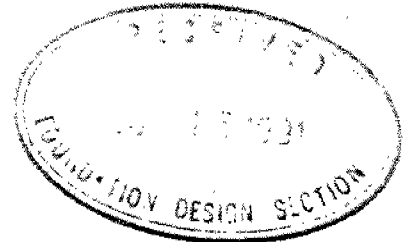
MINISTRY OF TRANSPORTATION

memorandum

TO: Mr. M. Devata
Chief Foundation Engineer
Foundation Design Section
3rd Floor, Central Building

DATE: June 13, 1991

Attn: Mr. Paul Payer,
Sr. Foundation Engineer



RE: G.W.P. 199-77-06
Hwy 403 from the Freeman Interchange to Hwy 5
District 4, Burlington

As shown on the attached Key Plan three new structures (Bridges #31, #32 and #36) will be constructed across the existing North Service Road as part of the above project (the North Service Road has just been constructed and open to traffic under control 89-26). You are kindly requested to prepare Foundation Reports for each of the proposed bridges as described below:

1. Hwy 403 E.B.L. over the North Service Road (Bridge #32)
W.P. 199-77-10, Site 10-484

This new bridge, as shown schematically on the attached two prints of Bridge Site Drawing E-83-403-16 will have three spans (16±m, 24±m, 16±m). The deck width needs initially to accommodate two Hwy 403 E.B. lanes. Ultimately, it will be widened towards the Hwy 403 Centreline to suit a three lane cross-section for the Hwy 403 Eastbound lanes. Both deck layouts are depicted on the drawing. The scope of your work should cover both the initial and ultimate lengths of the abutments and piers foundations required for both schemes as illustrated in red and blue, respectively, on Drawing E-83-403-16. The superstructure will be constructed over existing traffic. The superstructure could consist of either a cast-in-place post-tensioned deck or prestressed concrete girders with a concrete slab. False-work openings will be provided at the time of construction, as required. The profiles of Hwy 403 E.B.L. and North Service Road have been depicted on the bridge site drawing along with the location on plan and elevation of the existing 400 mm PVC sanitary sewer.

2. Hwy 403 W.B.L. over the North Service Road (Bridge #31)
W.P. 199-77-09, Site 10-483

Subject bridge, as shown on the attached two prints of Bridge Site Drawing E-83-403-15, will have three spans similar to Bridge #32 (16±m, 24±m, 16±m). This is to line up the columns of both bridges thus offering a better appearance to the motorists driving along the North Service Road while at the same time satisfying the required stopping sight distances. The deck will also be constructed initially to accommodate two Hwy 403 W.B. lanes. In the future, when traffic volumes warrant it, the deck will be widened towards the median to a three lane cross-section. Both lay-outs have been depicted on Drawing E-83-403-15.

The foundation recommendations should cover both the initial and ultimate lengths of the abutment and pier footings.

This bridge will also be constructed over existing traffic and as a result false-work openings will be built during construction of the superstructure, depending on the type of deck used, as pointed out for Bridge #32.

Profiles for both the Hwy 403 W.B. lanes and the North Service Road have been detailed on Drawing E-83-403-15 along with the location on plan and elevation of the existing 400 mm PVC sanitary sewer.

3. Ramp Hwy 403/E-QEW/S over the North Service Road (Bridge #36)
W.P. 199-77-11; Site 10-482

Subject bridge will have three spans (25±m, 38±m, 25±m) as depicted on the attached two prints of Bridge Site Drawing E-83-403-14. The pier locations are dictated by the minimum stopping light distance required by the degree of curve on the North Service Road and also by the design speed of 60 km/h.

The deck will be constructed to accommodate two 3.75m wide lanes and 2.5 m wide shoulders, which represent both the initial and future traffic demands along Ramp 403/E-QEW/S in this area. There is therefore no need to consider future widening of abutments and piers footings. All footings will be constructed perpendicular to the centreline of Ramp 403/E-QEW/S. The superstructure will consist, most likely, of a cast-in-place post-tensioned concrete deck built over traffic using false-work openings. Profiles for both Ramp 403/E-QEW/S and the existing North Service Road have been depicted on drawing E-83-403-14 along with the location in plan and elevation of the present 400 mm PVC sanitary sewer.

Since only one lane is now operational on the North Service Road it is possible that roadway protection be minimal. Please comment on this aspect of the work, bearing in mind that, for Bridges #31 and

#32, the face of the columns will be located 10.5 m away from the centreline of the North Service Road. For Bridge #36 this distance will be 11.5 m. These offsets are measured perpendicularly to the curved centreline of this roadway.

Recent photos are included for the three sites under consideration. Access to all sites is easy by driving along the North Service Road (all property is within MTO R.O.W.). Any surveys support like staking the foundation locations, supplying elevations and co-ordinates etc. should be requested to Mr. J. Elliot, P. Eng. from McCormick Rankin, Consultant Engineers (telephone 823-8500).

We also include the Foundation Reconnaissance Report listing the utilities known to exist in subject general area.

Finally, please note that nearby bridge Site 10-339 (W.P. 199-77-12) is being built under Contract 91-22 with a start construction of August, 1991. Your Foundation Report prepared for this bridge (Ramp Q.E.W/S - 403/E,W over the North Service Road) could assist you in your studies.

To comply with present scheduling your Foundation Recommendations should be available by 91-09-12 and the Foundation Reports by 91-10-11.

As usual, please feel free to call if additional information/clarification is required.



M. D. Bendayan
Sr. Structural Engineer
for:
V. F. Boehnke
Head, Structural Section

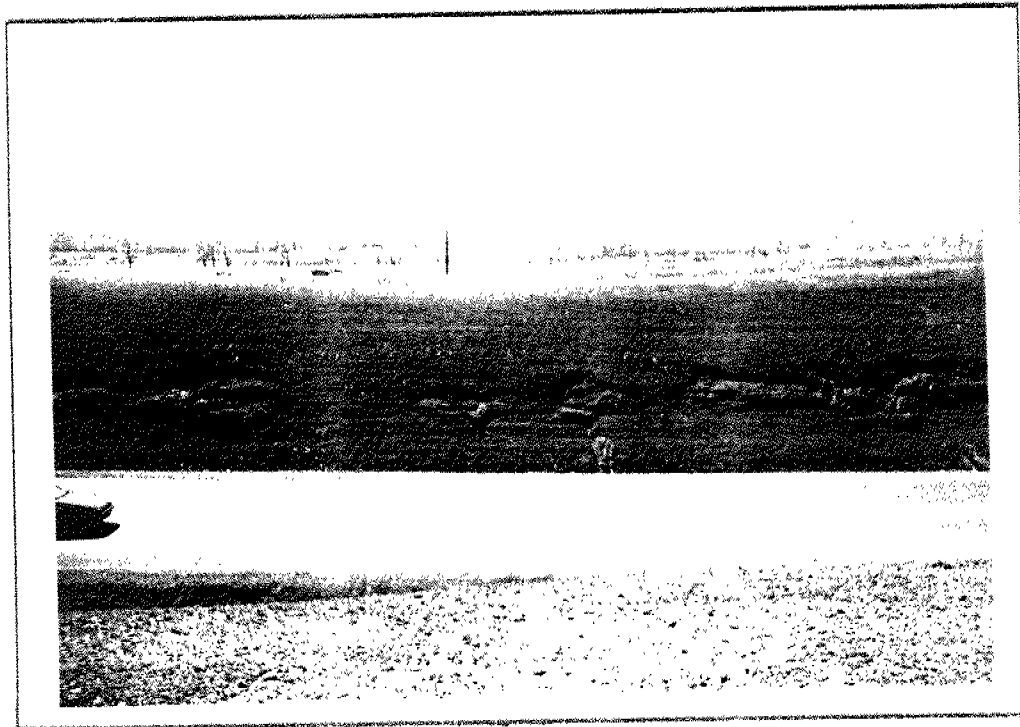
MB:dd
Encl.

cc: R. C. McCormick
I. Harrod
G. Cautillo
D. Riseboro
E. Salva

HWY. 403/E - QEW/S OVER NORTH SERVICE ROAD (BRIDGE # 36)
W.P. 199-77-10 ; SITE 10-482
DISTRICT 4, BURLINGTON



LOOKING NORTH ACROSS NORTH SERVICE ROAD

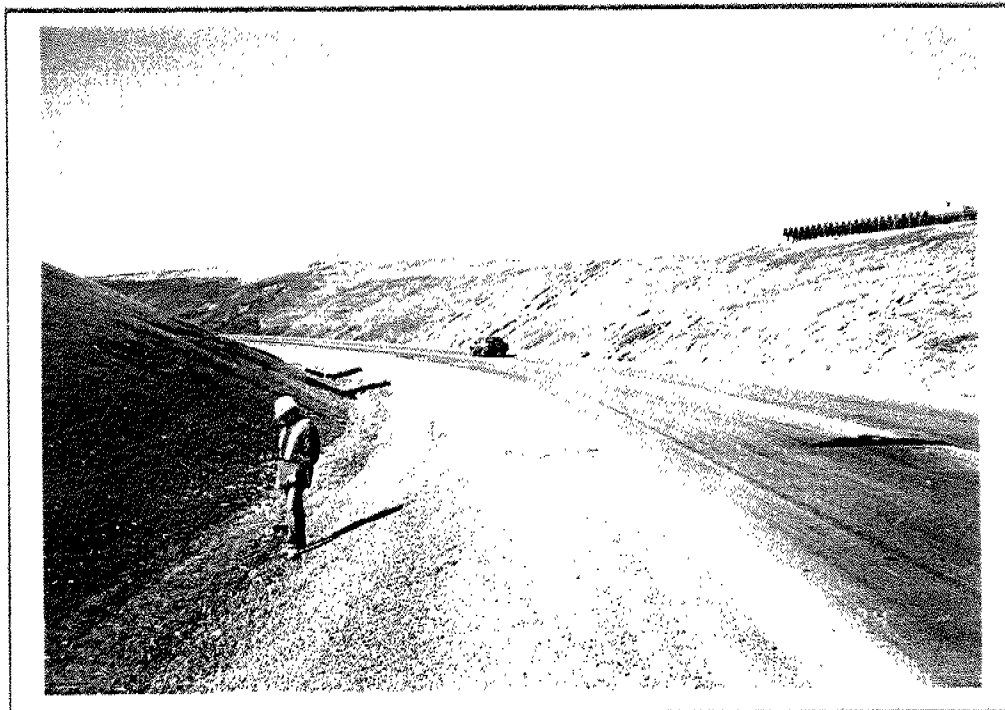


LOOKING SOUTH ACROSS NORTH SERVICE ROAD
(FREEMAN INTERCHANGE IN BACKGROUND)

BRIDGE SITE 10-482
BRIDGE # 36



LOOKING EAST ALONG NORTH SERVICE ROAD



LOOKING WEST ALONG NORTH SERVICE ROAD