

DIST. 4 REGION \_\_\_\_\_

W.P. No. 199-77-10

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. 10-484

HWY. No. 403

LOCATION Hwy 403 E.B.L  
OVER NORTH SERVICE RD

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_



Ministry  
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Transportation

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## **FOUNDATION DESIGN SECTION**

**foundation  
investigation and  
design report**

ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION

WP 199-77-10 DIST 4  
HWY 403 STR SITE 10-484

Proposed Bridge No. 32 at  
North Service Road-Hwy. 403/QEW Interchange

DISTRIBUTION

V.F. Boehnke (3)  
G. Cautillo  
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FOUNDATION INVESTIGATION REPORT  
For  
Proposed Bridge No. 32 at  
North Service Road-Hwy. 403,/QEW Interchange  
Highway 403, Site 10-484  
W.P. 199-77-10  
District 4, Burlington

INTRODUCTION

A foundation investigation was carried out at the above-mentioned site, between 91 09 17 and 91 09 25, for proposed Bridge No. 32 to be constructed across the existing, recently constructed, North Service Road, which is located immediately to the west of the Highway 403/Brant Street interchange.

During this investigation, a total of five boreholes were advanced to depths of 6.4 to 13.9 m.

This report contains the factual information obtained from these five boreholes, as well as from two relevant boreholes, which were drilled simultaneously, for a similar structure located immediately to the west (ie. Bridge No. 31 - Ref. W.P. 199-77-09).

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.

Four of boreholes were then extended to depths of up to 13.9 m, using conventional diamond drilling (BXL and NQ) techniques, in order 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 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

The subsurface conditions, described in the following sections, are based on the five (5) boreholes drilled during this investigation, as well as BH's 31-C and 31-E, which were drilled specifically for Bridge No. 31 (Ref. No. W.P. 199-77-09) but are sufficiently close to Bridge No. 32 to apply to it as well.

#### General 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 non-cohesive deposit of a 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 appears to range from an elevation of about 107.0 m to 107.2 m.

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 to brownish grey, silty clay to clayey silt was encountered at some locations to depths of up to 1.4 m above the cut on the south side of North Service Road, and within the ditches directly adjacent to the road. Much of this material has been referred to as 'Fill', since it was found to contain traces of root fibres and/or occasional topsoil enclosures.

At some locations, the soil did not appear to contain any appreciable organics. However, the lack of any well-defined structure, within the soil, indicates that this material may have been disturbed, and therefore it has been referred to as 'Possible Fill'.

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

A deposit 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, in all but one of the boreholes. This deposit was found to extend to depths of up to 10.6 m or elevations of from 106.6 to 108.7 m.

Atterberg Limits tests, carried out on two samples, from this deposit, which were obtained from the boreholes, had liquid limits of 28 percent and plasticity indices of 13 percent. These results, which have been plotted on Figure 1, indicate that this soil can be classified as CL or a clayey silt of low plasticity. Moisture contents of samples, from this deposit, ranged from 8.5 to 16 (average of 13) percent.

Two grain size distribution tests carried out on samples of soil obtained from this deposit, and shown on Figure 2, indicate a relatively well-graded material consisting of 35 to 42 percent silt, 23 to 28 percent clay, 18 to 32 percent sand and 10 to 12 percent gravel-sized particles. These results, coupled with a

visual examination of the texture of the soil, indicate that this material is likely to be of glacial origin and, therefore, may be referred to as glacial till.

Standard Penetration resistances ('N' values) of 16 to more than 130 blows/0.3 m, which were measured in this deposit, indicate that the clayey silt till is of generally very stiff to hard consistency. It should be noted, however, that a somewhat weaker zone, with an 'N' value of 7 blows/0.3 m, was encountered in BH 31-E (Ref. No. W.P. 199-77-09) at a depth of 3.7 m to 4.4 m (ie. an elevation of 112.8 m to 113.5 m).

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

At several locations, the cohesive clayey silt till referred to in the previous section, was found to be immediately underlain by a non-cohesive, heterogeneous mixture of sandy silt containing some gravel and clay, at depths ranging from 0.6 m to 8.6 m. The base of this deposit was found to extend to depths of up to 10.2 m or elevations of 106.5 to 107 m at the borehole locations.

Moisture content tests, carried out on samples of soil obtained from this deposit, ranged from 7 to 12 (average of 9.5) percent.

A grain size distribution test carried out on a sample of soil obtained from this deposit, and shown of Figure 3, indicates a generally well-graded soil with 13 percent gravel, 29 percent sand, 46 percent silt and 12 percent clay-sized particles. These results, coupled with a visual examination of the soil, indicate that this material 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, this deposit is considered to be in a very dense state.

#### Shale Bedrock

All boreholes reached bedrock (or the assumed bedrock surface), at depths ranging from 2.6 m to 10.6 m, or elevations of 105.6 m to 107.0 m. The bedrock

surface was found to generally slope to the south. In five of the boreholes, the underlying bedrock was cored to depths of 6.4 m to 16.2 m. Recoveries and RQD's ranged from 10 to 100 percent and 0 to 81 percent, respectively.

The bedrock consists of a greyish red, weak to very weak, shale containing relatively hard, interbedded, greenish grey siltstone bands. Additional details of the bedrock core samples obtained, 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 two of the piezometers, at least 24 hours after their installation, were found to be at elevations of 107.0 m to 107.2 m.

It should be noted, however, that a somewhat higher water level (elevation 109.4 m) was recorded in one of the piezometers (32-E). However, since the water level in this piezometer was measured less than 24 hours after its installation, it is likely that the water level did not have sufficient time to stabilize and did not represent the long term groundwater table.

In any case, the groundwater table will experience 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. 32 across the open cut which carries the newly-constructed, single lane North Service Road. Another bridge (No. 31 - Ref. No. W.P. 199-77-09) which is currently being investigated, will be located immediately to the west of Bridge No. 32. In addition, it should be noted that Bridge No. 40, which is presently under construction (under Contract No. 91-22), will be located immediately to the east of Bridge No. 32.

Bridge No. 32 will consist of a three-span structure, with the inner span being supported by piers. Initially, the bridge will be used to accomodate two Highway 403 eastbound lanes. Ultimately, however, it is proposed to widen the bridge, in order to accommodate one more additional lane.

On the south side of Bridge No. 32, it is proposed to raise the grade by up to 7.5 m, within 50 m of its south abutment. However, the grade on the north side of the bridge will be slightly lowered.

### Design Considerations

#### Abutments

The loads, at the abutment areas for the bridge, 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 bridge will be dealt with below.

#### South Abutment

The loads, from the south abutment area, 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 a maximum elevation of 114.4 m (assuming they are less than or equal to 3 m wide).

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

#### North Abutment

The north abutment may be supported on conventional shallow spread footing foundations. The foundations should be located at or below elevation 116.6 m, below any surficial fill or organics encountered in this area. A somewhat weaker zone was encountered at about elevation 113 m, within the generally very stiff to hard clayey silt till, in the north abutment area. To prevent overstressing the above-mentioned weaker zone, it is recommended that the foundations are located above elevation 115 m. Footings placed on native clayey silt till, as discussed above, shall be designed using a factored U.L.S. capacity of 400 kPa and an S.L.S. Type II capacity of 200 kPa (assuming that they are less than or equal to 3.0 m wide).

Alternatively, the north abutment foundation may be designed as a spread footing located on a well-compacted Granular 'A' pad. For a 3 m wide footing resting on a minimum 1.2 m thick granular pad, placed and compacted above elevation 115 m, the factored U.L.S. and S.L.S. Type II capacities will be 850 kPa and 300 kPa, respectively.

#### Piers

From the drawings provided to us, it appears that the proposed piers, for Bridge No. 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 such footings, a design value of 750 kPa may be used for the factored bearing capacity at U.L.S. (assuming that the footings are less than 3 m wide). This value may be assumed at or below the following elevations:

<u>Structure</u>	<u>North Pier</u>	<u>South Pier</u>
	<u>(m)</u>	<u>(m)</u>
Bridge No. 32	108.6	108.3

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.

#### 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 the 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 somewhat weaker clayey silt till. Finally, for footings placed on a pad of compacted Granular 'A', an unfactored coefficient of friction of 0.50 may be assumed.

#### 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 ( $\phi$ )	35°	30°
Unit Weight ( $\text{kN/m}^3$ ), $\gamma$	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

##### Excavation 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. 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.

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.

#### Concluding Remarks

As an alternative to spread footing foundations, deep foundations may be considered at the abutments. If this option is favoured, we would be pleased to provide detailed recommendations for their design and construction.

MISCELLANEOUS

The field investigation was supervised by A. Hildebrand, Engineering Trainee, M. Michalek, Project Foundation Engineer, and J. Blair, Project Foundation Engineer, 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, Senior Foundation Engineer 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 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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
$H$	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$e$	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$n$	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$w$	1, %	WATER CONTENT	$D$	mm	GRAIN DIAMETER
$\gamma_w$	kn/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	$q$	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kn/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	$v$	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	$i$	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kn/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	$j$	kn/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kn/m <sup>3</sup>	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
31-B	14	13.11-14.63	96	72	13.11-16.15	<b>SHALE</b> , greyish red, with interbedded greenish grey <b>SILTSTONE</b> (21%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	15	14.63-16.15	100	77		
31-D	6	3.35-4.88	92	47	3.35-6.40	<b>SHALE</b> , greyish red, with interbedded greenish grey <b>SILTSTONE</b> (18%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	7	4.88-6.40	100	70		
31-E	11	10.16-11.68	10	0	10.16-16.21	<b>SHALE</b> , greyish red, with interbedded greenish grey <b>SILTSTONE</b> (20%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 10.16-11.76 m); fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	12	11.68-13.21	80	27		
	13	13.21-14.68	83	37		
	14	14.68-16.21	70	24		
31-G	6	2.82-4.32	94	47	2.82-5.79	<b>SHALE</b> , greyish red, with interbedded greenish grey <b>SILTSTONE</b> (15%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat, planar to undulating, smooth.
	7	4.32-5.79	68	43		

\*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



# ROCK CORE DESCRIPTION WP 199-77-10

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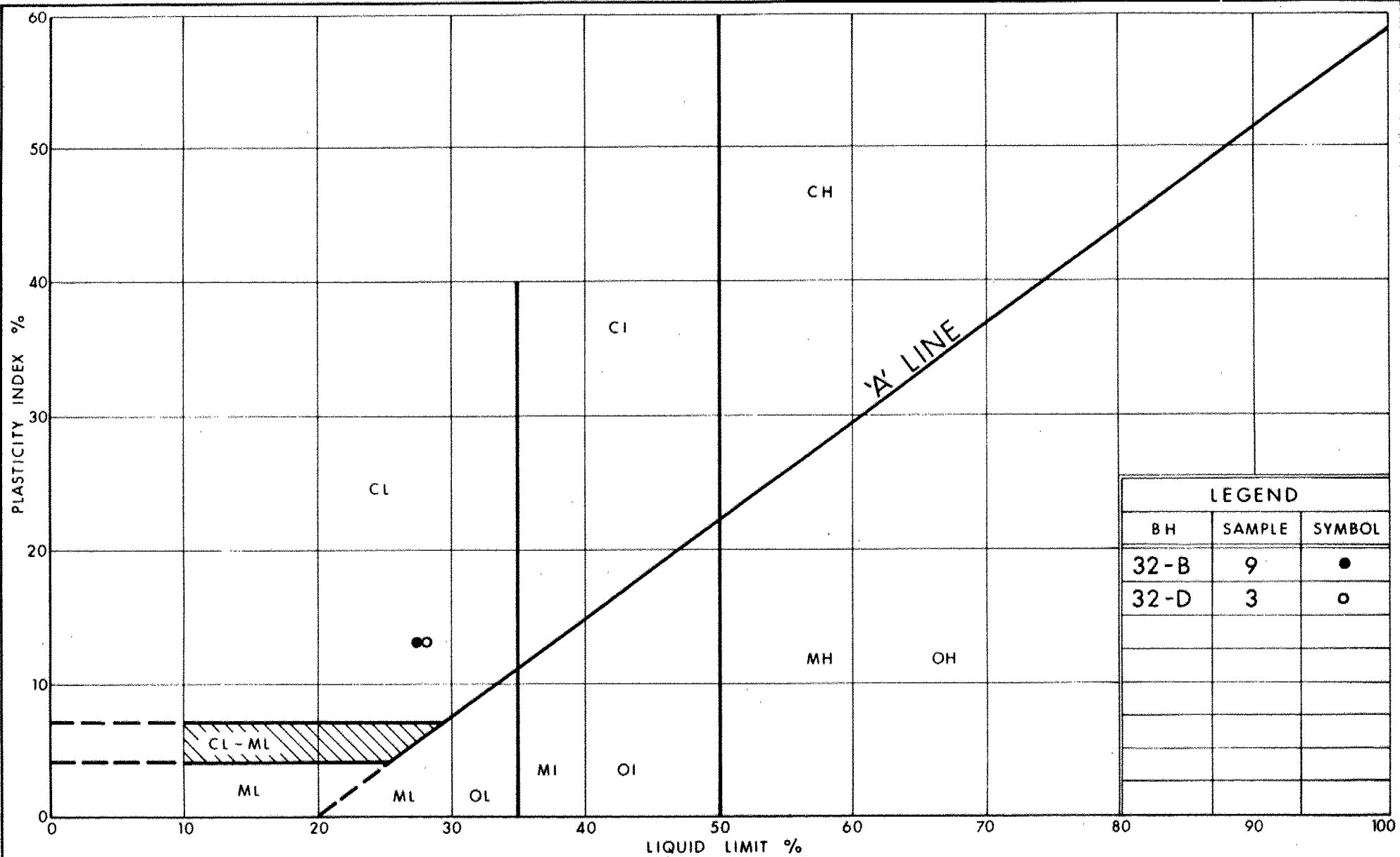
CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
32-B	12	9.91-11.43	83	67	9.91-12.95	SHALE, greyish red, with interbedded greenish grey SILTSTONE (18%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 9.91-9.98 m); fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	13	11.43-12.95	95	81		
32-C	6	3.96-5.43	57	0	3.96-6.81	SHALE, greyish red, with interbedded greenish grey SILTSTONE (13%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	7	5.43-6.81	89	0		
32-D	7	3.51-4.90	60	17	3.51-6.43	SHALE, greyish red, with interbedded greenish grey SILTSTONE (16%); very fine grained; weak to very weak; unweathered to slightly weathered (moderately weathered, 3.51-3.83 m); fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	8	4.90-6.43	100	80		
32-E	12	10.90-12.34	100	45	10.90-13.87	SHALE, greyish red, with interbedded greenish grey SILTSTONE (13%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	13	12.34-13.87	92	33		

\*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



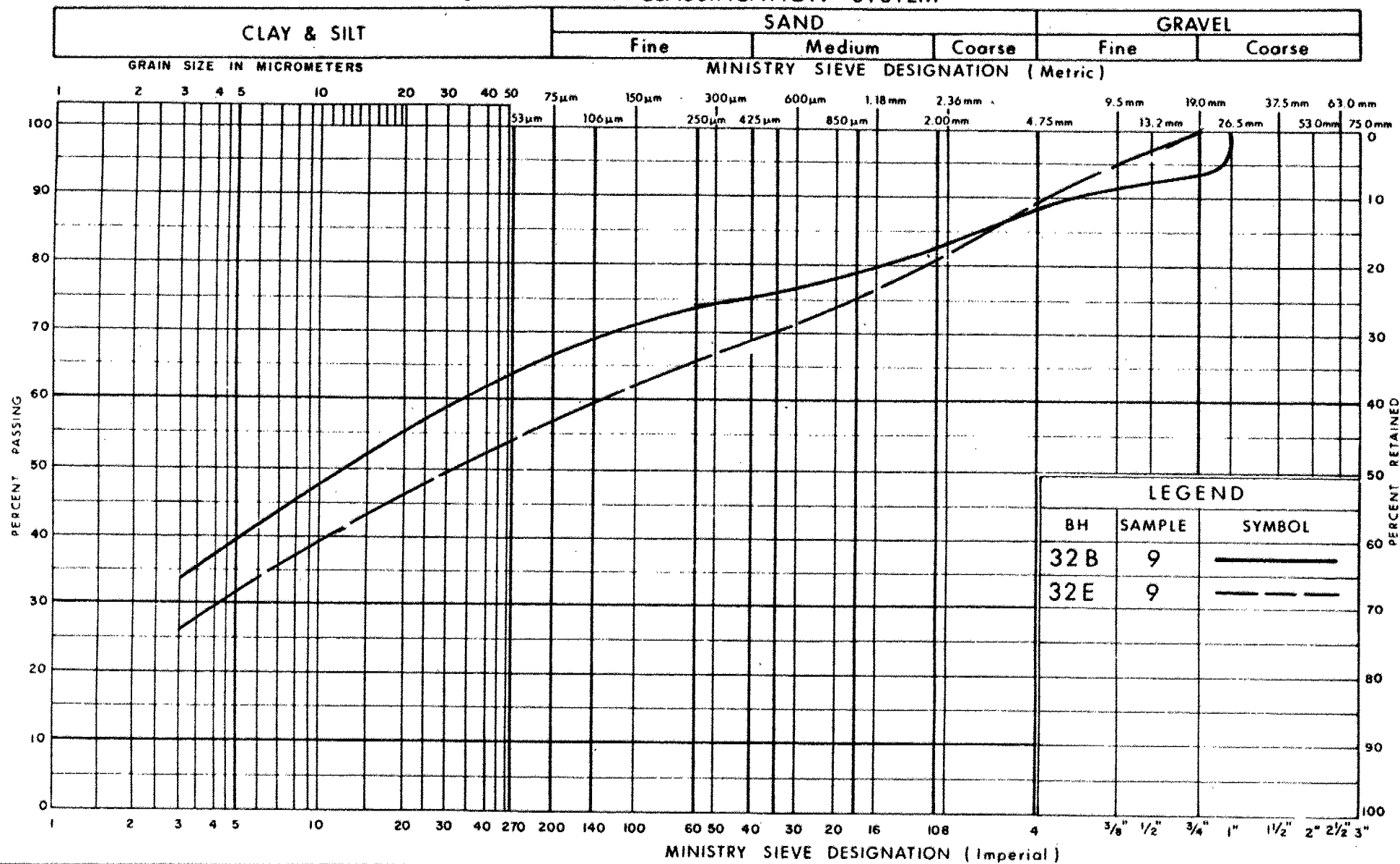
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PLASTICITY CHART  
HETEROGENEOUS MIXTURE OF CLAYEY SILT  
SOME SAND & GRAVEL (GLACIAL TILL)

FIG No 1

W P 199-77-10

## UNIFIED SOIL CLASSIFICATION SYSTEM



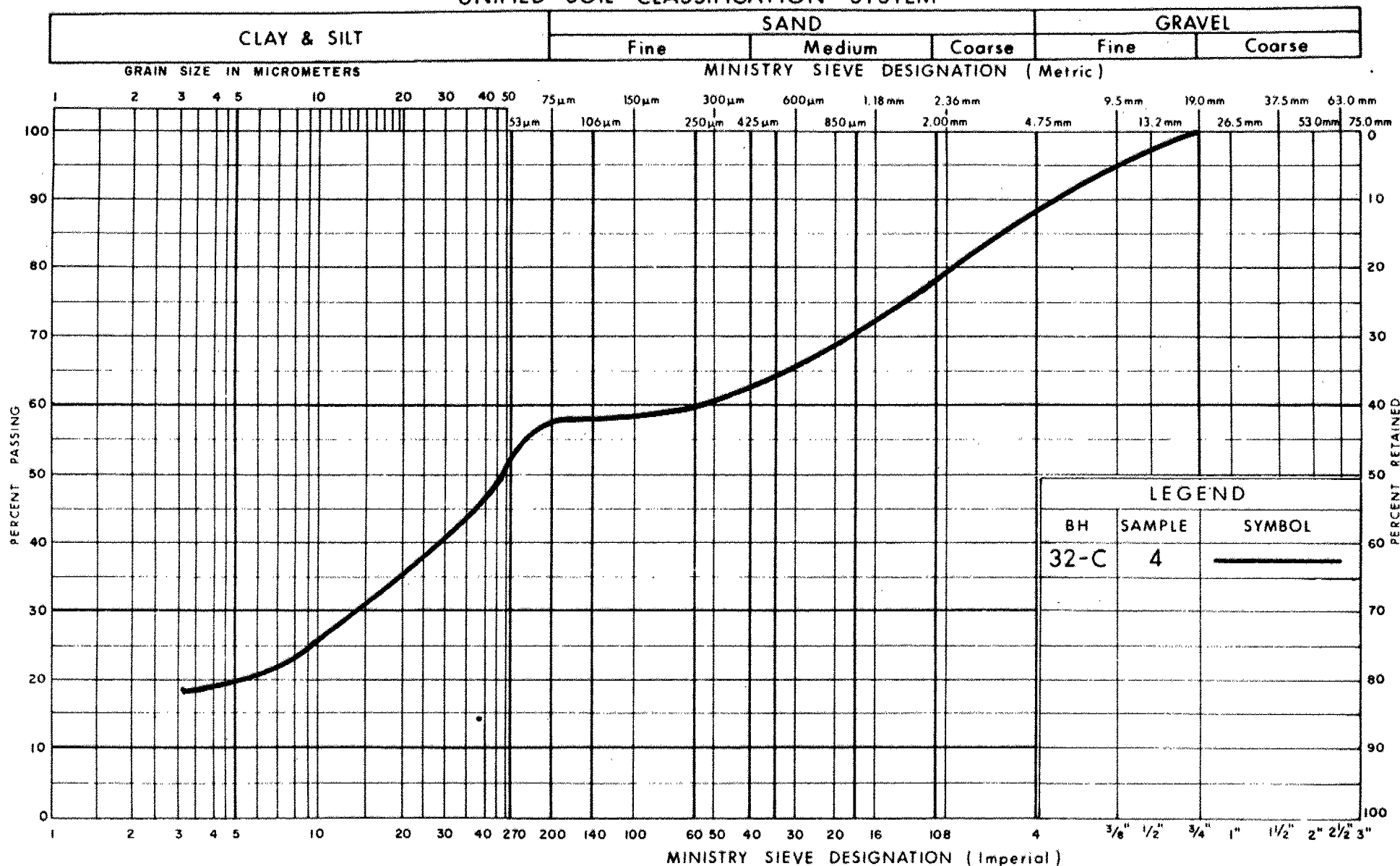
Ministry of  
Transportation

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

FIG No 2

W P 199-77-10

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

Ministry of  
Transportation

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

FIG No 3

W P 199-77-10

# RECORD OF BOREHOLE No 31-C 1 OF 1 METRIC

W.P. 199-77-09 LOCATION Co-ords: N 4 799 885; E 277 944 ORIGINATED BY JB  
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger COMPILED BY JB  
 DATUM Geodetic DATE September 19, 1991 CHECKED BY BJ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	w <sub>p</sub>	w	w <sub>L</sub>		
109.1	Ground Surface																
108.9	0.08m Topsoil Clayey Silt, Root Fibres		1	SS	17												
108.5	Dark Grey to Reddish Brown Clayey Silt, Root Fibres		2	SS	55/13cm												
0.6	Heterogenous Mixture of Sandy Silt, Some Gravel and Clay (Glacial Till) Very Dense		3	SS	60/13cm		108										
106.5	Refusal - Possible Bedrock		4	SS	60/14cm												
2.6	End of Borehole • Water level not established.																

# RECORD OF BOREHOLE No 32-A 1 OF 1 METRIC

W.P. 199-77-10 LOCATION Co-ords: N 4 799 820; E 277 952 ORIGINATED BY MM  
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger 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 <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	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		

113.1	Ground Surface																
0.0	Heterogeneous Mixture of Clayey Silt Some Sand and Gravel (Glacial Till)		1	SS	22												
			2	SS	16												
			3	SS	14												
	----- Very Stiff		4	SS	31												
	Hard		5	SS	78												
			6	SS	81												
			7	SS	130/23cm												
	Brownish Grey		8	SS	175/23cm												
106.6	Brown to Greyish Red																
6.5	Practical Refusal - Possible Bedrock																
	• Water level not established.																

## 1 OF 1

W.P. 199-77-10

LOCATION Co-ords: N 4 799 867; E 277 948

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

[illegible]

# RECORD OF BOREHOLE No 32-C 1 OF 1 METRIC

W.P. 199-77-10 LOCATION Co-ords: N 4 799 891; E 277 963 ORIGINATED BY MM  
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger / NO 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 <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
109.7	Ground Surface																
0.0	Heterogeneous Mixture of Clayey Silt Some Sand, and Gravel (Glacial Till)		1	SS	46												
108.3	Hard Brownish Grey		2	SS	100/15cm												
1.4	Greyish Red Heterogeneous Mixture of Sandy Silt, Some Gravel and Clay (Glacial Till)		3	SS	100/15cm												
106.0	Very Dense		4	SS	120/8cm												13 29 46 12
3.7	Weathered Sound		5	SS	120/15cm												
	Bedrock		6	RC	REC 57%												RQD 0%
	Shale		7	RC	REC 89%												RQD 0%
102.9	Containing Siltstone Interbeds																
6.8	End of Borehole																
	* The water level was measured in the open borehole on September 24, 1991.																



# RECORD OF BOREHOLE No 32-D 1 OF 1 METRIC

W.P. 199-77-10 LOCATION Co-ords: N 4 799 908; E 277 948 ORIGINATED BY AH  
 DIST 4 HWY 403 BOREHOLE TYPE S.S. Auger/BXL Coring COMPILED BY JB  
 DATUM Geodetic DATE September 20, 1991 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
109.6	Ground Surface																
0.0	0.6m Silty Clay Fill - Trace of Root Fibres Heterogeneous Mixture of Clayey Silt Some Sand and Gravel (Glacial Till)		1	SS	21												
108.2	Hard Brown to Brownish Grey Grayish Red		2	SS	85												
1.4	Heterogeneous Mixture of Sandy Silt, Some Gravel and Clay (Glacial Till)		3	SS	80/13cm												
106.5	Very Dense		4	SS	89/18cm												
3.1	Weathered Sand		5	SS	50/10cm												
	Bedrock Shale		7	RC	REC 60%									RQD 17%			
103.2	Containing Siltstone Interbeds		8	RC	REC 100%									RQD 80%			
6.4	End of Borehole																
	1991 09 24 * GROUND WATER CONDITIONS																
	PIEZO. NO.																
	GROUND WATER ELEVATION (Metres)																
	1																
	107.0																



**METRIC**

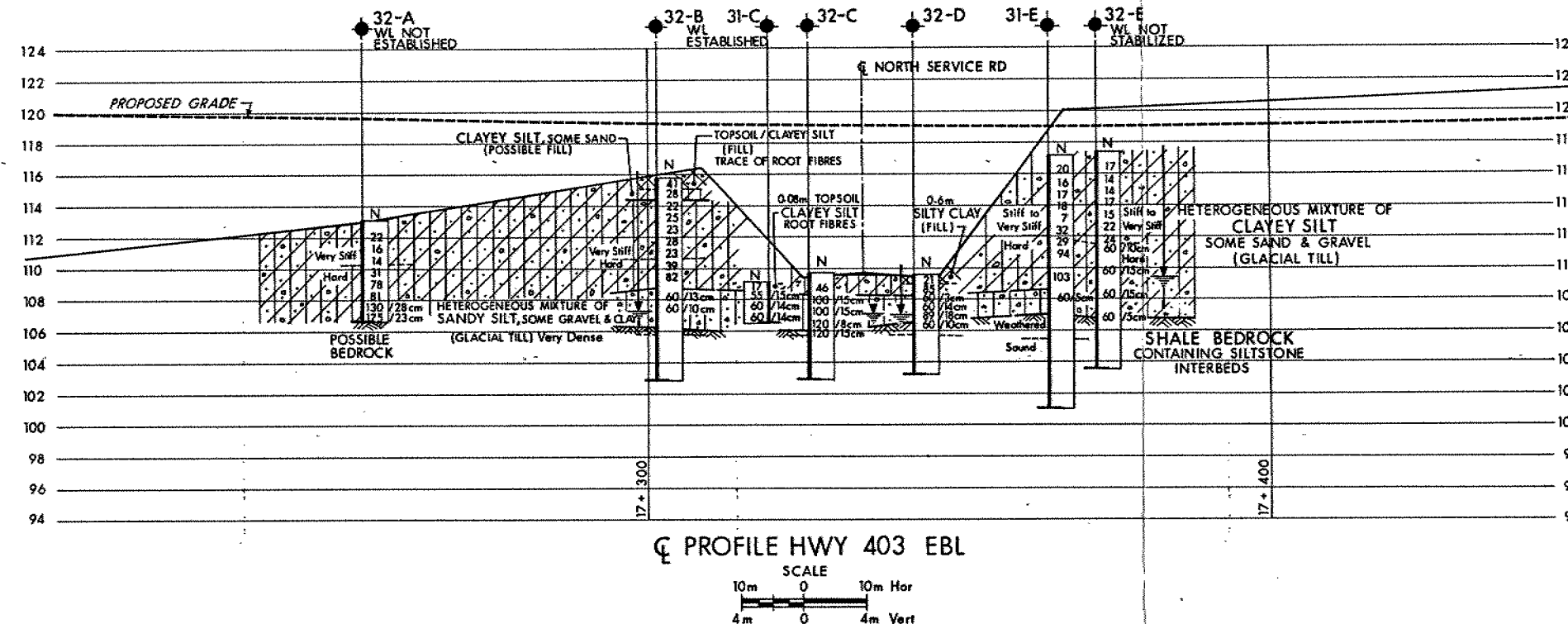
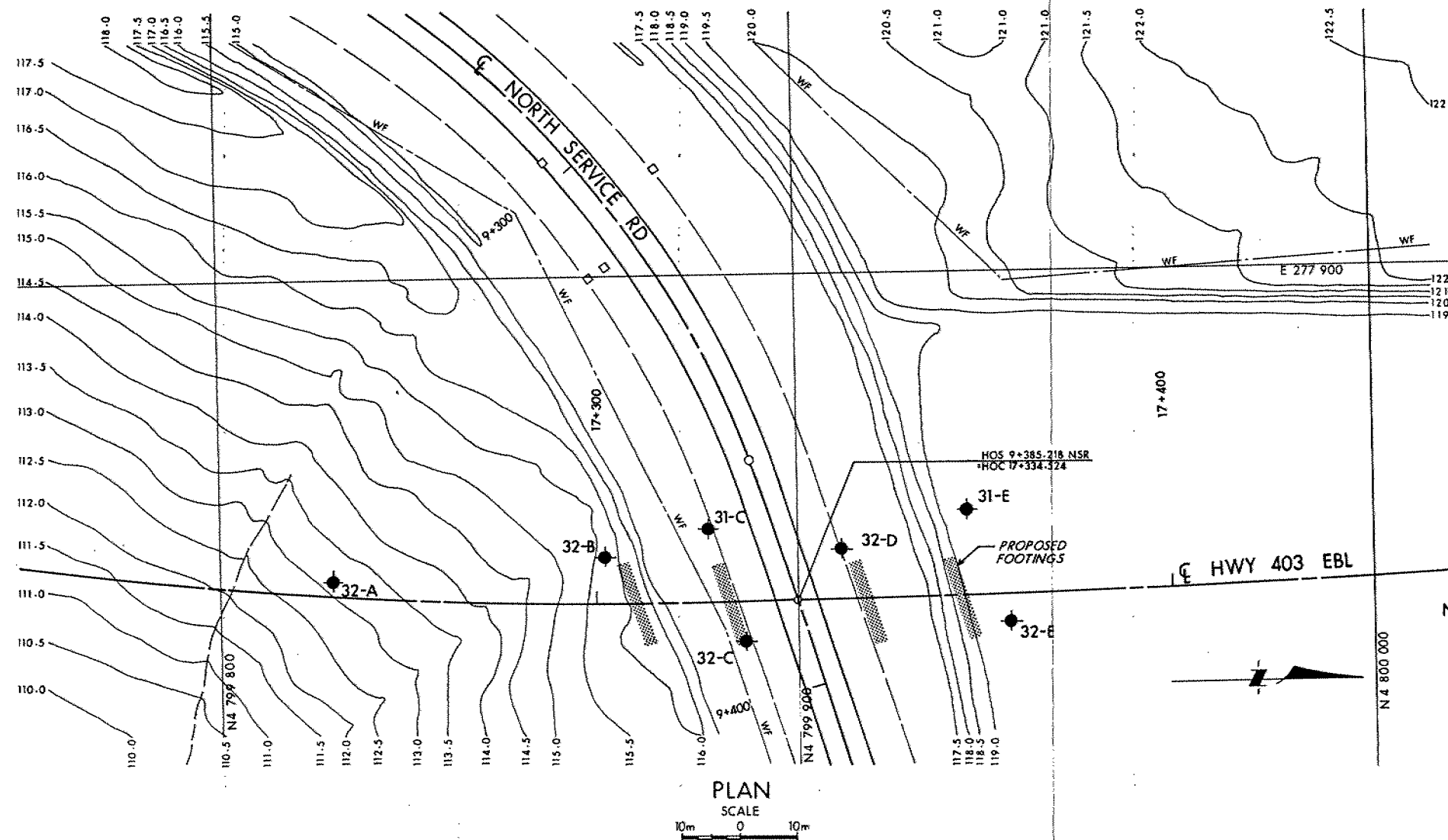
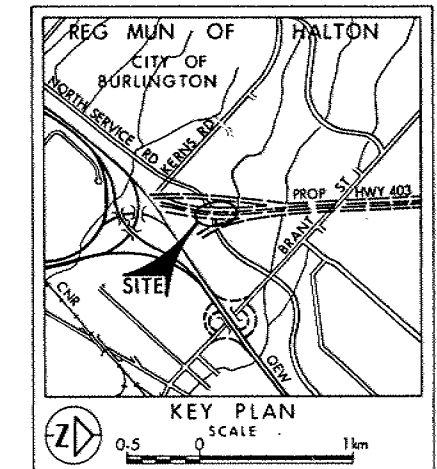
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES.

CONT No  
WP No 199-77-10

NORTH SERVICE RD  
HWY 403/GEW INTERCHANGE  
(BRIDGE-32)  
BORE HOLE LOCATIONS & SOIL STRATA



SHEET



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	WL at time of investigation 91 09		
W	WL in Piezometer		
P	Piezometer		

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
32-A	113.1	4 799 820.0	277 952.0
32-B	115.8	4 799 867.0	277 948.0
32-C	109.7	4 799 891.0	277 963.0
32-D	109.6	4 799 908.0	277 948.0
32-E	117.4	4 799 937.0	277 961.0
31-C	109.1	4 799 885.0	277 944.0
31-E	117.2	4 799 930.0	277 941.0

**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

Geocres No 30M5-186	
HWY No 403	DIST 4
SUBM'D JB [CHECKED]	DATE 91 12 12
DRAWN DT [CHECKED]	APPROVED
	SITE 10-484
	DWG 1997710-A

# MEMORANDUM



To: G. Al-Bazi  
Design Engineer  
Structural Office  
7th Floor, Atrium Tower

From: Foundation Design Section  
Room 315, Central Building

Re: North Service Road  
Hwy 403, W.B.L. and E.B.L.  
W.P. 199-77-09; Site 10-483  
W.P. 199-77-10; Site 10-484  
District 4, (Burlington)

Date: June 23, 1993

This is the summary of our discussion with you on 93 06 21 concerning the above structures.

- 1) In view of the proposed cut slope geometry and the encountered subsurface conditions, we recommend that the abutments be supported on piled foundations.

- 2) The steel "H" piles should be driven to the following levels:

W.P. 199-77-09

South and North Abutments: "Piles to be driven to bedrock" (El. 106±).

W.P. 199-77-10


South Abutment: "Piles to be driven to bedrock" (El. 106±)

North Abutment: "Piles to be driven in accordance with Standards SS 103-10 or SS 103-11 using an ultimate capacity of 3450 kN per pile but must be driven below El. 110.

- 3) For the purposes of the O.H.B.D.C., the following design values are recommended.

	<u>HP 310 X 110</u>	<u>HP 310 X 79</u>
Factored Axial Capacity at U.L.S.	1600 kN	1150 kN
Capacity at S.L.S. Type II	1150 kN	900 kN

- 4) All the piles should be equipped with driving shoes.

  
P. Payer, P. Eng.  
Senior Foundation Engineer  
for  
M. Devata, P. Eng.  
Chief Foundation Engineer

MD/PP/jb

c.c. - M. Bendayan

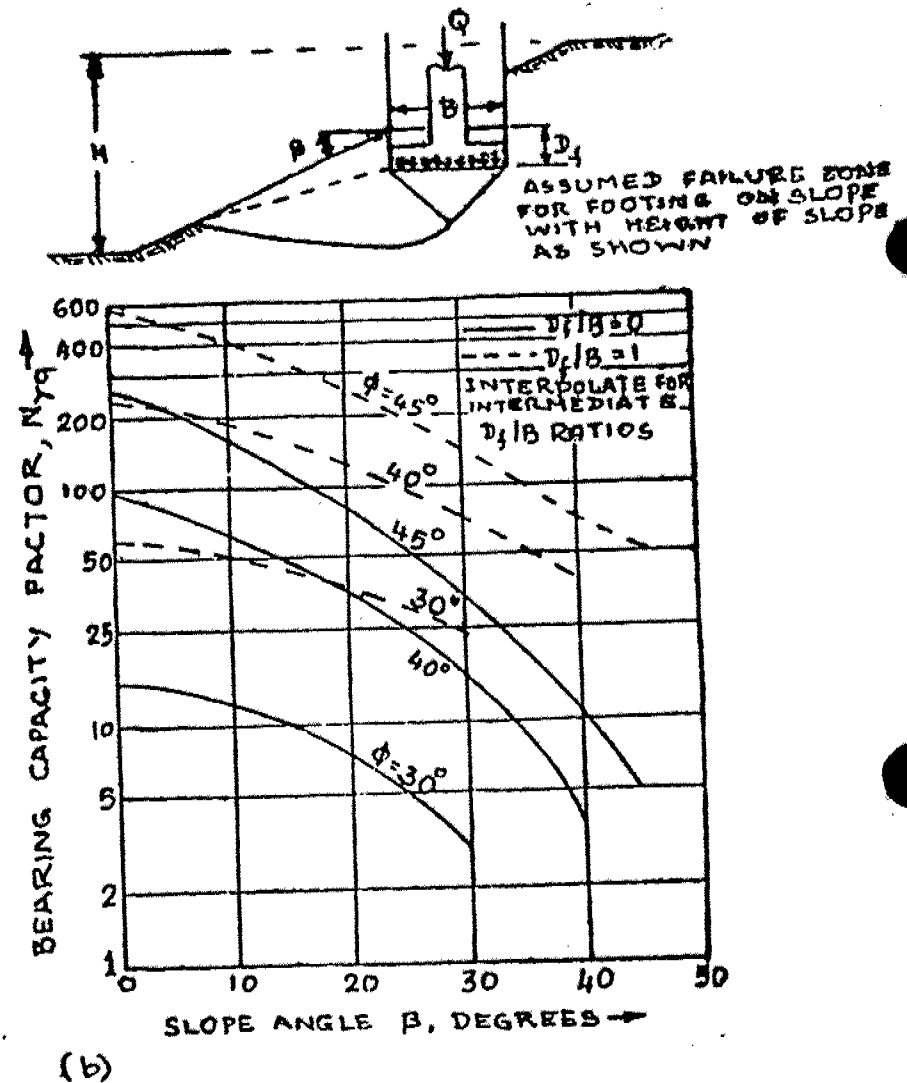
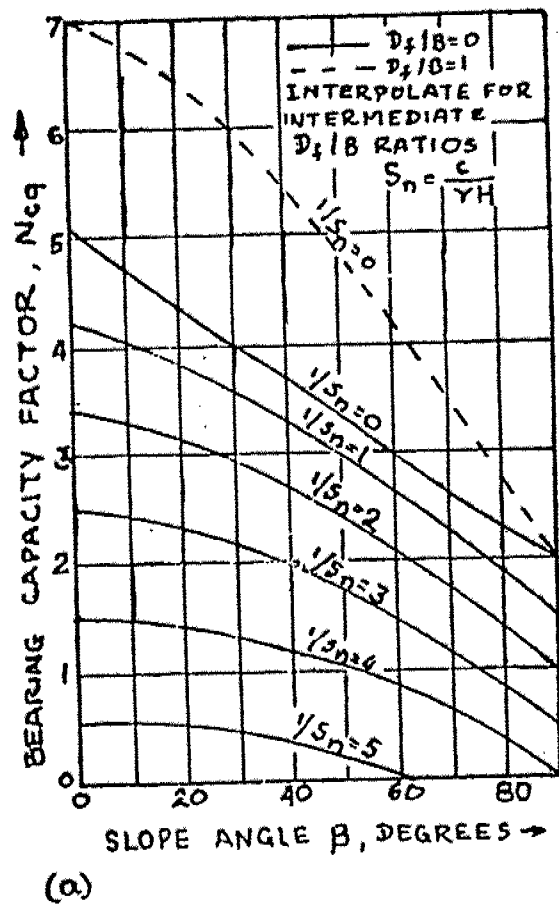


Fig. 2.7.6. Bearing capacity factors for a strip footing located on a slope  
 (a) For a purely cohesive soil  
 (b) For a purely cohesionless soil [After Meyerhof, 1965]

HWY. 403 W.B.L OVER NORTH SERVICE ROAD.



South Abutment:

Assume width  $B = 3.0 \text{ m}$ ,  $B:0 \Rightarrow N_c = 5.14$

$$D_f = 1.2 \text{ m}$$

$$C_u = 150 \text{ kpa} \Rightarrow N = 24$$

$$\gamma = 19 \text{ kN/m}^3$$

$$Q_u = 19 \times 1.2 + 5.14 \times 150 \left[ 1 + 0.2 \times \frac{1.2}{3} \right] \left[ 1 + 0.2 \times \frac{B}{L} \right]$$
$$= 22.8 + 832.7 \approx 855 \text{ kpa}$$

Assuming Factor of Safety 3  $\approx 285 \text{ kp}$ .

Say!  $q_a = \underline{300 \text{ kpa}}$  SLS  $R_{ec} = 300$

$$Q_u = 19 \times 1.2 + 5 \times 0.5 \times 150 \left[ 1 + 0.2 \times \frac{1.2}{3} \right]$$
$$= 22.8 + 405 \approx 430 \text{ k}$$

ULS = 450 kN

$R_{ec} = 600$

North Abutment:

Assume width  $B = 3.0 \text{ m}$

$$D_f = 1.2 \text{ m}$$

$$C_u = 120 \text{ kp} \Rightarrow N = 17$$

$$\gamma = 19 \text{ kN/m}^3$$

$$Q_u = 19 \times 1.2 + 5.14 \times 120 \left[ 1 + 0.2 \times \frac{1.2}{3} \right] \times 1$$
$$= 22.8 + 666 \approx 689 \text{ kpa}$$

assuming Factor of Safety of 3  $\approx 230$

Say!  $q_a = \underline{250 \text{ kpa}}$  SLS  $\Rightarrow 250 \text{ kp}$   $R_{ec} = 200$

$$Q_u = 19 \times 1.2 + 5 \times 0.5 \times 120 \left[ 1 + 0.2 \times \frac{1.2}{3} \right]$$
$$= 22.8 + 324 \approx 350 \text{ kpa}$$

ULS = 350 kpa

$R_{ec} = 400$

$$Q_u = 19 \times 1.2 + 3.9 \times 0.5 \times 120 [1.08] = 22.8 + 253$$

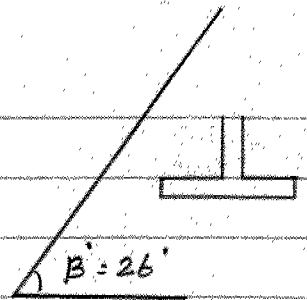
$$= \underline{275 \text{ kpa.}}$$

$$K_{BC} = 400 \text{ kpa.}$$

CONSIDERING THE FUTURE WIDENING & THE CLOSE PROXIMITY OF THE SLOPE, THE ABUTMENTS SHOULD BE SUPPORTED EITHER ON CAISSON OR H-PILES. IN THIS SITE CONDITIONS, CAISSON MAY BE ECONOMICAL.

HWY. 403 W.B.L OVER NORTH SERVICE ROAD

SOUTH ABUTMENT



Assume width  $B = 3.0 \text{ m}$ ;  $B = 26'$

$$D_f = 1.2 \text{ m}$$

$$C_u = 150 \text{ kPa} \implies \text{No 24}$$

$$\gamma = 19 \text{ kN/m}^3$$

Effective Height  $H = 10 \text{ m}$

$$\gamma = 20 \text{ kN/m}^3$$

$$S_n = \frac{C}{\gamma H} = \frac{150}{20 \times 10} = 0.75$$

$$\therefore 1/S_n = 1.3$$

$$D_f/B = \frac{1.2}{3} = 0.4$$

$$\therefore N_c = 3.9$$

$$\therefore Q_u = 19 \times 1.2 + 3.9 \times 150 [1.08] = 22.8 + 632 \approx 655$$

Assuming a Factor of Safety 3.

$$q_a = 655/3 \approx 218 \text{ kPa}$$

$$\therefore \text{Say } \underline{200 \text{ kPa}}$$

$$R_{ec} = 300 \text{ kPa}$$

$$Q_u = 19 \times 1.2 + 3.9 \times 0.5 \times 150 [1.08] = 22.8 + 316$$

$$\therefore \text{ULS} = 340 \text{ kPa}$$

$$R_{ec} = 600 \text{ kPa}$$

NORTH ABUTMENT

Assume  $C_u = 120 \text{ kPa}$

$$S_n = \frac{120}{20 \times 10} = 0.6$$

$$1/S_n = 1.67 \implies N_c = 3.3$$

$$B = 26'$$

$$Q_u = 19 \times 1.2 + 3.3 \times 120 [1.08] = 22.8 + 428 \approx 451 \text{ kPa}$$

Assuming a Factor of Safety of 3

$$q_a = 150 \text{ kPa}$$

$$R_{ec} = 200 \text{ kPa}$$



# 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. 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 to ground surface)

SEPT 24 / 91 09:00

APPROXIMATELY 25 hrs after last rain

	<u>BH</u>	<u>WATER LEVEL (m)</u>	<u>BOTTOM OF PIEZOMETER (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	$2.95 - 1.92 = 1.03$	
P	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

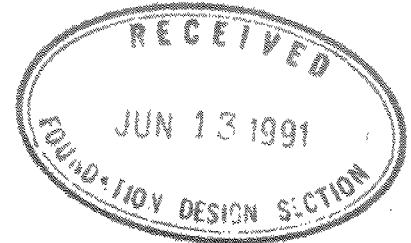
## MINISTRY OF TRANSPORTATION

### 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 (16m, 24m, 16m). 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 m 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.

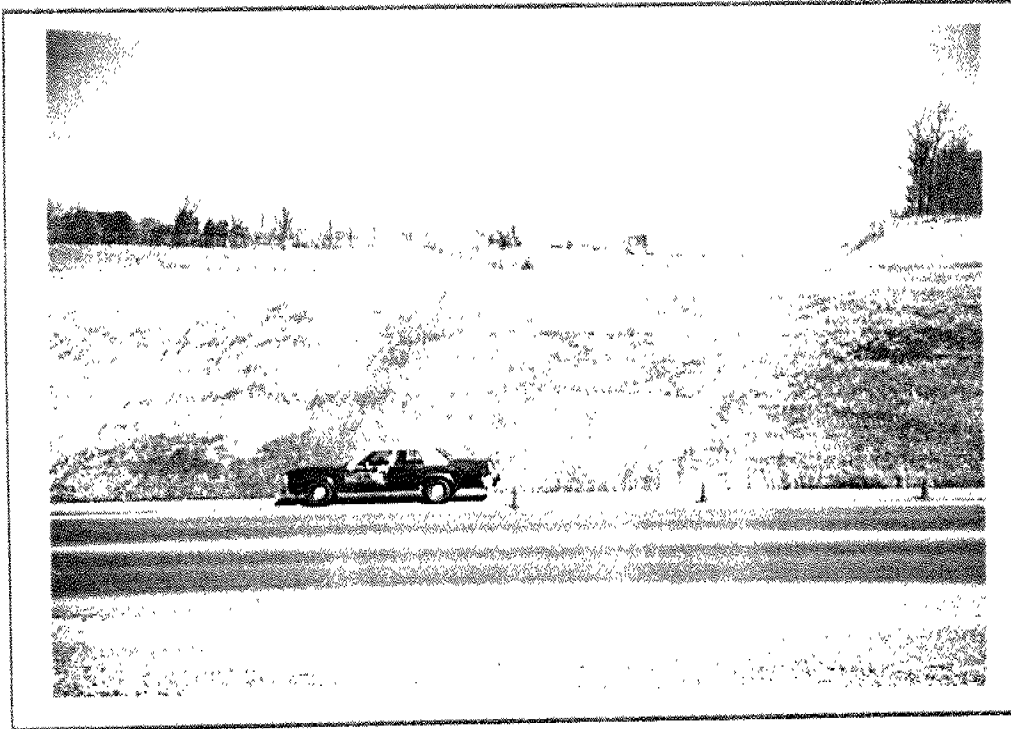


M. D. Bendayan  
Sr. Structural Engineer  
for:  
V. F. Boehnke  
Head, Structural Section

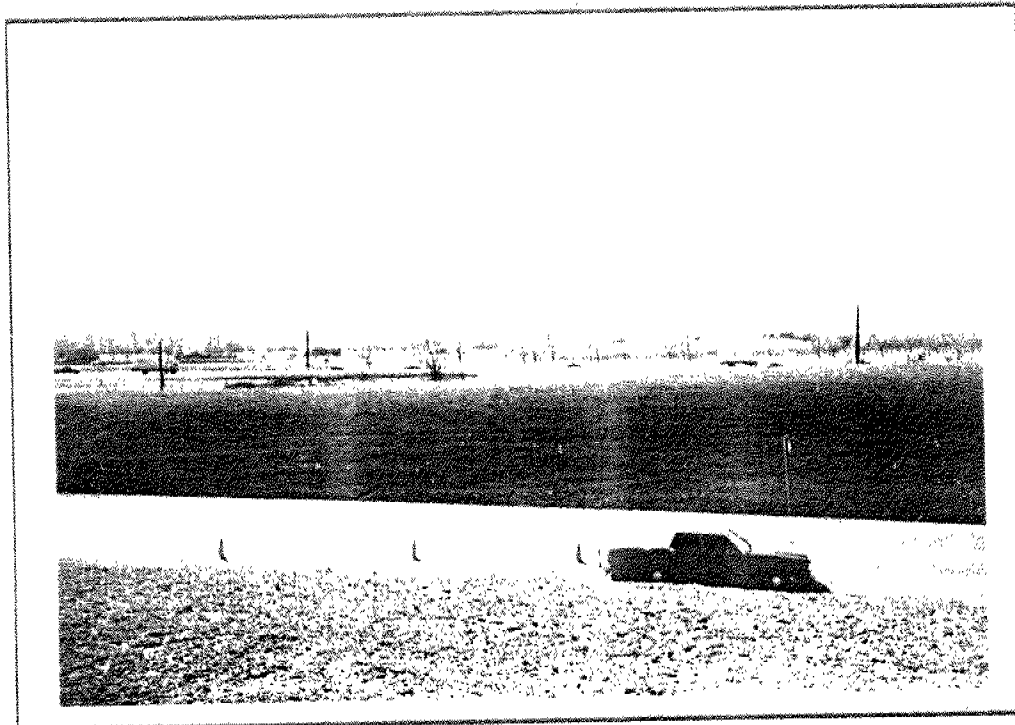
MB:dd  
Encl.

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

HWY. 403 E.B.L. OVER NORTH SERVICE ROAD (BRIDGE # 32)  
W.P. 199-77-10 ; SITE 10-484  
DISTRICT 4, BURLINGTON

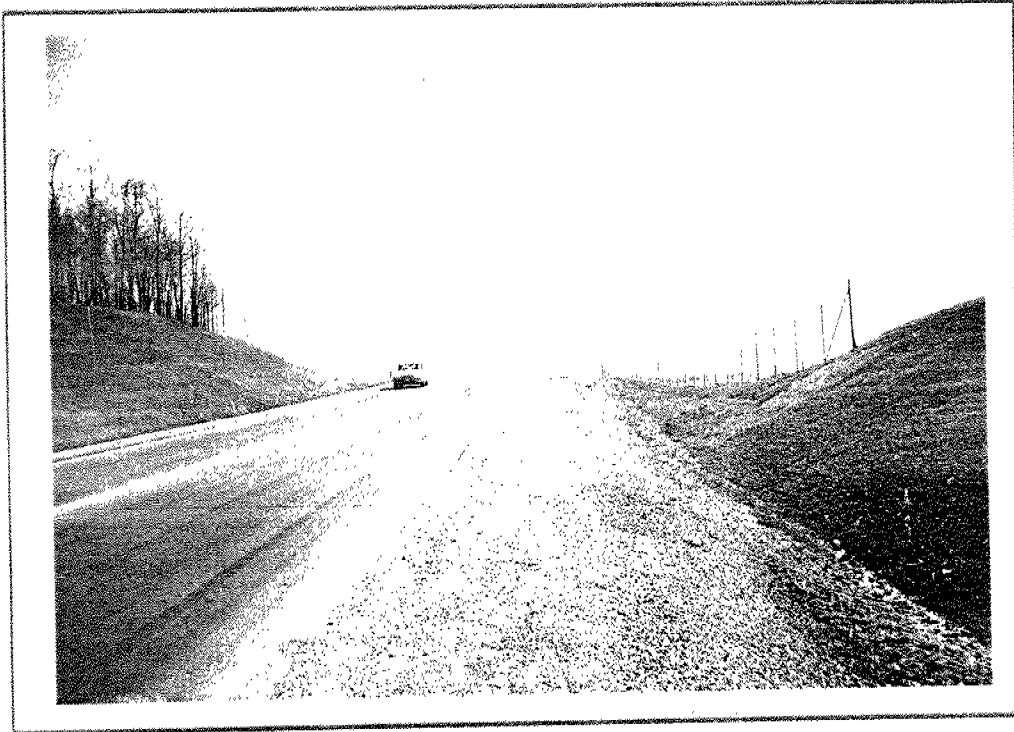


LOOKING NORTH ACROSS NORTH SERVICE ROAD

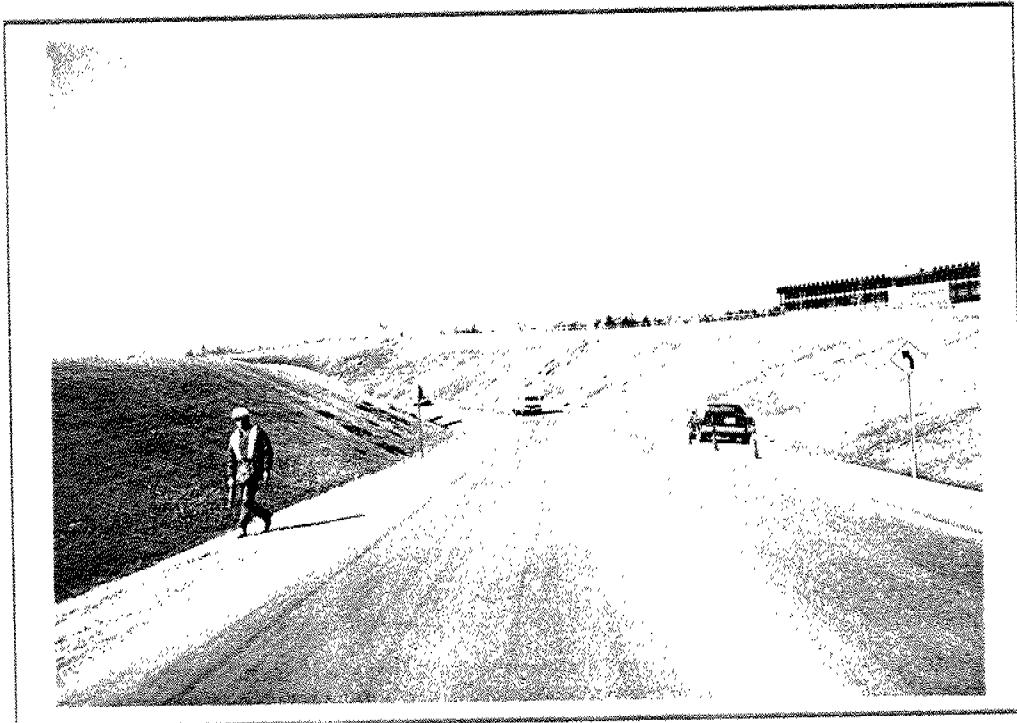


LOOKING SOUTH ACROSS NORTH SERVICE ROAD  
(FREEMAN INTERCHANGE IN BACKGROUND)

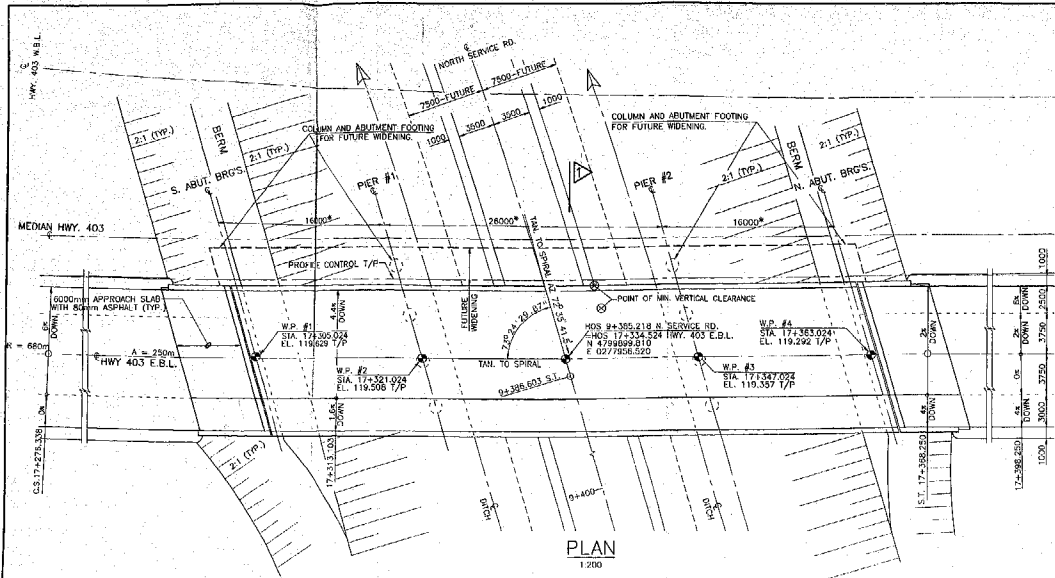
BRIDGE SITE 10-484  
BRIDGE # 32



LOOKING EAST ALONG NORTH SERVICE ROAD



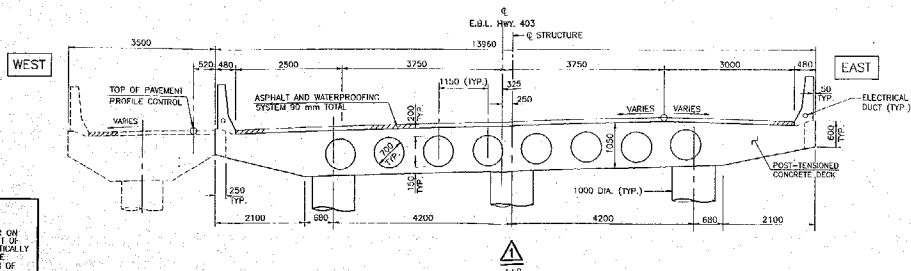
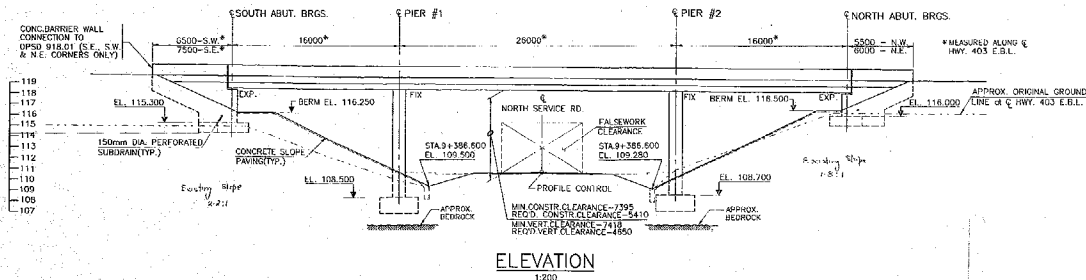
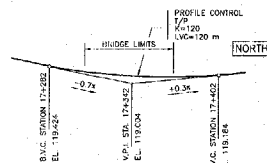
LOOKING WEST ALONG NORTH SERVICE ROAD



METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

#### LEGEND

W.P. -- DENOTES WORKING POINT  
T/P -- DENOTES TOP OF PAVEMENT



MTO BM 838011  
EL. 102.394m  
CONCRETE BASE OF SIGNAL TOWER ON  
SOUTH SIDE OF CULVERT 25.00m WEST OF  
UPPER BENCH MARK IS SET VERTICALLY  
IN TOP OF CONCRETE BASE AT THE  
NORTHWEST CORNER 2.40m SOUTH OF  
THE MOST SOUTHERLY WALL OF MAIN LINE

DIST 4  
CONT No  
WP No 199-77-10



HIGHWAY 403 EAST BOUND  
OVER THE NORTH SERVICE RD.  
(BRIDGE # 32)

GENERAL ARRANGEMENT

**MCCORMICK RANKIN**  
CONSULTING ENGINEERS

SHEET

#### GENERAL NOTES

##### CLASS OF CONCRETE

DECK & PIER COLUMNS 35 MPa  
REMAINDER 30 MPa

##### CLEAR COVER TO REINFORCING STEEL

FOOTINGS 100 ± 25  
ADJUSTERS & WINGWALLS: FRONT FACE 100 ± 25  
BACK FACE 70 ± 20  
PIERS 70 ± 20  
DECK TOP 70 ± 20  
BOT. & SIDES 50 ± 10  
REMAINDER (UNLESS OTHERWISE SPECIFIED) 70 ± 20

##### REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS NOTED OTHERWISE.  
BAR MARKS WITH SUFFIX "C" DENOTE COATED BARS.

##### CONSTRUCTION NOTES

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY  
DETECTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF  
BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE  
DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE  
CONTRACTOR SHALL ADJUST THE BEARING DECK STEEL TO SOIT.

#### LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. FOUNDATION LAYOUT AND FOOTING REINFT.
4. NORTH ABUTMENT
5. SOUTH ABUTMENT
6. WINGWALLS
7. PIERS
8. DECK DETAILS
9. LONGITUDINAL TENDONS
10. TRANSVERSE TENDONS
11. DECK REINFORCEMENT I
12. DECK REINFORCEMENT II
13. DECK REINFORCEMENT III
14. DECK REINFORCEMENT IV
15. JOINT ANCHORAGE AND ARMOURING
16. BARRIER WALL W/O RAILING
17. 6000 mm APPROACH SLAB
18. DETAILS OF CONCRETE SLOPE PAVING
19. AS CONSTRUCTED ELEV. & DIM.
20. STANDARD DETAILS
21. ELECTRICAL EMBEDDED WORK
22. QUANTITIES - STRUCTURE I

#### NORTH SERVICE ROAD ELEVATION AT TOP OF PAVEMENT

7.5m NORTH	6.0m NORTH	4.5m NORTH	SEASON CL. AT P.C.I.	4.5m SOUTH	6.0m SOUTH	7.5m SOUTH
FUTURE	EXISTING	EXISTING		EXISTING	EXISTING	FUTURE
110.445	110.335	110.381	9+362.5 110.385	110.189	110.099	110.125
110.601	110.525	110.573	9+375.0 110.524	110.434	110.344	110.374
110.797	110.737	110.797	9+386.6 110.797	110.707	110.617	110.647
110.765	110.720	110.767	9+397.5 110.816	110.726	110.637	110.666
111.058	111.042	111.117	9+400 110.160	111.070	110.880	110.010

#### APPLICABLE STANDARD DRAWINGS

QPSD 918.01 CONC. BARRIER WALL TRANSITION TO STRUCTURES  
QPSD 920.01 GRANULAR BACKFILL REQUIREMENTS  
QPSD 4602.00 FALSEWORK CLEARANCES



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

DESIGN	REV	CHK	APP	DATE	DESCRIPTION
DESIGN	1	CHK	APP	1993	STRUCT. SCHEME
DRAWN	JDC	CHK	KNT	1993	STRUCT. SCHEME
DATE	MAY 1993				