562

FOUNDATION INVESTIGATION AND DESIGN
QEW INTERCHANGE AT THIRD LINE
AND THIRD LINE FROM THE QEW NORTHERLY TO
KING'S COLLEGE DRIVE
REGIONAL MUNICIPALITY OF HALTON
GWP: 180-00-00
THIRD LINE / FOURTEEN MILE CREEK OVERPASS

Golder Associates

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
QEW INTERCHANGE AT THIRD LINE
AND THIRD LINE FROM THE QEW NORTHERLY TO
KING'S COLLEGE DRIVE
REGIONAL MUNICIPALITY OF HALTON
GWP: 180-00-00
THIRD LINE / FOURTEEN MILE CREEK OVERPASS

Submitted to:

Morrison Hershfield Limited
235 Yorkland Boulevard
Suite #600
Toronto, Ontario
M2J 1T1

DISTRIBUTION:

4 Copies - The Ministry of Transportation, Ontario, Downsview, Ontario

2 Copies - Morrison Hershfield Limited, Toronto, Ontario

2 Copies - Golder Associates Ltd., Mississauga, Ontario

September 2001 001-1158-02

TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

LIST OF FIGURES

LIST OF DRAWINGS

PART A

FOUNDATION INVESTIGATION REPORT
QEW INTERCHANGE AT THIRD LINE
AND THIRD LINE FROM THE QEW NORTHERLY TO
KING'S COLLEGE DRIVE
REGIONAL MUNICIPALITY OF HALTON
GWP: 180-00-00
THIRD LINE / FOURTEEN MILE CREEK OVERPASS

TABLE OF CONTENTS

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by Morrison Hershfield Limited (Morrison Hershfield) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation at the site of the proposed Third Line and Queen Elizabeth Way (QEW) Interchange in the Region of Halton, Ontario. The project involves reconstruction of the Third Line and QEW interchange and Third Line from the QEW northerly to King's College Drive. The project includes a new underpass structure to carry Third Line over the QEW, a bridge at Fourteen Mile Creek, a culvert extension, retaining wall, high embankments and removal and backfill of the existing Third Line overpass. This report addresses the Third Line / Fourteen Mile Creek overpass structure and culvert extension.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed structure by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. The terms of reference for the scope of work are outlined in our Total Project Management proposal P01-1321, dated October 2000. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, Agreement No. 2005-A-000290, dated October 2000.

The General Arrangement plan for the Third Line / Fourteen Mile Creek overpass structure showing the proposed abutment and pier layout has been provided to us on digital format in June 2001.

2.0 SITE DESCRIPTION

The bridge site is located immediately south and west of the existing intersection of Third Line and the existing W-N/S ramp where Fourteen Mile Creek crosses Third Line (see Drawing 1). The existing culvert runs underneath the QEW about 300 m west of the existing Third Line and the culvert will be extended to the south (see Drawing 1). Both sites are located in MTO District 4 in the Town of Oakville, in the Region of Halton.

The topography of the site area is generally level and slopes downwards towards the south. Fourteen Mile Creek flows in a southeasterly direction; the creek valley is about 6 m wide with valley slopes about 2 m high. Within the project limits, the vegetation cover generally consists of grass, bushes, and mature trees.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between February 14 and March 20, 2001. At this time, seven boreholes were put down at the site. Boreholes 1A, 2A, 3A and 4A were put down within the limits of the proposed bridge foundation units and Boreholes 5A and 8 for the approaches. Borehole C1 was put down at the south end of the existing culvert which crosses the QEW west of Third Line.

The investigation was carried out using a truck-mounted D-90 drill rig (for the borehole drilled on existing ramp) and bombardier-mounted B-57 drill rig (for the boreholes drilled elsewhere at the site) supplied and operated by Master Soil Investigation of Toronto. In the boreholes, samples of the overburden were obtained at regular intervals of depth of 0.75 m to 1.5 m using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. The boreholes were extended to depths of between 1.2 m and 9.1 m below the existing ground surface. NQ size core samples were obtained from the bridge foundation boreholes. Groundwater conditions in the open boreholes were observed throughout the drilling operations. A piezometer was installed in one borehole to permit monitoring of the groundwater level at the site. The piezometer consisted of a 200 mm long slotted tip threaded into 12 mm diameter PVC rigid tubing.

The field work was supervised on a full-time basis by a member of our engineering staff who located the boreholes in the field, directed the drilling, sampling and coring operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported to our laboratory in Mississauga for further examination. Index and classification tests consisting of grain size analyses, Atterberg Limits tests and water content determinations were carried out on selected samples. In total, 17 soil samples and 17 bedrock samples (obtained by split-spoon procedures) were obtained and 14 natural water content, 3 Atterberg Limits and 1 grain size distribution tests were performed.

The limits of the proposed bridge abutments were staked in the field by Morrison Hershfield. Based on the information provided, the northing and easting co-ordinates of the borehole locations are given in UTM, and the borehole elevations are referenced to the Geodetic Datum. The co-ordinates of the boreholes are indicated on the Record of Borehole sheets and the locations of the boreholes are shown on Drawings 1 and 2.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site is located in the physiographic region known as the Iroquois Plain. The Iroquois Plain is generally composed of shallow deposits of sand and till covering portions between Hamilton and Toronto (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984). The surface topography slopes gradually and fairly uniformly towards Lake Ontario. The overburden at the site consists of a shallow cover of residual soil which is underlain by bedrock comprised of red shale of the Queenston Formation.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site generally consist of 0.6 m to 2.6 m of clayey silt / silty clay fill and flood plain deposits underlain at some locations by a 0.5 m thick deposit of silty clay residual soil. The fill / flood plain deposits or residual soil is directly underlain by shale bedrock of the Queenston Formation.

The locations and elevations of the borings at the bridge site together with the interpreted stratigraphical profile and sections are shown on the attached Drawing 2. The locations and elevations of all the borings are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

4.2.1 Road Base Fill

Borehole 5A was drilled through the paved shoulder of the existing W-N/S ramp. The asphalt thickness was 200 mm and the crushed limestone sand and gravel road base was 400 mm thick. The SPT 'N' values on one sample of the sand and gravel was 29 blows per 0.3 m of penetration indicating that the material is compact. Water was noted seeping into the hole from this layer during and following drilling.

4.2.2 Clayey Silt to Silty Clay Fill

About 0.6 m to 2.0 m of fill consisting of red brown clayey silt to silty clay was encountered below the ground surface or road base fill. The fill contains some to with sand, trace to some gravel, trace rootlets, topsoil and trace shale fragments. Measured SPT 'N' values on samples of the fill range between 9 and 21 blows per 0.3 m of penetration indicating a stiff to very stiff consistency. The natural water content measured on selected samples of the fill ranged from about 12 to 28 percent. In Borehole C1, the fill directly overlies the bedrock surface. In the rest of the boreholes, the fill overlies the residual soil or flood plain deposits.

4.2.3 Clayey Silt Flood Plain Deposit

About 0.9 m to 1.1 m of red brown clayey silt, which is considered to be a flood plain deposit, was encountered below the fill in Boreholes 2A, 3A, 4A and 8, at about 0.6 m depth below ground surface. The deposit contains trace sand and gravel and trace shale fragments. Measured SPT 'N' values on samples of the flood plain deposits range between 11 and 58 blows per 0.3 m of penetration indicating a stiff to hard consistency. A grain size distribution curve for a selected sample of this deposit is shown on Figure 1. Atterberg Limits testing was carried out on two selected samples of the flood plain deposit. The liquid limits were 27 and 28 percent and the plasticity index was about 11 percent, indicating that the clayey silt is of low plasticity. The natural water content measured on selected samples of the flood plain deposit ranged from about 8 to 13 percent. In Boreholes 2A and 4A, this deposit directly overlies the shale bedrock.

4.2.4 Clayey Silt Residual Soil

A 0.5 m to 0.7 m thick deposit of residual soil was encountered below the fill or flood plain deposits in Boreholes 1A, 3A and 5A. The residual soil is derived through weathering of the underlying shale bedrock and is comprised of red brown clayey silt containing trace sand. This deposit was encountered at Elevations 100.5 m to 100.8 m in Boreholes 3A and 5A, respectively, and at Elevation 101.8 m in Borehole 1A. Measured SPT 'N' values on samples of the residual soil were between 54 blows and greater than 100 blows per 0.3 m of penetration, indicating a hard consistency. Atterberg Limits testing was carried out on one sample of the residual soil. The liquid limit was 27 percent and the plasticity index was 9 percent indicating that the clayey silt is of low plasticity. Where encountered, the residual soil directly overlies the shale bedrock.

4.2.5 Bedrock

Shale bedrock was encountered in all boreholes. The bedrock was augered for lengths of 0.6 m to 3.1 m in the boreholes prior to commencing rock coring in order to ensure that core samples could be retrieved. Occasional grinding during augering through the bedrock was noted. The shale bedrock surface was encountered at the Elevations shown in the table below:

Borehole	*Location*	*Ground Surface Elevation (m)*	*Depth to Bedrock Surface (m)*	*Bedrock Surface Elevation (m)*
5A	North Approach	103.4	3.1	100.3
1A	North Abutment	102.6	1.5	101.1
2A		102.6	1.7	100.9
3A	South Abutment	102.2	2.3	99.9
4A		101.6	1.5	100.1
8	South Approach	102.6	1.5	101.1
C1	Culvert	103.6	0.6	103.0

The shale bedrock was cored in Boreholes 1A, 2A, 3A and 4A. The bedrock is described as moderately to highly weathered, red brown, thinly laminated, fine-grained, very weak to weak, calcareous shale of the Queenston Formation. Rock Quality Designation (RQD) values were measured between 0 and 74 percent indicating rock of very poor to fair quality. The quality of the rock generally improved below about Elevation 99.3 m and 96.5 m at the northeast and northwest corners of the proposed north abutment and below about Elevation 98.6 m at the proposed south abutment. Seams of slightly less weathered, grey shale were encountered within the core samples. Zones of residual soil / completely weathered shale between 25 mm and 75 mm thick were encountered in Boreholes 1A, 2A and 3A.

In terms of the strength classification as noted above, weak rock encompasses rock with unconfined compressive strength between 5 MPa and 25 MPa. Based on the core samples obtained, the calcareous shale typically has strength closer to the upper limit of the range. Uniaxial compression strength testing was carried out on two samples obtained from a previous borehole investigation carried out by Golder Associates (Report No. 871-1526, dated April 1988). The tests indicate uniaxial compressive strengths of 27 MPa and 46 MPa. This would classify the rock as having medium strength, where medium strong rock encompasses rock with unconfined compressive strength between 25 MPa and 50 MPa.

4.3 Groundwater Conditions

Water levels were noted in the open boreholes during and upon completion of the drilling operation; these levels are shown on the attached Record of Borehole sheets. A piezometer was sealed into the bedrock in Borehole 1A. Details of the piezometer installation and water level measurements are shown on the attached Record of Borehole sheets. A summary of the water level monitoring results are provided in the following table.

Borehole	*On Completion of Overburden Drilling*		*Water Level in Piezometer*			
			March 23, 2001		*April 30, 2001*	
	Depth (m)	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*
1A	1.8*	100.8	1.6	101.0	1.8	100.9
2A	Dry	---	N/A	N/A	N/A	N/A
3A	Dry	---	N/A	N/A	N/A	N/A
4A	2.1	99.5	N/A	N/A	N/A	N/A
5A	Dry	---	N/A	N/A	N/A	N/A
8	2.1**	100.5	N/A	N/A	N/A	N/A
C1	Dry	--	N/A	N/A	N/A	N/A

* Measured 12 hours after rock coring.
** Measured 24 hours after overburden drilling (borehole was dry upon completion of drilling).

These water levels as well as those measured in boreholes to the north indicate that the groundwater level generally follows the ground surface topography at shallow depth and slopes downward toward the south. The water level is likely influenced by the water level in Fourteen Mile Creek. It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

GOLDER ASSOCIATES LTD.

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

Sarah E.M. Poot, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

SEMP/ASP/FJH/clg
\\MIS_MAIN\DATA\ACTIVE\1100\001-1158\2001\REPORTS\FINALREPORT_01SEPTEMBER_FOURTEENMILECREEK.DOC

PART B

FOUNDATION DESIGN REPORT
QEW INTERCHANGE AT THIRD LINE
AND THIRD LINE FROM THE QEW NORTHERLY TO
KING'S COLLEGE DRIVE
REGIONAL MUNICIPALITY OF HALTON
GWP: 180-00-00
THIRD LINE / FOURTEEN MILE CREEK OVERPASS

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of proposed Third Line / Fourteen Mile Creek overpass and the Fourteen Mile Creek culvert extension based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

It is understood that the proposed Third Line will be carried over Fourteen Mile Creek on an alignment to the west of the existing alignment and that the existing bridge is to be removed. The existing Fourteen Mile Creek structure is a single-span concrete rigid frame structure with a top elevation of about 103.5 m. The proposed structure will be a single-span bridge with a top elevation at about 109.8 m and sloping downwards towards the south. The proposed bridge will span 21 m from the north abutment to the south abutment. There will also be an 18 degree skew angle to the creek, giving a normal opening of about 20 m to the stream course.

It is also understood that the existing arch culvert which carries Fourteen Mile Creek underneath the QEW will be extended to the south by about 17 m to accommodate the widening of the QEW. The existing elliptical concrete culvert is 9.1 m wide and 4.6 m high. The top of the existing footings is at about Elevation 103.0 m. The proposed structure is expected to match the existing.

5.2 Bridge Foundations

The subsoils encountered in the boreholes put down during the present investigation typically consist of fill, flood plain deposits and hard clayey silt residual soil overlying shale bedrock. The groundwater table is at about Elevation 100.9 m in the immediate vicinity of the proposed bridge.

Based on the subsurface information above, consideration may be given to support of the structure on shallow spread footings placed on the surface of or at depth within the shale bedrock. Consideration may also be given to supporting the structure on steel piles driven to practical refusal within the weathered shale or on caissons socketted into the shale bedrock. In order to achieve a minimum driven pile length of 5 m, pre-augering through the shale bedrock to the full depth of the pile will likely be required to advance the piles.

For wing walls which are stepped up away from the bridge, consideration may be given to support on a compacted granular pad.

5.2.1 Shallow Foundations

Shallow spread footings may be used to support the abutments of the proposed bridge. Consideration can be given to placing the footings at the surface of the weathered bedrock or at depth on the more competent bedrock.

5.2.1.1 Geotechnical Resistance

The highest recommended founding levels for spread footings founded on the surface of the weathered shale bedrock or at depth within the bedrock (to be below the upper weathered / fractured zone) are given in the table below. The design bearing resistances for the two options are also given.

Design Founding Levels for Spread Footings	*Abutment*		*Axial Geotechnical Resistance*
Founding Option	*North*	*South*	
Spread Footings on Surface of Weathered Shale Bedrock	100.9 m	100.0 m	750 kPa (ULS) 600 kPa (SLS)
Spread Footings Within Shale Bedrock Below Upper Fractured Portion	97 m	98 m	1,500 kPa (ULS)*

- SLS not applicable.

Spread footings for the abutments should be a cost-effective foundation as suitable bearing material (weathered shale bedrock) is at relatively shallow depth. The founding level for the footings placed on the surface of the bedrock is up to 1 m below the groundwater level and for the deeper founding option, the founding level is up to 4 m below the ground water level. The above geotechnical resistances assume that appropriate construction procedures are adopted to handle any seepage inflow during footing construction to ensure that the bedrock is not softened / disturbed prior to concrete placement. Given the potential quantity of groundwater inflow through the fractured bedrock, it is considered that the higher founding alternative given above is the most appropriate for this site. Where footings may be subjected to scour, rip-rap protection may be required.

For wing wall footings which are stepped up and back from the bridge and will therefore be placed above the surface of the weathered shale bedrock, the founding level may be raised by placement of a compacted Granular 'A' pad up to the founding level. All fill material and organic flood plain materials must be removed from the granular pad footprint area prior to the placement of granular material. The exposed subgrade should be proof-rolled and any soft spots sub-excavated and replaced with compacted granular fill. The Granular 'A' should be placed in regular lifts not exceeding 200 mm loose thickness and be compacted to 100 percent of the materials Standard Proctor maximum dry density. The flood plain deposits present at the site are variable and it is recommended that a minimum thickness of Granular 'A' equal to 1.5 times the footing width be provided under the wing wall footings unless residual soil and / or bedrock are present at shallower depth. In this case, the granular thickness can be reduced. This will likely require sub-excavation of all of the flood plain deposits for the portion of the wing walls close to the bridge. For the portion of wing wall furthest away from the abutment, the grade will be raised above the existing ground surface as part of the embankment construction prior to placing the Granular 'A' pad.

The axial geotechnical resistance at ULS for spread footings placed on a compacted Granular 'A' pad as outlined above may be taken as 400 kPa. This value assumes a footing width of 1 m and minimum granular pad thickness of 1.5 m. The settlement of footings placed on a granular pad will be dependent on the thickness of flood plain deposits under the granular pad. For design, the geotechnical resistance at SLS may be taken as 250 kPa. This value should be confirmed once the configuration of the wing wall footings and Granular 'A' pad thickness is established.

5.2.1.2 Resistance to Lateral Forces

Resistance to lateral forces / sliding resistance between the concrete spread footings and subsoil should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the properly prepared shale bedrock should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" wing wall footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.2.1.3 Frost Protection

All footings should be provided with a minimum of 1.2 m of earth cover for frost protection purposes.

5.2.1.4 Construction Considerations

For the alternative founding level for the bridge abutments placed on the surface of the weathered shale, the base of footing excavations will be at or up to 1 m below the groundwater level. For the alternative of founding deeper within the bedrock, the footing excavations will be extended up to 4 m below the groundwater level. Some water seepage into footing excavations should be expected for excavations extended below the creek water level at about Elevation 101 m. It should be noted that the water levels could be higher during wet periods of the year.

Significant inflows should be expected through the weathered, fractured shale where the excavations are extended to the deeper founding level. The quantity of seepage expected for the higher founding alternative will be less and will depend on the presence of more permeable zones within the overburden and / or fractured zones within the bedrock. It is expected that pumping from well-filtered sumps placed at the base of the excavation should provide sufficient groundwater control during foundation excavations. The number of sumps required will depend on the size, depth and duration of the footing excavation. Sumps should be maintained outside of the footing area and surface water run-off should be directed away from the excavation at all times. The appropriate NSSP should be included in the contract documents.

The shale bedrock foundation is sensitive to disturbance and softening due to water seepage or ponding. Any water should be directed away from the footing area at all times. Placement of a lean concrete mud coat will be required at the base of the excavation for the footing area. The cleaned excavation base should be inspected by qualified geotechnical personnel. The mud coat should be placed as soon as practical after footing inspection.

5.2.2 Deep Foundations

Consideration could be given to the use driven piles (for integral abutments) or caissons for support of the bridge abutments.

5.2.2.1 Geotechnical Resistance – Steel H-Piles

Depending on the proposed road grade, support of the structure on driven steel piles is probably not practical due to the shallow depth to shale bedrock. If the option of steel H-piles is to be considered for the abutments, pre-augering through the fill and shale bedrock for the full depth of the pile will be required. Steel H-piles could then be installed within the pre-augered hole and driven to practical refusal within the shale bedrock which was encountered at about Elevation 100 m and

101 m at the south and north abutments, respectively. For design, the factored axial resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to practical refusal within shale bedrock may be taken at 2,000 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken at 1,600 kN.

The steel H-piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ but not exceeding 60 kJ. Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven in accordance with Special Provision 903S01. The pile tips should be stiffened with MTO flange plates for protection during driving.

5.2.2.2 Geotechnical Resistance - Caissons

The use of caissons socketted into the shale bedrock below the upper fractured portion may also be considered as an alternative for the foundations. The fractured bedrock extends to about Elevation 97 m and 98 m at the north and south abutments, respectively. The load carrying capacity for caissons depends on the total length of the caissons, the length of the rock socket and the diameter of the caissons. Where the length to diameter ratio is less than 5, it is recommended that the caissons be designed as end bearing units. In this regard, an axial geotechnical resistance at ULS of 3.4 MN and 5.3 MN may be assumed for design for 1.2 m and 1.5 m diameter caissons, respectively, assuming the caissons are extended to Elevation 97 m and 98 m at the north and south abutments, respectively. Serviceability Limit States (SLS) does not apply for caissons founded within the unweathered shale bedrock at this site.

The use of caisson has the advantage of speed of operation and limited space requirements for installation. In addition, tremie concreting procedures may be used for concrete placement if the groundwater inflow cannot be readily handled by pumping from the base of the caissons. In this regard, sufficient inflows should be anticipated in the caisson excavations such that inspection of the base will not be feasible. The above design capacities, however, assume that the base is thoroughly cleaned of loose material prior to pouring concrete. A temporary liner must be provided through the overburden if the caisson is commenced above the bedrock surface.

5.2.2.3 Resistance of Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If integral abutments are considered, the vertical piles must provide the resistance to the lateral loading. In this case, the horizontal reaction to the pile can be estimated using the following equation and the range of values given in the table below where:

K_h = coefficient of horizontal subgrade reaction (MPa/m) = $K_{s1}/5d$

d = pile width or diameter (m)

K_{s1} = constant of horizontal subgrade reaction (MPa/m)

Soil Type	*K_{s1} (Mpa/m)*
Weathered Shale Bedrock	100 to 150

The resistance should be checked for the range of values as given and the design be based on the least or greatest resistance as appropriate.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

Pile Spacing in Direction of Loading D = Pile Diameter	*Subgrade Reaction Reduction Factor R*
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.2.2.4 Frost Protection

The pile caps should be provided with 1.2 m soil cover for frost protection.

5.3 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments in accordance with OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS501.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical up and back from the bottom of the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight (assuming clean earth fill)	20 kN/m^3
Coefficients of lateral earth pressure:	
'active'	0.43
'at rest'	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B' Type II**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficients of Lateral Earth Pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Where there is sloping ground behind the wall, the coefficient of lateral earth pressure must be adjusted (increased) to account for the slope. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3501.00.

5.4 Embankments

At the location of the proposed Third Line / Fourteen Mile Creek structure, the proposed Third Line will be constructed entirely in fill with approach embankment heights of up to 6.5 m.

5.4.1 Embankment Design

The approach embankment subgrade soils consist of a surficial layer of fill and flood plain deposits (1.5 m to 2.6 m thick) underlain by weathered shale bedrock. Providing that the fill and organic flood plain deposits are removed from the embankment subgrade and the exposed surface proof-rolled, the embankment with side slopes maintained at 2 horizontal to 1 vertical would be stable. Settlement of the subsoils below the embankment is expected to be less than 50 mm. Settlement of the new embankment fill itself will occur; the magnitude will depend on the type of fill used and is expected to be less than 75 mm if clean earth fill is used. If the fill is cohesive, much of the settlement would occur after construction. In this case, final paving could be delayed by about 6 months to allow the settlement to take place. The use of granular fill for embankment construction would reduce the amount of settlement since the majority of settlement of granular fills will occur during construction.

Where the proposed embankment height is greater than 8 m, a mid-height berm with a platform width of 2.0 m will be required on both sides of the embankment for the approaches.

Scour protection of the embankments in the vicinity of the bridge should be provided by means of rip-rap to an elevation that is 0.5 m above the high water level, i.e. the flood level.

5.4.2 Embankment Construction

The existing fill and organic flood plain material should be stripped from below the fill embankment areas. The subgrade should be proof-rolled and any softened zones exposed by the proof-rolling should be sub-excavated and replaced with compacted granular fill. Construction of the embankment above the prepared subgrade may be carried out using clean earth fill meeting the specifications of OPSS 212 or Select Subgrade Material meeting the specifications of OPSS 1010, depending on material availability. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The permanent soil slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical (2H:1V). Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion, as per OPSS 572.

5.5 Culvert Extension

It is understood that the existing arch culvert that carries Fourteen Mile Creek beneath the QEW, west of the existing Third Line, will be extended in length by about 17 m on the south side of the QEW. It is further understood that the extension will be an arch culvert, to match the existing, founded on shallow spread footings. The existing concrete culvert is 9.1 m wide and 4.6 m high and the top of the existing footings is at about Elevation 103.0 m.

5.5.1 Geotechnical Resistance – Culvert Extension

The subsoils at the site consist of a thin veneer of fill underlain by weathered shale bedrock at about Elevation 103.0 m. It was noted during the field investigation that the bedrock was outcropped within the creek bed on the south side of the existing culvert. Based on the results of bedrock coring elsewhere at the site, the upper 1 m of the shale is highly weathered and becomes moderately weathered with depth. The creek water level is at about Elevation 103.3 m.

It is assumed that the culvert extension will be an arch culvert, matching the original, founded on the moderately weathered shale bedrock. ~~The existing footings are founded at about Elevation 102 m~~. A factored geotechnical resistance at Ultimate Limit States (ULS) of 750 kPa may be assumed for design. A geotechnical resistance at Serviceability Limit States (SLS) of 600 kPa may be assumed for design, based on a total settlement of 25 mm.

ASF Feb 26 '0 Assumed ht. of ftg is 1.8m based on MH info

Foundations placed at Elevation 101.2 ~~102.2~~ m will be below the creek water level and the creek will have to be diverted during construction of the culvert. Significant inflows should be expected through the weathered, fractured shale. The number of sumps required will depend on the size, depth and duration of the footing excavation. It is expected that pumping from well-filtered sumps placed at the base of the excavation should provide sufficient groundwater control during foundation excavations. Sumps should be maintained outside of the footing area and surface water run-off should be directed away from the excavation at all times. The appropriate NSSP should be included in the contract documents.

The shale bedrock foundation is sensitive to disturbance and softening due to water seepage or ponding. Any water should be directed away from the footing area at all times. Placement of a lean concrete mud coat will be required at the base of the excavation for the footing area. The cleaned excavation base should be inspected by qualified geotechnical personnel. The mud coat should be placed as soon as practical after footing inspection.

5.5.2 Subgrade Reaction – Arch Culvert

The coefficients of horizontal and vertical subgrade reaction, k_n and k_v respectively, to be used for design of the arch culvert footings are given by the following equations and the range in constants of subgrade reaction given below:

$$k_h = k_{s1}/2d$$
$$k_v = k_{s1}$$

where: d = depth into the stratum

k_{s1} = 80 MPa/m to 150 MPa/m for highly weathered shale bedrock below Elevation 103 m

= 150 MPa/m to 300 MPa/m for moderately weathered shale bedrock below Elevation 102 m

The design should be checked for both the low end and the high end of the range of values given.

5.5.3 Backfilling

Based on the factual information obtained from this investigation and assuming that the founding level of the culvert is about 3 m depth, the proposed culvert extension will be founded on the weathered shale bedrock.

For protection of the founding stratum a working mat of lean concrete should be placed as soon as practical after reaching the base of the excavation and following completion of inspection.

Backfill to the culvert should be in accordance with OPSD 803.010 and 803.02. For general backfilling within the future roadway limits, granular fill meeting the specifications for OPSS Granular 'B' Type 2 should be used. Representative samples of the materials proposed for use should be submitted to a qualified laboratory for suitable testing and determination of laboratory Proctor values prior to placement and compaction. All backfill should be placed in lifts not exceeding 300 mm loose thickness and be compacted to 95 percent of the Standard Proctor dry density. Inspection and testing should be carried out by qualified geotechnical personnel during fill placement and compaction. Scour protection should be provided by means of rip-rap to the slopes in the vicinity of the culvert outlet.

Sediment control such as silt fences, erosion control blanket may be required during construction and diversion of the creek to mitigate migration of fine soil particles in to the creek.

5.6 Excavations and Temporary Cut Slopes

Excavations for footing construction for the bridge abutments and culvert extension will generally extend through clayey silt / silty clay fill and weathered shale bedrock. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the excavation could also be carried out within the fully braced excavation.

Where space is restricted and / or roadway protection is required for footing construction at the proposed bridge location, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. The temporary support system could consist of soldier piles and lagging where the piles would be socketted into pre-augered holes extended into the weathered shale bedrock below the excavation base. Some cobbles and boulders should be expected during augering for the soldier pile installation. Support to the soldier pile and lagging wall system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of retaining wall excavations.

The design of braced soldier pile and lagging walls should be based on a rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker / anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

Unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth), can be calculated as follows:

$$p = K_a \gamma H$$

where

H = the height of the excavation at any point in metres

K_a = 0.3 for level ground behind excavation

γ = soil unit weight = 21 kN/m^3

Unfactored rectangular earth pressure distribution (p in kN/m^2; constant with depth), can be calculated as follows:

$$p = K \gamma H$$

where

H = the height of the excavation

K = 0.25 for level ground behind excavation

γ = soil unit weight = 21 kN/m^3

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter. The coefficient of passive lateral earth pressure, K_p, for the socket within the hard residual soil or weathered bedrock may be taken as 8.7. The weathered bedrock unit weight should be taken as 21 kN/m^3. A groundwater elevation at Elevation 101 m can be assumed at the proposed bridge foundation units.

Where roadway protection is required, grouted rock anchors are feasible and may be designed based on the following ultimate bond stresses between grout and rock:

300 kPa – over the upper 3.0 m of bedrock

600 kPa – below the upper 3.0 m of bedrock

A factor of safety of 2.0 should be applied to the ultimate rock anchor capacity calculated from the above adhesion values. The maximum permissible stress in the anchor tendon or bar under the design load should not exceed 0.625 of the guaranteed ultimate tensile strength of the tendon or bar.

A performance test should be carried out on at least one anchor to confirm the design and the Contractor's installation method. The performance test should be carried out to 2.0 times the design working load. In addition, each anchor should be proof tested to 1.5 times its working load. The tensile stress in the anchor bar during test loading should not exceed 0.8 of the guaranteed ultimate tensile strength of the bar. Anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. Anchor installation and preloading should be complete before the excavation proceeds below the anchor elevation.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health & Safety Act.

GOLDER ASSOCIATES LTD.

Sarah E.M. Poot, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

SEMP/ASP/FJH/clg
\\MIS_MAIN\DATA\ACTIVE\1100\001-1158\2001\REPORTS\FINALREPORT_01SEPTEMBER_FOURTEENMILECREEK.DOC

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_p plasticity Index $= (w_l - w_p)$
w_s shrinkage limit
I_L liquidity index $= (w - w_p)/I_p$
I_C consistency index $= (w_l - w)/I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_s swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p/\sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = tan δ
c' effective cohesion
c_u, s_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan\phi'$
2. Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
II - Parallel To	
⊥ - Perpendicular To	

Form...G.A.-R-3

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1A

METRIC

PROJECT 001-1158			ORIGINATED BY PKS
W.P. 180-00-00	LOCATION N 4809217; E 286715		COMPILED BY SEP
DIST 4	HWY QEW	BOREHOLE TYPE 114mm Solid Stem Augers	CHECKED BY ASP
DATUM Geodetic	DATE Feb 21/01		

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
102.6	GROUND SURFACE			
0.0	Clayey Silt to Silty Clay, some sand, trace gravel and shale fragments, topsoil rootlets (Fill) Stiff to hard Red-brown Moist	1	SS	9
101.8 / 0.8	Clayey Silt with shale fragments (Residual Soil) Hard Red-brown Moist	2	SS	58
101.1 / 1.5	Highly to moderately weathered, red brown with occasional grey seams, calcareous SHALE BEDROCK (Queenston Formation).	3	SS	100/.13
	Bedrock cored from 4.6m to 9.1m.	4	SS	105
	For bedrock coring details see Record or Drillhole 1A.	~~5~~	~~SS~~	~~100/.08~~
		~~6~~	~~SS~~	~~100/.08~~
93.5 / 9.1	END OF BOREHOLE			

ELEVATION SCALE: 102, 101, 100, 99, 98, 97, 96, 95, 94

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

GROUND WATER CONDITIONS

Note:
1. Water level in open borehole (12 hours after bedrock coring) at 1.8m depth (Elev.100.8m) approximately creek level.
2. Water level in piezometer at 1.6m depth (Elev. 101.0m) on March 26, 2001 and at 1.8m depth (Elev. 100.8m) on April 30, 2001.

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 1A

PROJECT: 001-1158

LOCATION: N 4809217; E 286715

INCLINATION: -90°   AZIMUTH: ---

DRILLING DATE: Feb.21/01

DRILL RIG: B-57

DRILLING CONTRACTOR: Master Soil Investigation Ltd.

SHEET 1 OF 1

DATUM: Geodetic

FR-FRACTURE	F-FAULT	SM-SMOOTH	FL-FLEXURED	BC-BROKEN CORE
CL-CLEAVAGE	J-JOINT	R-ROUGH	UE-UNEVEN	MB-MECH. BREAK
SH-SHEAR	P-POLISHED	ST-STEPPED	W-WAVY	B-BEDDING
VN-VEIN	S-SLICKENSIDED	PL-PLANAR	C-CURVED	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH (COLOUR % RETURN)	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous page		98.00												
5	NQ	Highly to moderately weathered, red brown with occasional grey seams, thinly laminated, fine-grained, weak, calcareous SHALE (Queenston Formation).		4.60	1	1.2	100						B,PL,SM (x11)			
6													B,R,IR B,PL,SM (x9) B,SM,PL B,SM,C B,R,ST B,SM,PL (x5)			
7					2	1	90						B,SM,PL (x3)			
8													B,SM,PL (x8) J,SM,PL (x3) B,SM,PL (x5)			
		Completely weathered zone from 8.2m to 8.3m depth.			3	0.9	100						B,SM-R,PL (x2)			
9				93.50 9.10												
		END OF BOREHOLE														
10																
11																
12																
13																
14																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: GM

CHECKED: ASP

RECORD OF BOREHOLE No 2A

METRIC

PROJECT 001-1158			
W.P. 180-00-00	LOCATION N 4809228; E 286731		ORIGINATED BY GM
DIST 4	HWY QEW	BOREHOLE TYPE 114mm Solid Stem Augers	COMPILED BY SEP
DATUM Geodetic	DATE Feb.20/01		CHECKED BY ASP

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
102.6	GROUND SURFACE					
0.0	Clayey Silt, trace sand, gravel, shale fragments and rootlets (Fill) Very stiff Red brown Moist	1	SS	15		102
101.8 0.8	Clayey Silt, trace sand, gravel and shale fragments (Flood Plain Deposit) Very stiff Red brown	2	SS	15		
100.9 1.7	Highly to moderately weathered, red brown with occasional grey seams, calcareous SHALE BEDROCK (Queenston Formation). Bedrock cored from 3.3m to 6.4m. For bedrock coring details see Record or Drillhole 2A.	3	SS	100/.23		101
		4	SS	100		100
						99
						98
						97
96.2 6.4	END OF BOREHOLE Note: 1. Open borehole dry upon completion of overburden drilling.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

+ ³, × ³: Numbers refer to Sensitivity ○ ³% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 2A

PROJECT: 001-1158

SHEET 1 OF 1

LOCATION: N 4809228; E 286731

DRILLING DATE: Feb.20/01

DATUM: Geodetic

INCLINATION: -90°   AZIMUTH: ---

DRILL RIG: B-57

DRILLING CONTRACTOR: Master Soil Investigation Ltd.

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous page		99.30												
4	NQ	Highly to moderately weathered, red brown with occasional grey seams, thinly laminated, fine-grained, weak, calcareous SHALE (Queenston Formation).		3.30	1		85						B,SM,PL-C (x3) B,SM,W (x2) B,R,W (x2) B,IR,R (x3) B,R,W B,R,IR B,SM,PL B,SM,PL (x5) BC B,SM,PL (x6)			
5 6		Residual soil zone from 5.3m to 5.5m depth (Residual Soil).			2		80						B,SM,IR (x2) B,SM,PL J,SM,ST BC B,VR,IR B,R,W (x5) B,SM,PL (x2)			
				96.20 6.40												
7–13		END OF BOREHOLE														

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: GM
CHECKED: ASP

RECORD OF BOREHOLE No 3A

METRIC

PROJECT 001-1158			
W.P. 180-00-00	LOCATION N 4809205; E 286731		ORIGINATED BY GM
DIST 4	HWY QEW	BOREHOLE TYPE 114mm Solid Stem Augers	COMPILED BY SEP
DATUM Geodetic	DATE Feb.19/01		CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
102.2	GROUND SURFACE					
0.0	Clayey Silt to Silty Clay, trace sand, gravel and rootlets (Fill) Very stiff to stiff Red brown	1	SS	18		102
101.6 / 0.6	Clayey Silt, trace sand, gravel and shale fragments (Flood Plain Deposit) Stiff Red brown	2	SS	11		101
		3	SS	115/.25		
100.5 / 1.7	Clayey Silt, trace sand (Residual Soil) Hard Red brown					100
100.0 / 2.2	Highly to moderately weathered, red brown with occasional grey seams, calcareous SHALE BEDROCK (Queenston Formation). Smooth grinding from 2.3m to 3.1m depth. Bedrock cored from 3.3m to 6.2m. For bedrock coring details see Record or Drillhole 3A.	4	SS	100/.08		99 98 97
96.0 / 6.2	END OF BOREHOLE Note: 1. Open borehole dry upon completion of overburden drilling.					96

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

+³, ×³: Numbers refer to Sensitivity O ³% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 3A

PROJECT: 001-1158

LOCATION: N 4809205; E 286731

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Feb.19/01

DRILL RIG: B-57

DRILLING CONTRACTOR: Master Soil Investigation Ltd.

SHEET 1 OF 1

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH (COLOUR / % RETURN)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous page	98.90							
4	NQ	Highly to moderately weathered, red brown with occasional grey seams, thinly laminated, fine-grained, weak to very weak, calcareous SHALE (Queenston Formation). Residual soil zone, 50mm thick at 4.2m depth. Extremely weak rock from 4.3m to 4.5m depth.	3.30	1		95	B,SM,PL (x6) B,SM,PL (x8) B,SM,PL (x6)			
5 6	NQ			2		90	B,SM,IR B,SM,PL-W (x4) B,SM,PL-W (x7) B,SM,PL B,SM,C (x2) B,PL,SM (x3) B,SM,W-ST B,SM,PL (x8)			
			96.00 6.20							
7 8 9 10 11 12 13		END OF BOREHOLE								

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: GM

CHECKED: ASP

RECORD OF BOREHOLE No 4A

METRIC

PROJECT 001-1158			ORIGINATED BY GM
W.P. 180-00-00	LOCATION N 4809215; E 286746		COMPILED BY SEP
DIST 4 HWY QEW	BOREHOLE TYPE 114mm Solid Stem Augers		CHECKED BY ASP
DATUM Geodetic	DATE Feb.19/01		

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
101.6	GROUND SURFACE								
0.0	Clayey Silt to Silty Clay, trace sand, gravel, rootlets (Fill) Stiff to very stiff Red brown	1	SS	12					
101.0 / 0.6	Clayey Silt, trace sand, gravel and shale fragments (Flood Plain Deposit) Very stiff Red brown	2	SS	19		8	23	53	16
100.1 / 1.5	Highly to moderately weathered, red brown with occasional grey seams, calcareous SHALE BEDROCK (Queenston Formation).	3	SS	101	▽				
	Becoming wet at 2.1m depth.	4	SS	70/.08					
	Grinding of augers from 2.1m to 2.3m depth and from 2.4m to 3.0m depth.								
	Bedrock cored from 3.0m to 7.6m.								
	For bedrock coring details see Record or Drillhole 4A.								
94.0 / 7.6	END OF BOREHOLE								

Elevation scale: 101, 100, 99, 98, 97, 96, 95, 94

Note:
1. Water level in open borehole at 2.1m depth (Elev. 99.5m) on completion of overburden drilling.

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 4A

PROJECT: 001-1158

LOCATION: N 4809215; E 286746

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Feb.19-20/01

DRILL RIG: B-57

DRILLING CONTRACTOR: Master Soil Investigation Ltd.

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous page		98.60			
3	NQ	Highly to moderately weathered, red brown with occasional grey seams, thinly laminated, fine-grained, weak to very weak, calcareous SHALE (Queenston Formation).		3.00	1	B,SM,PL B,SM,PL-C (x2) J,SM,IR B,SM,PL (x3) B,SM,PL (x4) J,SM,W B,SM,PL (x4) B,SM,PL (x2)	
4–5					2	B,SM,PL-ST (x5) B,SM,PL (x4) B,SM,PL (x2) B,SM,C-FL B,R,UE B,SM,PL (x2)	
6–7					3	B,SM,PL (x2) B,SM,C B,SM,ST B,SM,PL-W B,SM,W-C (x3) B,SM,W (x3) B,SM,W-C (x6) B,SM,ST (x3) B,SM,UE (x3) B,R-SM,IR	
				94.00 7.60			
		END OF BOREHOLE					

Column headings also printed: PENETRATION RATE (m/min), COLOUR % RETURN, FLUSH, RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER 0.3, DIP w.r.t. CORE AXIS, HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}), DIAMETRAL POINT LOAD INDEX (MPa)

Depth scale: 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: GM

CHECKED: ASP

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 5A

METRIC

PROJECT 001-1158
W.P. 180-00-00
DIST 4 HWY QEW
DATUM Geodetic

LOCATION N 4809232; E 286708
BOREHOLE TYPE 114mm Solid Stem Augers
DATE Mar 20/01

ORIGINATED BY GM
COMPILED BY SEP
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.4	GROUND SURFACE									
0.0	Asphalt	1	SS	29		103				
	Crushed Limestone Road base (Fill) Compact Wet									
102.8 / 0.6	Clayey Silt to Silty Clay, trace to some sand, trace gravel and rock fragments (Fill) Stiff Red brown Moist	2	SS	14						
		3	SS	11		102				
		4A	SS	14		101				
100.8 / 2.6	Clayey Silt, trace sand (Residual Soil) Hard Red brown	4B		54						
100.3 / 3.1	Red-brown SHALE bedrock. (Queenston Formation)	5	SS	82/.23		100				
		6	SS	100/.23		99				
98.8 / 4.6	END OF BOREHOLE	7	SS	62/.03						

Note:
1. Water entering borehole through upper road base layer.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity　　$O^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 8

METRIC

PROJECT 001-1158			
W.P. 180-00-00	LOCATION N 4809196; E 286753		ORIGINATED BY GM
DIST 4	HWY QEW	BOREHOLE TYPE 114mm Solid Stem Augers	COMPILED BY SEP
DATUM Geodetic	DATE Feb.19/01		CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
102.6	GROUND SURFACE					
0.0	Silty Clay, trace sand and gravel, trace shale fragments and rootlets (Fill)	1	SS	9		
102.0 / 0.6	Firm to very stiff Red brown Clayey Silt, trace sand and gravel and shale fragments (Flood Plain Deposit)	2	SS	21		102
101.1 / 1.5	Very stiff Red brown Red-brown to grey SHALE bedrock. (Queenston Formation)	3	SS	60/.08		101
					∇	
	Smooth grinding at 3.2m depth.	4	SS	100/.08		100
99.4		5	SS	85/.13		
3.2	END OF BOREHOLE					

Note:
1. Spoon refusal at 1.5m depth.
2. Open borehole dry upon completion of drilling.
3. Water level in open borehole at 2.1m depth (Elev. 100.5m), 24 hours after drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity   ○$^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No C1

METRIC

PROJECT 001-1158

W.P. 180-00-00

DIST 4 HWY QEW

DATUM Geodetic

LOCATION N 4809125; E 286434

BOREHOLE TYPE 114mm Solid Stem Augers

DATE Feb.14/01

ORIGINATED BY GM

COMPILED BY SEP

CHECKED BY ASP

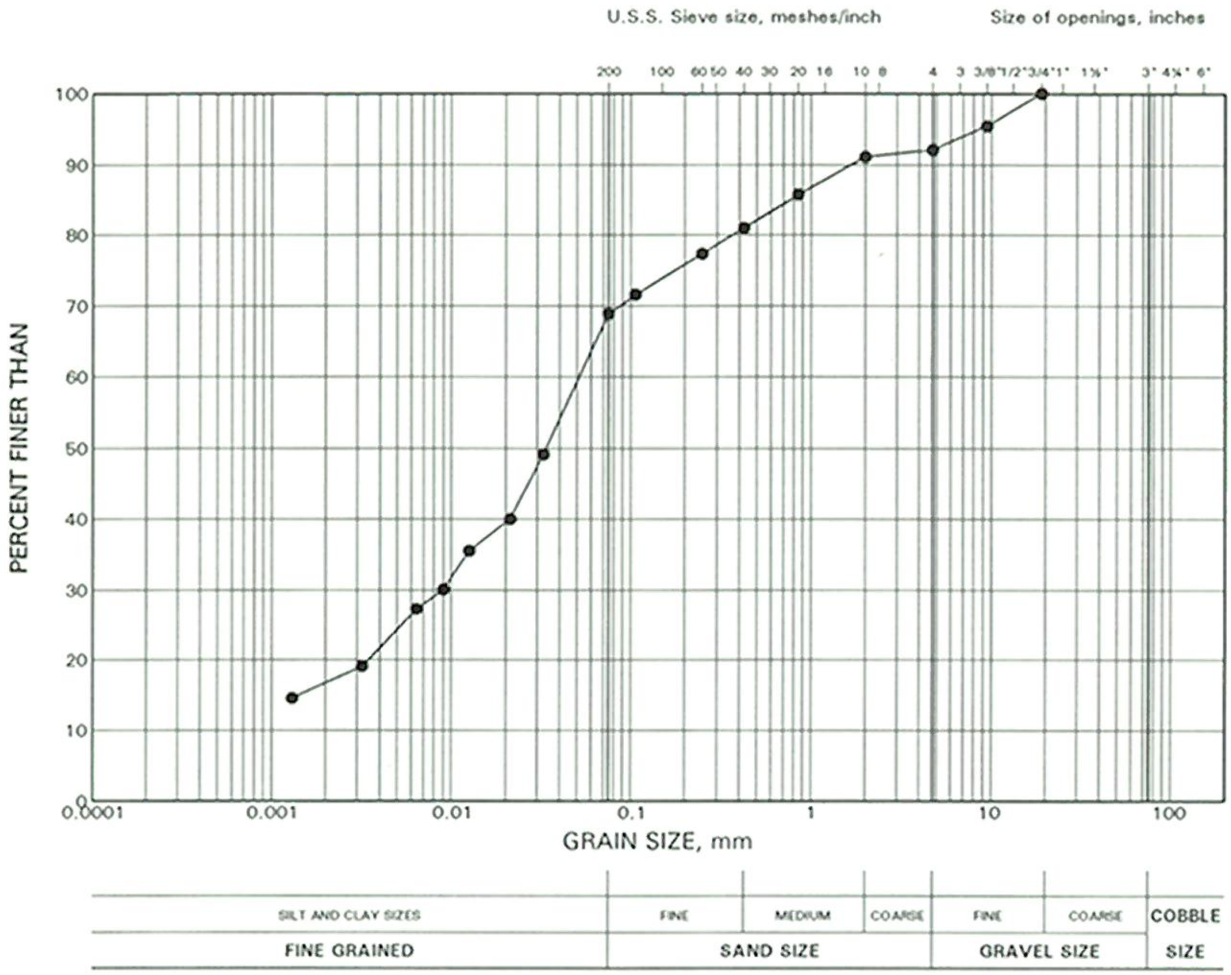
ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.6	GROUND SURFACE										
0.0	Clayey Silt with sand and gravel, trace rock fragments and rootlets (Fill) Very stiff		1	SS	21				○ (approx. 21)		
103.0 / 0.6	Red-brown SHALE bedrock. (Queenston Formation)		2	SS	100/.18		103				
			3	AS							
102.4 / 1.2	END OF BOREHOLE Note: 1. Open borehole dry upon completion of drilling.										

ON_MOT 001-1158.GPJ ON_MOT.GDT 25/9/01

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

GRAIN SIZE DISTRIBUTION
CLAYEY SILT (FLOOD PLAIN DEPOSIT)

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	4A	2	100.4

Golder Associates

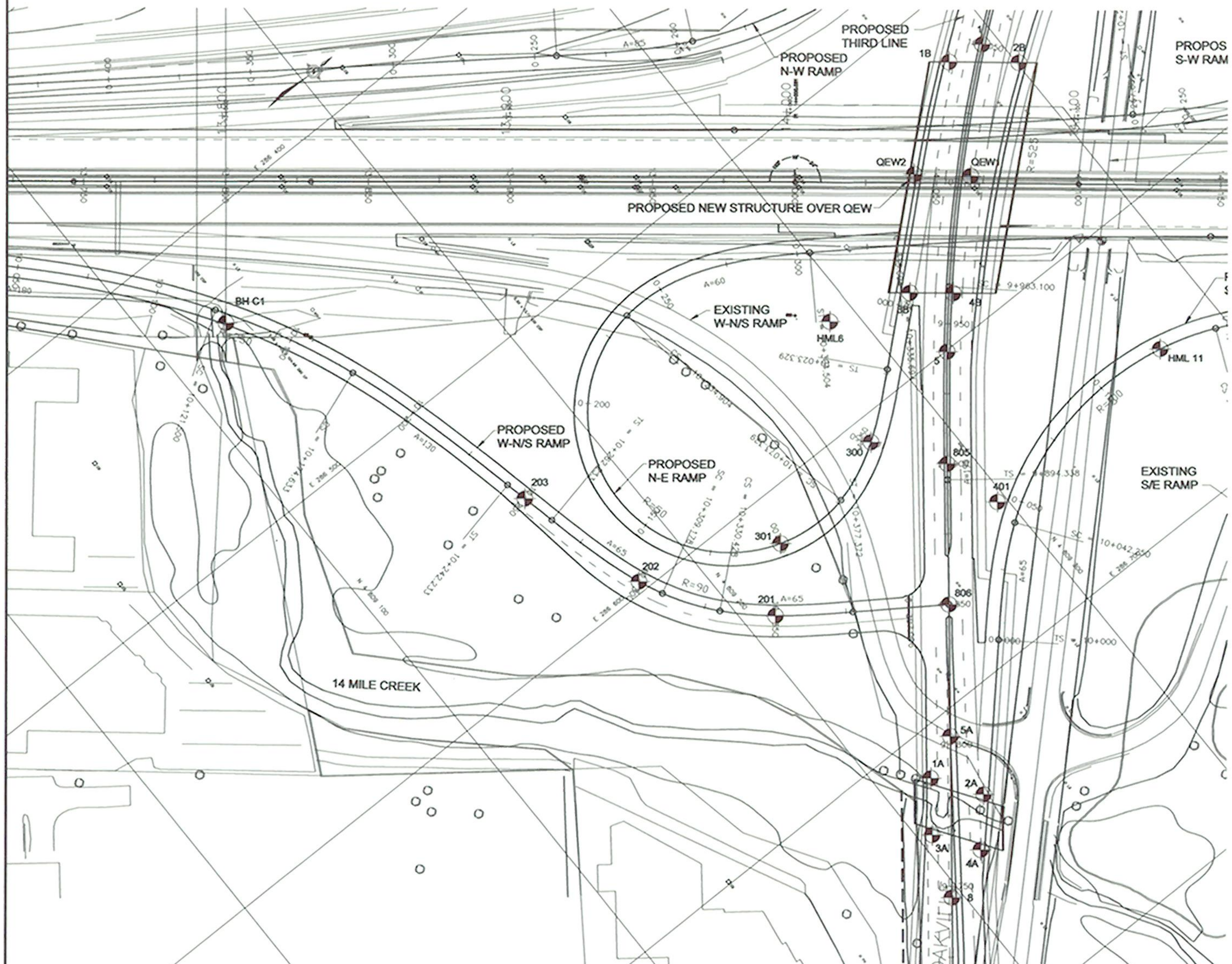
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DIST. HWY. QEW
CONT No.
WP No. 180-00-00

QEW / THIRD LINE INTERCHANGE
BOREHOLE LOCATION PLAN

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

Borehole

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	108.9	4809395	286523
1B	108.9	4809382	286521
1C	116.9	4809637	286316
2B	108.7	4809401	286536
2C	116.8	4809599	286312
3B	108.0	4809319	286576
4B	108.1	4809334	286588
5	107.9	4809316	286600
201	105.9	4809210	286636
202	106.5	4809180	286596
203	107.2	4809168	286548
300	107.0	4809275	286609
301	106.1	4809228	286617
401	107.3	4809297	286653
805	107.4	4809291	286632
806	107.4	4809260	286671
HML6	107.4	4809291	286566
HML 11	108.0	4809376	286647
QEW1	108.5	4809362	286557
QEW2	108.4	4809347	286544

REFERENCE

Drawings provided in digital form from Morrison Hershfield File names "4173-03.DWG, ALIGN.DWG and QEWBASE.DWG"
Drawing recieved MARCH, 2001

0 10 20 30 40 60 80 100 METRES

SCALE

NO.	DATE	BY	REVISION

Geocres No.

HWY. QEW	PROJECT NO. 001-1158-2		DIST.
SUBM'D. SEP	CHKD. ASP	DATE: SEPT. 2001	SITE:
DRAWN: JFC	CHKD. SEP	APPD.	DWG. 1

P1158001.DWG

DIST. HWY. QEW
CONT No.
WP No. 180-00-00

THIRD LINE BRIDGE OVER 14 MILE CREEK

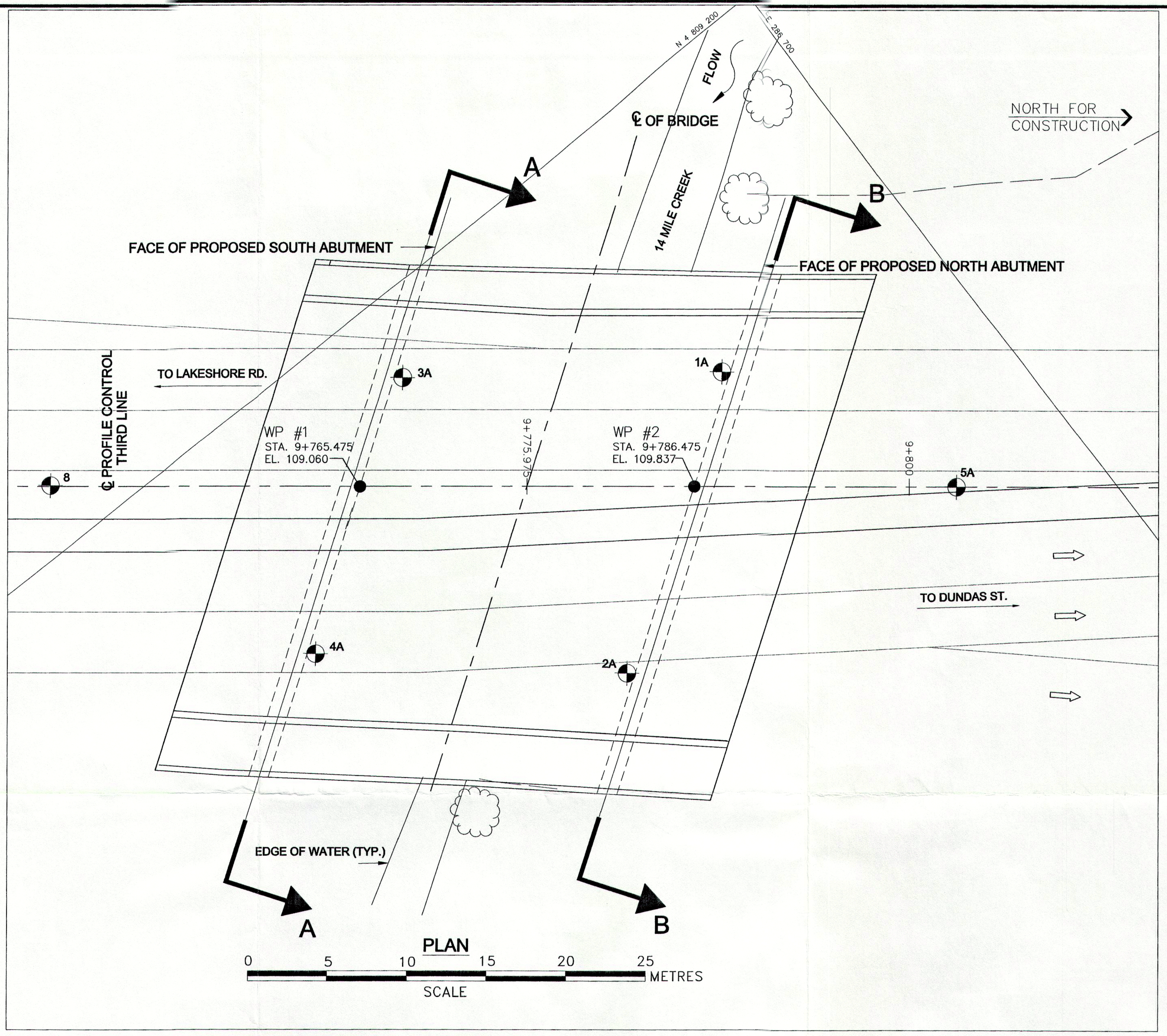
BOREHOLE LOCATION AND SOIL STRATA

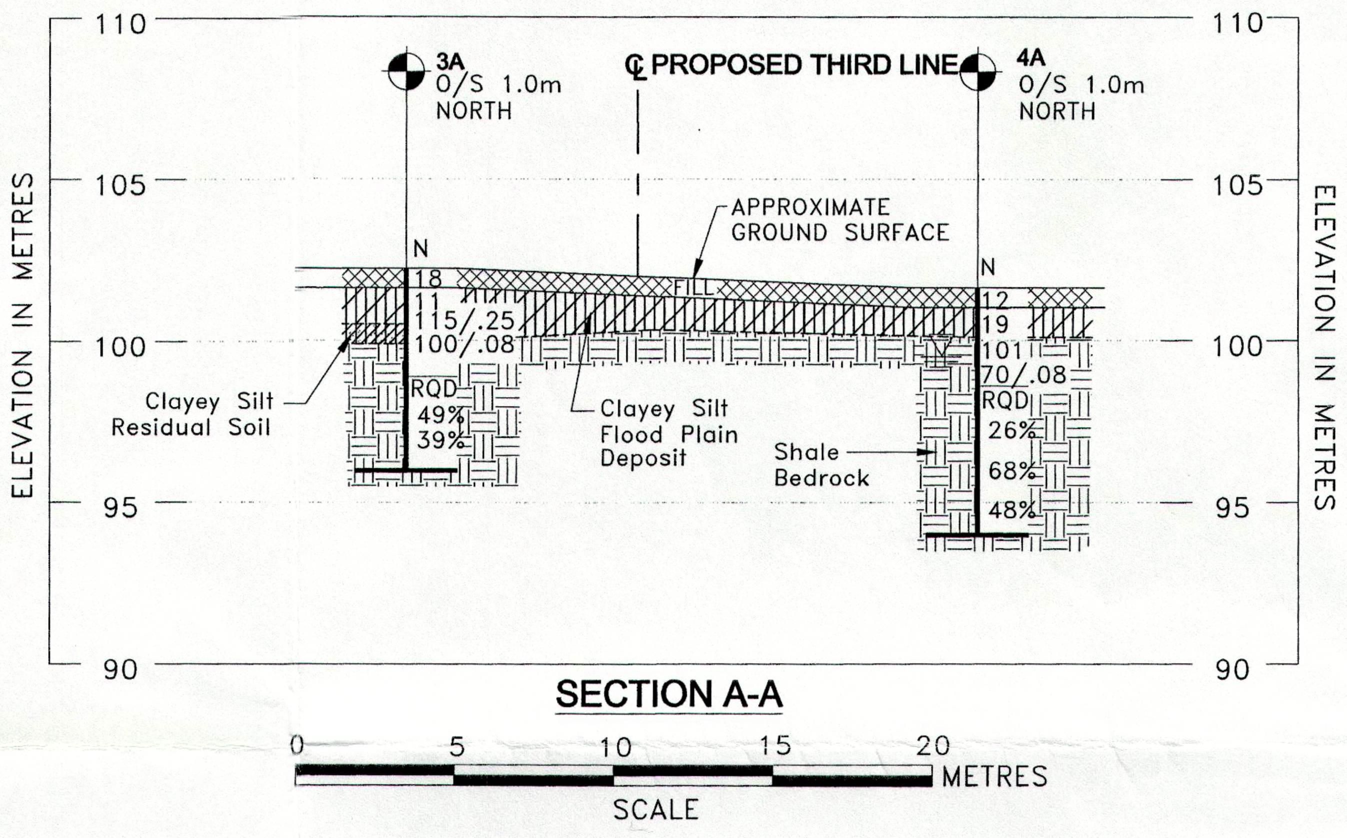
SHEET

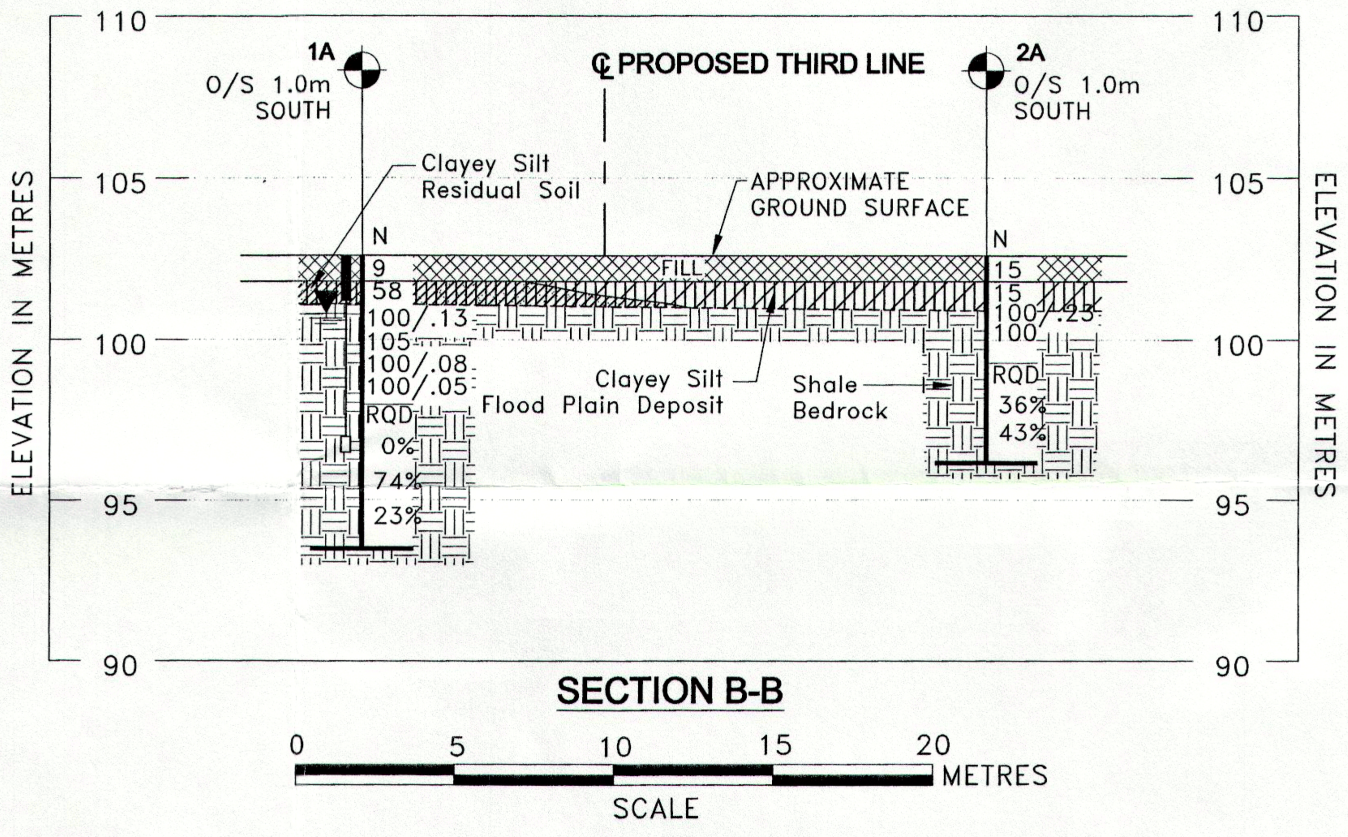
Golder Associates

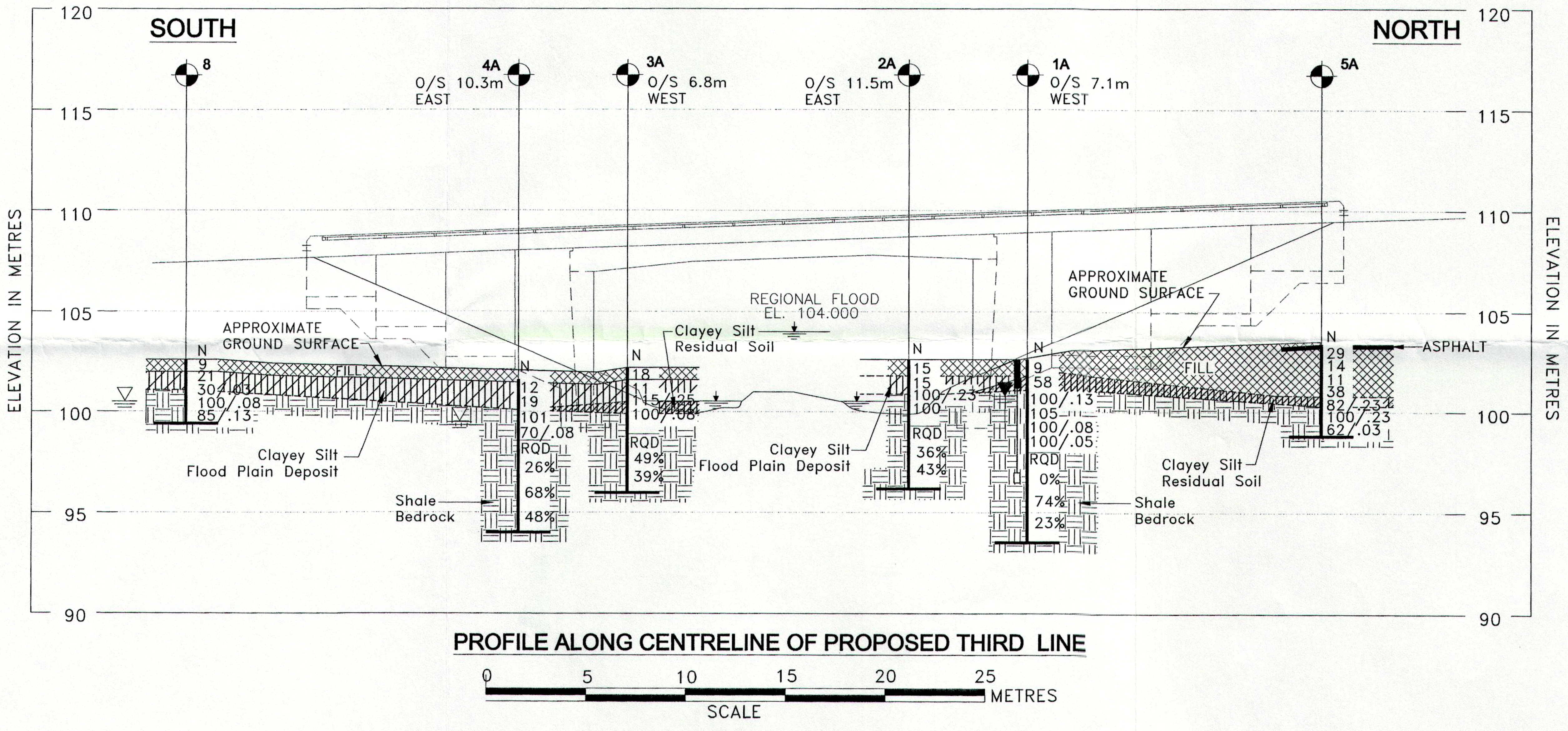
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

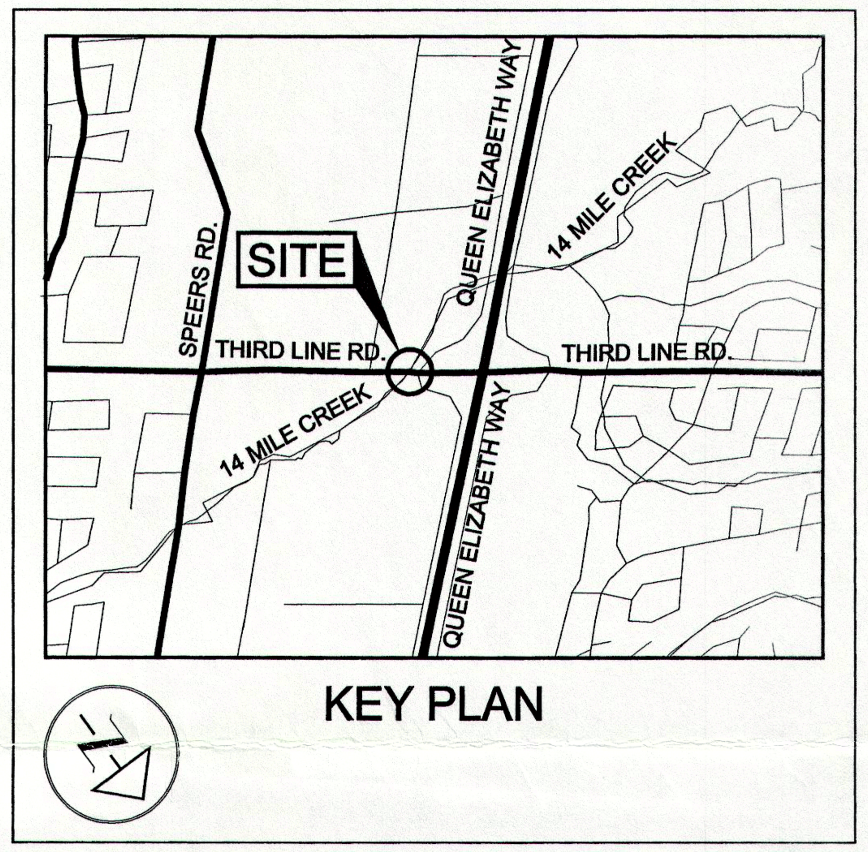
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN











LEGEND

	Borehole
	Seal
	Piezometer
N	Standard Penetration Test value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, April 30, 2001
	WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1A	102.6	4809216.8	286715.2
2A	102.6	4809227.8	286731.4
3A	102.2	4809204.7	286731.2
4A	101.6	4809214.8	286746.1
5A	103.4	4809231.5	286708.0
8	102.6	4809196.3	286752.8

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological interpretation.

The proposed bridge as shown on this drawing is shown only for general reference purposes and may differ from that shown on the structural drawings.

REFERENCE

Digital File prepared by Morrison Hershfield Ltd. File No. 14MOS1.DWG Dated June 7, 2001.

NO.	DATE	BY	REVISION

Geocres No.

HWY. QEW		PROJECT NO. 001-1158.-2	DIST.
SUBM'D. SEP	CHKD. ASP	DATE: SEPT. 2001	SITE:
DRAWN: JFC	CHKD. SEP	APPD.	DWG. 2

P1158002.DWG

1"=1" PS (1:200 MS)