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REPORT ON

**SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED WEST TRANSITWAY EXTENSION
STATION 7+070 TO 7+310 AND THE HWY 417
EAST – NORTH/SOUTH RAMP
AT RICHMOND ROAD
OTTAWA, ONTARIO**

Submitted to:

Harmer Podolak Engineering Consultants Ltd.
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Ottawa, Ontario
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April 2007

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April 30, 2007

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Harmer Podolak Engineering Consultants Ltd.
39 Robertson Road, Suite 221
Ottawa, Ontario
K2H 8R2

Attention: Mr. J. Podolak, P.Eng.

**RE: SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED WEST TRANSITWAY EXTENSION
STATION 7+070 TO 7+310 AND THE HWY 417
EAST – NORTH/SOUTH RAMP AT RICHMOND ROAD
OTTAWA, ONTARIO**

Dear Sir:

Please find attached our report on the supplemental geotechnical investigation for the above section of the proposed West Transitway Extension in Ottawa, Ontario.

We trust that this report provides sufficient information for your present requirements. If you have any questions concerning this report, or if we can be of further service to you on this project, please call us.

Yours truly,

GOLDER ASSOCIATES LTD.

M.I. Cunningham, P.Eng.
Associate

T.J. Nicholas, P.Eng.
Principal

TMS:TJN:MIC:kdc

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1.0 INTRODUCTION

Golder Associates has been retained by Harmer Podolak Engineering Consultants Ltd., consulting engineers to the City of Ottawa, to carry out a supplemental subsurface investigation along a section of the proposed alignment for the extension of the West Transitway. This section of the Transitway extends from Station 7+070 (west limit) easterly for a distance of 240 metres to Station 7+310 (east limit), through which the Transitway will be constructed in 'cut'. This project therefore also includes reconstruction of the Highway 417 East-North-South (E-N/S) ramp at Richmond Road, from about Station 17+135 to Station 17+390. A new structure will be constructed to carry the E-N/S ramp over the Transitway.

Golder Associates carried out a previous geotechnical investigation for this project in 1997/98. That investigation included seven boreholes put down along the Transitway and ramp alignment. Of those, four of the boreholes were put down at the site of the proposed structure for the crossing of the Highway 417 ramp over the Transitway. The remaining three boreholes were put down along the Transitway alignment to the east of that structure but could not generally be put down directly on the alignment since the existing houses restricted the access for drilling equipment; some of the boreholes could only be advanced somewhat to the south of the actual alignment. However, the City of Ottawa has now acquired the full property of the Transitway extension and access along the proposed alignment is now feasible. This report therefore includes the results of additional investigation along the Transitway alignment.

The Transitway will be constructed in 'cut', about 5 to 7 metres below the existing grade, and will pass beneath the existing ramp. It is understood that the design of the retaining walls which will be used to form the future 'cut' will be modified from the previous design concept. The previous design included micro-pile foundations for the retaining walls; that previous design also allowed an open cut temporary side slope adjacent to Highway 417. In contrast, the retaining walls are now to be constructed with conventional H-pile foundations, and temporary shoring during construction will be required to form the excavation adjacent to Highway 417.

Based on the above, the purpose of this supplementary investigation is to determine the general subsurface conditions along the Transitway alignment, east of the proposed ramp structure, by means of six additional boreholes advanced at locations which were previously not accessible and, based on an interpretation of the factual data obtained from both this and the previous investigation, to provide updated guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The results of the previous investigation had been provided in a report to Harmer Podolak Engineering titled "Geotechnical Investigation, Proposed West Transitway Extension, Station 7+070 to 7+310 and East-North/South Ramp at Richmond Road," dated April 1998 (report number 971-2082). However, that previous report is entirely superseded by the current report.

The reader is referred to the 'Important Information and Limitations of this Report' which follows the text but forms an integral part of this document.

2.0 PROJECT DESCRIPTION AND SITE GEOLOGY

The West Transitway extension is to be an arterial corridor for buses between Acres Road and Pinecrest Road in Ottawa, Ontario. The section of the West Transitway considered by this investigation extends from a west limit at Station 7+070 to the east limit at Station 7+310, for a distance of 240 metres, and includes the E-N/S ramp at Richmond Road between Stations 17+135 and 17+390. The relative location of this section of the Transitway corridor is shown on the Key Plan, Figure 1. The proposed Transitway is aligned in an east/west direction and is essentially parallel to Highway 417. From Station 7+070 (west limit), the proposed grade of the Transitway decreases slightly from about elevation 63.8 metres to 63.7 metres at the ramp overpass structure at about Station 7+130 (east limit). From this point eastward, the Transitway grade increases at 2 percent to a proposed elevation of 65.7 metres at Station 7+310. A 675 millimetre diameter storm sewer is proposed to extend under the Transitway.

This section of the Transitway will be primarily constructed in 'cut'. The depth of cut below finished grade increases from about 2 metres at the east limit to about 5.5 metres at the ramp overpass structure and then decreases to about 4 metres at the west limit of the project. The general design concepts being considered consist of cut side slopes, retaining walls or a combination of cut slope and retaining walls. The overpass superstructure will consist of three separate single span rigid frame structures with vertical abutments and wingwalls. From about Stations 7+240 to 7+310, the proposed Transitway encroaches to within about 1 metre of the existing Highway 417 barrier.

The gradeline of the new E-N/S ramp will be generally in 'fill' between about Stations 17+160 and 17+310. The maximum fill is typically less than about one metre above existing grade. During construction of the ramp and structure, a detour ramp will be provided. The design of the detour ramp is being carried out by others.

In keeping with the present Transitway pavement design concept, the Transitway will be a two lane urban cross section roadway with a flexible pavement structure. It is understood that the proposed Transitway pavement is to be designed to carry a projected peak traffic volume of some 790 buses per day (both directions). The projected initial two lane (one way) AADT for the E-N/S ramp is 10,200, of which 60 to 65 percent use the Richmond Road lane. Approximately 7 percent of the E-N/S ramp traffic will consist of commercial/truck traffic and the assumed growth rate is approximately 2 percent.

Topography in and around the proposed West Transitway corridor between Stations 7+070 and 7+310 is relatively flat east-to-west but slopes upward to the north. The Transitway will cut through an existing residential development and the alignment is directly over some existing private residences.

Surficial geology maps indicate that the surficial materials consist of marine deposits of clay and silt overlying glacial till. Bedrock geology maps indicate the site to be underlain by shale bedrock of the Rockcliffe formation.

A preliminary subsurface investigation was carried out by Jacques Whitford and Associates (JWA) for the functional design phase of the West Transitway. The results of that investigation are provided in JWA report number 10494, entitled "Geotechnical Investigation, RMOC West Transitway Extension, Functional Design, Acres Road to Pinecrest Road", dated July 1995. Within the framework of JWA's earlier investigation, no boreholes were advanced along the specific section of the West Transitway extension covered by this report. However, the report included one borehole (89-1) put down by the Ministry of Transportation of Ontario (MTO) which is within the limits of this project. In addition, JWA borehole 94-4 is located at approximately Station 7+340, just to the east of this section of the Transitway. These boreholes indicate variable subsurface conditions. Borehole 89-1 encountered predominantly sandy silt and sand to a depth of about 4.8 metres. This predominantly granular deposit is underlain by shale bedrock with some sandstone lenses. The bedrock was encountered at elevation 63.2 metres. At borehole 94-4, some 10 metres of loose silty sand was encountered overlying about one metre of glacial till. Auger refusal was encountered at about 11 metres depth at elevation 56.7 metres.

3.0 PROCEDURE

The field work for this investigation was carried out between August 18 and 25, 1997 and between August 14 and 17, 2006. In 1997, seven boreholes (numbered 97-1 to 97-7, inclusive) were put down at the approximate locations shown on the Site Plan, Figure 2. The boreholes were advanced using a track mounted, hollow stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths ranging from approximately 3.7 to 8.2 metres below ground surface. Upon encountering practical refusal to augering, boreholes 97-4 to 97-7, inclusive, were extended into the bedrock using rotary diamond drilling equipment while retrieving NQ or HQ sized bedrock core. The boreholes were terminated at depths ranging from approximately 6.0 to 8.9 metres below the ground surface.

In 2006, six boreholes (numbered 06-101 to 06-106, inclusive) were put down at the approximate locations shown on the Site Plan, Figure 2 and generally within the properties acquired by the City along the Transitway alignment. The boreholes were also advanced using a track mounted, hollow stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. These boreholes were all advanced to practical refusal to augering which varied in depth from approximately 6.3 to 12.4 metres below the existing ground surface. All of the boreholes were then extended into the bedrock using rotary diamond drilling equipment while retrieving NQ sized bedrock core. The boreholes were terminated at depths ranging from approximately 9.7 to 15.9 metres below the existing ground surface.

Standpipes or 38 millimetre diameter PVC monitoring wells with #10 screens were sealed into boreholes 97-1 to 97-7 (inclusive), 06-101, 06-103, and 06-106 to allow subsequent measurement of the groundwater levels and to permit in-situ rising head permeability testing.

The field work was supervised by an experienced technician from our staff who located the boreholes, directed the drilling operations, logged the boreholes and samples, directed the in-situ testing, and took custody of the soil and bedrock samples retrieved.

On completion of the drilling operations, samples of the soils and bedrock encountered in the boreholes were transported to our laboratory for examination by the project engineer and for laboratory testing. The laboratory testing included grain size distribution analysis, natural water content determinations, and Atterberg limit testing.

In boreholes 97-1, 97-3, 97-6, and 97-7, the groundwater levels were measured and in-situ rising head permeability testing was carried out on September 4, 1997. The groundwater levels in the standpipes in boreholes 97-2, 97-4, and 97-5 were measured on September 19, 1997. The groundwater levels in the standpipes in boreholes 06-101, 06-103, and 06-106 were measured on August 18, 2006. In-situ rising head permeability testing was carried out between September 30 and October 1, 1997.

Samples of water from boreholes 97-1, 97-3, and 97-6 were submitted to Accutest Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The borehole locations were selected by Harmer Podolak. The approximate locations of the boreholes are shown on the Site Plan, Figure 2. The locations and elevations of boreholes 97-1 to 97-7, inclusive, were determined by Harmer Podolak and are understood to be referenced to Geodetic datum. For boreholes 06-101 to 06-106, the locations and elevations were determined by Golder Associates. The elevations were referenced to the cover of a sanitary sewer located at the intersection of Pulford Crescent and Burgess Avenue. It is understood that the elevation of this cover is 69.05 metres (Geodetic datum).

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes put down by Golder Associates Ltd., for both the current and previous investigations, are shown on the Record of Borehole sheets in Appendix A. The subsurface conditions encountered in two boreholes put down on the site from previous investigations by others are shown on the Record of Borehole Sheets in Appendix B.

The results of the laboratory water content and Atterberg limit testing on the selected soil samples are given on the Record of Borehole sheets. The results of grain size distribution testing carried out on selected soil samples are provided on Figures 3 to 7. The results of the basic chemical analysis carried out on samples of groundwater from boreholes 97-1, 97-3, and 97-6 are provided in Appendix C.

In general, the subsurface conditions on this site consist of a thin surficial layer of fill material overlying layered sandy silt, silty clay, and clayey silt, which is underlain by sandy soils (along the eastern portion of the site) and glacial till. The surface of sandstone bedrock exists at about 4 metres depth at the western project limit (Station 7+070) and deepens to about 12.5 metres depth at the western project limit (Station 7+310).

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes.

4.2 Overburden

4.2.1 Topsoil and Fill Materials

Fill material exists at ground surface at all of the borehole locations. The overall thickness of the fill material ranges from 0.5 to 2.3 metres. Surficial topsoil fill forms the upper part of the fill material at ground surface at boreholes 97-1, 97-3, 97-6, 97-7, 06-101, 06-102, 06-103, and 06-106. The surficial topsoil fill varies in thickness from about 80 to 300 millimetres and averages approximately 155 millimetres. Otherwise, the fill material generally consists of sand with variable amounts of silt, sand and gravel, cobbles, clayey silt, silty clay, gravel, concrete, and crushed stone.

The fill materials are typically thicker at those boreholes put down within the existing Highway 417 and E-N/S ramp right-of-way; the fill material is up to about 2.3 metres thick. The fill materials in boreholes 6-101 to 06-106, put down in the yard areas of the houses on Burgess Avenue are typically no more than about 1.1 metres thick.

The results of the grain size distribution testing on one sample of the fill material are shown on Figure 3. The results indicate that the tested sample consists of silty sand and gravel. The measured moisture content of the fill material varied from 8 to 12 percent (where tested).

Standard penetration test N values in the fill generally ranging from 3 to 14 blows per 0.3 metres of penetration indicate a loose to compact state of packing. Higher N values of up to 59 blows were recorded, but could reflect cobbles and gravel within the fill material.

A buried layer of topsoil (i.e., the native topsoil layer) underlies the fill material in boreholes 06-101 through 06-105 and is approximately 150 to 310 millimetres thick.

4.2.2 Existing Ramp Shoulder Pavement Structure

Boreholes 97-2, 97-4, and 97-5 were advanced through the shoulders of the existing 417 westbound to Richmond Road ramp. The existing pavement structure of the ramp, based on these three boreholes, consists of the following:

- 90 to 150 millimetre asphaltic concrete
- 400 to 520 millimetres sand and gravel base
- 970 to 1680 millimetres sand or silty sand subbase

Standard penetration test N values obtained within the subbase material ranging from 5 to 14 blows per 0.3 metres of penetration indicate a loose to compact state of packing.

4.2.3 Layered Sandy Silt, Silty Sand, and Clayey Silt

The topsoil and fill materials in most of the boreholes are underlain by a deposit of layered silty sand, sandy silt and clayey silt. This deposit extends to depths ranging from about 3.1 to 9.5 metres.

The deposit is, however, absent at boreholes 97-3 and 97-5 through 97-7, where the glacial till underlies the surficial fill materials directly. All of these boreholes, including borehole 97-4 in which the deposit is only about 0.8 metres thick, are located at the west end of the alignment, near the ramp structure. The deposit thickens to the east.

The results of grain size distribution testing on selected samples of this deposit are provided on Figure 4.

Standard penetration test N values of 'weight hammer' to 16 blows per 0.3 metres of penetration in the sandier portions of this deposit indicate a very loose to compact state of packing.

Boreholes 06-101 to 06-106 (inclusive) encountered more distinct clayey silt layers in the deposit, interlayered with sandy silt. The upper portion of these deposits has been weathered to a stiff grey brown crust. The weathered zone varies from approximately 0.9 to 1.5 metres in thickness and extends to approximately 2.8 to 3.4 metres depth below the existing ground surface (i.e., extending to elevation 65.5 to 66.2 metres). Standard penetration tests carried out within the weathered clayey silt gave N values ranging from 2 to 4 blows per 0.3 metres of penetration indicating a stiff to very stiff consistency.

Standard penetration tests carried out within the lower unweathered portions of the clayey silt gave N values ranging from 'weight of hammer' to 1 blow per 0.3 metres of penetration. The results of in-situ vane testing in this material gave undrained shear strengths ranging from 48 to 92 kilopascals indicating a firm to stiff consistency.

The measured water content of the layered clayey silt and sandy silt ranges from approximately 23 to 48 percent.

4.2.4 Sand

The layered sandy silt deposit in borehole 97-2 is underlain by a deposit of sand. The results of grain size distribution testing carried out on one sample from this deposit are provided on Figure 5. This deep sand deposit was not fully penetrated in borehole 97-2 but was proven to extend to at least 8.2 metres depth below the ground surface (i.e., extending to about elevation 60.0 metres) prior to the borehole being terminated within this deposit.

Standard penetration test N values ranging from 4 to 7 blows per 0.3 metres of penetration indicates a very loose to loose state of packing.

The measured water content of the deep sand deposit ranges from approximately 17 to 19 percent.

4.2.5 Glacial Till

Glacial till underlies the surficial fill materials and/or the layered clayey silt, sandy silt and silty sand in all boreholes with the exception of borehole 97-2 which did not penetrate the overlying sand. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand and sandy silt with a trace to some clay. The glacial till was fully penetrated within boreholes 97-4 to 97-7 (inclusive) and boreholes 06-101 and 06-106 (inclusive) and varies from approximately 0.8 to 5.4 metres in thickness (i.e., extending to the bedrock surface at elevations 57.0 to 64.4 metres).

Standard penetration test 'N' values obtained within the glacial till ranging from 3 to greater than 50 blows per 0.3 metres of penetration indicate a very loose to very dense state of packing. The

higher N values may, however, reflect the presence of cobbles and boulders within the glacial till rather than the state of packing of the soil matrix.

The measured water content of the glacial till deposit ranges from approximately 7 to 19 percent.

The results of grain size distribution testing carried out on selected samples of the glacial till are provided on Figure 6.

In-situ rising head permeability testing was carried out in boreholes 97-1, 97-3, 97-6 and 97-7. These tests indicate a hydraulic conductivity ranging from approximately 9×10^{-6} to 8×10^{-5} centimetres per second.

4.3 Auger Refusal and Bedrock

Practical refusal to augering was encountered in borehole 97-3 at a depth of about 7 metres below the ground surface (i.e., at about elevation 61.9 and 56.7 metres, respectively). Auger refusal could potentially indicate the bedrock surface or could also be due to the presence of boulders in the glacial till.

Bedrock was encountered beneath the glacial till at all of the other borehole locations, with the exception of boreholes 97-1, 97-2, and 97-3 which were terminated in the overburden, at depths ranging from about 3.7 metres (i.e., about elevation 64.4 metres) at the western limit of the project to about 12.4 metres (i.e., about elevation 57.0 metres) at the eastern limit of the project. The boreholes were extended into the bedrock for depths ranging from about 2.0 to 3.5 metres using rotary diamond drilling techniques while obtaining NQ or HQ sized bedrock core samples. Based on the core samples retrieved, the bedrock consists of slightly weathered to fresh light grey fine to medium grained sandstone with numerous grey green thinly laminated to very thinly bedded shale interbeds. An approximate 70 millimetre thick mud seam was encountered at about elevation 62 and 61 metres at boreholes 97-4 and 97-5 respectively.

The Total Core Recovery (TCR) varied between 70 and 100 percent, generally increasing with depth, indicating possible soil infilled seams and weathered beds within the upper portion of the bedrock. The Solid Core Recovery (SCR), the percentage of core that is completely circular in section, ranges from 20 percent to 100 percent, generally increasing with depth, indicating that there exists some inclined fracturing of the bedrock. The Rock Quality Designation (RQD), the length of intact core longer than 100 millimetres in length, varies between 35 and 100 percent, indicating variable quality bedrock.

4.4 Groundwater

The groundwater levels in the standpipes in boreholes 97-1, 97-3, 97-6, and 97-7 were measured on September 4, 1997. The groundwater levels in the standpipes in boreholes 97-2, 97-4, and 97-5 were measured on September 19, 1997. The groundwater levels in the standpipes in boreholes 06-101, 06-103, and 06-106 were measured on August 18, 2006.

The groundwater levels obtained within boreholes put down during the 1997 investigation varied between 1.9 and 3.5 metres below the existing ground surface (i.e., between elevations 65.1 and 66.4 metres). The standpipe in Borehole 97- 5 was dry at the time the measurements were taken.

The groundwater levels obtained within the boreholes put down during the current (i.e., 2006) investigation varied from about 2.3 and 4.6 metres below the existing ground surface (i.e., between elevations 63.0 and 66.6 metres).

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

5.0 DESIGN CONSIDERATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of this project based on our interpretation of the borehole information and project requirements, and subject to the limitations in the “Important Information and Limitations of This Report” attachment which follows the text of this report.

5.2 Excavations

In general, the subsurface conditions on this site consist of a surficial layer of fill material overlying layered sandy silt, silty clay, and clayey silt, which is underlain by sandy soils (along the eastern portion of the site) and glacial till. The surface of sandstone bedrock exists at about 4 metres depth at the western project limit (Station 7+070 metres) and deepens to about 12.5 metres depth at the eastern project limit (Station 7+310 metres).

It is understood that the proposed Transitway gradeline will be at about elevation 63.7 metres at Station 7+070 and increase to about elevation 65.7 metres at Station 7+310. It is anticipated that the excavations would extend about 1 to 2 metres below the Transitway gradeline to allow for placement of the storm sewer, the Transitway pavement structure, and the foundations for the adjacent retaining walls and structure.

Based on the available subsurface information, excavation for the Transitway along the western portion of the site (about from Station 7+070 to 7+150, which includes the ramp structure) will be through topsoil, fill material, layered silt and sand, glacial till, and into the underlying sandstone bedrock. East of about Station 7+150, the excavations are expected to terminate within the overburden materials and not extend into the bedrock.

No unusual problems are anticipated in trenching through the overburden using conventional hydraulic excavating equipment, although boulder removal (from within the glacial till deposit) should be expected. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes.

The Occupational Health and Safety Act (OHSA) of Ontario indicates that short term excavation side slopes in the overburden should be sloped at a minimum of 1 horizontal to 1 vertical (i.e. Type 3 soils). Where sandy material is encountered, and unless the water level is lowered in advance of construction, the side slopes would have to be flatter to prevent sloughing; side slopes of 3 horizontal to 1 vertical may be required in this material (i.e. Type 4 soils). These sandy materials appear to be more prevalent along the eastern part of the alignment. A steeper side slope, which is anticipated along the south side of the excavation adjacent to Highway 417 due to space restrictions, would require shoring to meet the requirements of the OHSA. Along the north

side of the excavation, there may not be sufficient space to accommodate a 3 horizontal to 1 vertical side slope. It should therefore be planned to lower the groundwater level in the overburden in advance of the excavation. Temporary construction shoring could be considered for that slope.

In general, there are three basic shoring methods that are commonly used in local practice: steel soldier piles and lagging, driven steel sheet piles and, less commonly, continuous concrete (secant pile or diaphragm) walls, each with appropriate lateral support. It is understood that steel soldier piles and lagging is planned for this site. Additional guidelines on temporary shoring are provided in Section 5.3 of this report.

Along the central portion of the alignment, where the excavation will be entirely within the glacial till and layered silt and sand, but with the surface of the underlying bedrock at only shallow depth below the excavation floor, there would be an inadequate factor of safety against basal heaving of the trench floor; basal heaving would occur where the weight of the soil cover is less than the piezometric pressure in the underlying bedrock. Such basal heaving could result in instability of the excavation side slopes and disturbance of the subgrade. There could also be a loss of lateral toe restraint for the shoring. It is therefore suggested that the groundwater level in the bedrock be lowered in advance of excavation. The excavation may be preceded by pumping from sumps excavated into bedrock, or wells installed in the bedrock, outside the excavation footprint and taken to at least 1 metre below the lowest excavation level. Lowering of the groundwater level in the bedrock may also be at least partially effective at lowering the groundwater level in the sandy overburden deposits. However the relatively lower hydraulic conductivity of the glacial till layer may limit this effect. Therefore, some active groundwater level lowering in the overburden, such as by wells installed in the sandy deposits, should also be planned.

It is expected that the rate of groundwater pumping/inflow will exceed 50,000 litres per day. A Permit to Take Water from the Provincial Ministry of the Environment will be required. The contractor should prepare and submit a groundwater management plan for review and approval. That plan should be prepared by a specialist in this field.

It is also expected that the bedrock removal for this project will be carried out using drill and blast techniques. Mechanical methods of rock removal (such as hoe ramming), though technically feasible, would likely be slow and tedious. The sandstone bedrock is hard and abrasive. Greater than typical equipment wear should be expected.

Based on the quality of the bedrock encountered in the boreholes, it is expected that near vertical bedrock walls can generally be provided for the construction period. Blast induced damage to the bedrock must be avoided; otherwise some rock reinforcement could be required. It should therefore be planned to either line drill the bedrock along the perimeter of the excavation at a close spacing in advance of blasting so that a clean bedrock face is formed or perimeter drill or

pre-shear the excavation limits using controlled blasting. It is considered that 75 millimetre diameter holes drilled at a spacing of 200 millimetres would be appropriate for line drilling without pre-shearing.

The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field.

A pre-blast survey should be carried out of all of the surrounding structures. Selected existing interior and exterior cracks in the structures should be identified during the pre-blast survey and should be monitored for lateral or shear movements, such as by means of telltales.

The contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

The contractor should be limited to only small controlled shots. The following frequency dependent peak vibration limits at the nearest structures and services are suggested.

Frequency Range (Hz)	Vibration Limits (millimetres/second)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting operations be carried out both in the ground adjacent to the closest structures and within the structures themselves.

As an alternative to blasting, where excavation depths are less than about one metre, bedrock removal could be carried out with large hydraulic hoe ram excavation equipment, which would be more productive if used in conjunction with line drilling on 150 millimetre centres or saw cutting the rock in advance of excavation.

Where temporary slopes excavated in overburden will exist above the bedrock excavation, the toe of the overburden slope should be set back at least 1.0 metre from the crest of any subsequent rock excavation. This set back should provide sufficient space for the rock excavation and will reduce the potential for undercutting of the overburden slope during the rock removal.

The construction of the Transitway will lower the groundwater level a maximum of about 3 metres. The planned temporary and permanent groundwater level lowering would be an issue with regards to surrounding ground settlements if sensitive and compressible clay soils exist within the expected zone of influence of the groundwater level lowering. However the results of this investigation as well as published geologic mapping do not indicate such soils as being present. The sandy soils and glacial till that are indicated to be present are not considered to be sensitive to groundwater level lowering.

5.3 Excavation Shoring

5.3.1 General Shoring Options

The shoring method(s) being considered to support the excavation sides must take account the soil stratigraphy, the groundwater conditions, the methods adopted to manage the groundwater, the ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities. In general, as mentioned previously, there are three basic shoring methods that are commonly used in this area: steel soldier piles and timber lagging, driven steel sheet piles and, less commonly, continuous concrete (secant pile or diaphragm) walls, each with appropriate lateral support. The first two types are more generally suitable where the objective is to maintain an essentially vertical excavation wall and the movements above and behind the wall need only be sufficiently limited that relatively flexible features (such as roadways) will not be adversely affected. Where foundations lie within the zone of influence of the shoring, the lateral deflections of the wall need to be greatly limited and continuous concrete shoring or potentially interlocked steel sheet piling with pre-stressed tie backs are required. Continuous concrete shoring systems could include diaphragm walls constructed using slurry excavating techniques (which could also serve as the permanent structure wall) or a series of interlocked cast-in-place concrete caissons (i.e., secant piles). Continuous concrete construction is particularly suitable where significant groundwater inflow is expected and the groundwater levels outside of the excavation need to be maintained. For all of the above systems, some form of lateral support to the wall is required for excavation depths greater than about 4 metres. Lateral restraint could be provided by means of tie-backs consisting of either soil anchors or grouted bedrock anchors.

For this site, it is considered that tight restrictions on the settlement of the ground above and behind the shoring are not required since the adjacent structures are set relatively far back from the excavation. The use of steel soldier piles and lagging would therefore be appropriate.

The contractor should be responsible for the detailed design of the shoring. However this section of the report provides some general guidelines on possible concepts for the shoring, to be used by the designers for assessing the costs and possible impacts of the shoring design and construction on the design of the excavation and site works as well as to evaluate, at the design stage, the potential for impacts of this shoring on the adjacent properties.

5.3.2 Lateral Earth Pressures

The shoring should be designed to account for lateral earth pressures resulting from the weight of the retained earth and other dead and surcharge loads.

The earth pressure distribution used for shoring design is dependent upon the specific wall design and on the nature of the lateral support provided.

Where a soldier pile and lagging wall would be supported by internal struts (including rakers), the walls should be designed to resist a rectangular earth pressure distribution having a constant magnitude with depth of:

$$\sigma_h = 0.65 K_a (\gamma H + q)$$

Where:

- σ_h = Lateral earth pressure on the shoring, kilopascals
- K_a = active earth pressure coefficient, use 0.33
- γ = unit weight of retained soil, use 22 kilonewtons per cubic metre
- H = height of shoring (i.e., depth of excavation), metres
- q = surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 15 kilopascals).

Tied-back soldier pile and lagging walls should be designed to resist an earth pressure distribution having a (variable) magnitude with depth equal to:

$$\sigma_h(z) = K_a (\gamma z + q)$$

Where:

- $\sigma_h(z)$ = Lateral earth pressure on the shoring at depth 'z', kilopascals
- K_a = active earth pressure coefficient, use 0.33
- γ = unit weight of retained soil, use 22 kilonewtons per cubic metre
- z = depth below top of shoring, metres
- q = surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 15 kilopascals).

This earth pressure distribution is applicable over the full height of the wall, down to the wall toe.

It is expected that tie-backs will be the more practical option for this site, since struts or rakers would interfere with the construction of the retaining walls.

The above lateral earth pressures have not been factored; factoring of these loads will be required if the shoring is being designed in accordance with Limit States Design.

The lateral restraint to the embedded toe of the soldier piles will depend on the material into which it is embedded, and how successfully the material was dewatered. Where there is continuing groundwater seepage from the overburden, the overburden may soften (particularly the sandy soils) and offer little lateral resistance. As a preliminary guideline, the lateral resistance from the overburden could be conservatively neglected.

Wherever the excavation will extend into the bedrock below the shoring toe level, it should be planned to secure the toe of the soldier piles to the bedrock by means of grouted anchors or pins into the bedrock.

5.3.3 Resistance to Lateral Earth Pressures

It is expected that lateral restraint of the temporary shoring will be provided by means of tie-backs anchored into either the soil (particularly on the eastern part of the site where the bedrock is deeper) or bedrock (on the western part of the site where bedrock is shallow).

Rock Anchors

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

- i) failure of the steel tendon or top anchorage
- ii) failure of the grout/tendon bond
- iii) failure of the rock/grout bond
- iv) failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion. The results of chemical analysis on the groundwater indicate an aggressive environment with regard to corrosion (see Section 5.8)

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ULS design purposes. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an

apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

where:

Q_r	=	factored uplift resistance of the anchor, kilonewtons
ϕ	=	resistance factor, see below
γ'	=	effective unit weight of rock, use 17 kilonewtons per cubic metre
D	=	anchor length in metres
θ	=	½ of the apex angle of the rock failure cone, use 30 degrees

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r^* = Q_r \cos(\alpha)$$

where:

Q_r^*	=	factored uplift resistance of the anchor subject to inclined load in kilonewtons
Q_r	=	factored uplift resistance of the anchor, kilonewtons
α	=	angle between the load direction and the vertical

Table 6.6.2.1 of the 2000 Canadian Highway Bridge Design Code (CHBDC) suggests that design be based on the following geotechnical resistance factors:

Ground Anchors (Soil or Rock)

Static Analysis (Tension)	0.4
Static Test (Tension)	0.6

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors.

It is suggested that proof-load tests be carried out on the anchors. The proof load tests should be carried out to 1.3 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

It is suggested that the installation and testing of the anchors be supervised by the geotechnical engineer. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to ensure an adequate bond between the grout and the rock.

Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

Soil Anchors

If soil anchors are used for the shoring within the eastern portion of the alignment, the bond zone will be formed within the layered silt and sand and/or the glacial till. These deposits, however, are saturated (i.e., water bearing) and are considered to have a moderate hydraulic conductivity. Water pressure fluctuations within this soil during drilling or anchor installation could result in disturbance of this soil type. Given these soil conditions, the following two options are considered suitable for the anchors:

- Steel tendons or bars grouted in-place and subsequently secondary grouted.
- Helical anchors

The anchor holes for drilled anchors will need to be fully cased during installation. Installation with continuous-flight-auger equipment would be preferred.

For the grouted strand or bar, the secondary grouting increases the apparent bond stress and also allows individual anchors to be re-grouted to improve the bond stress if proof-testing during construction demonstrates a particular anchor to be inadequate. To be effective in increasing anchor capacity, the grout pressure must be sufficient to fracture the primary grout and the surrounding soil mass.

The design of the soil anchors should be the responsibility of the contractor who will install the shoring, however for the purposes of developing shoring concepts based on soil anchored supports and for costing such concepts, the following guidelines are provided for estimating the anchor capacity. The anchors may be installed into layered silt and sand or glacial till. However, given the variability of the stratigraphy, it is difficult to predict exactly which formation into which the anchors would be installed and it should therefore be conservatively assumed that they will be installed into the layered silts and sand. Alternatively, it is possible that two designs could be prepared and the anchors constructed based on the soil type encountered during installation. The capacity of the anchors also depends on the effective stress level at the depth of installation. Therefore, the Ultimate Limit States (ULS) factored geotechnical resistance can be estimated as being:

$$R_f = \phi \pi D L \sigma_v' K_f$$

Where:

- R_f = factored resistance
- ϕ = resistance factor, see below
- D = diameter of hole (πD = circumference of drilled hole)
- L = length of anchored zone
- σ_v' = effective stress level at depth of anchor
 - = γH above the water table
 - = $\gamma H_w + (\gamma - \gamma_w)(H - H_w)$ below the water table
- γ = unit weight of overlying soil, use 20 kilonewtons per cubic metre
- γ_w = unit weight of water, 9.8 kilonewtons per cubic metre
- H = depth to anchored zone below ground surface, metres
- H_w = depth to groundwater level below ground surface, metres
- K_f = anchorage coefficient, use 0.2 for silts and sands

Table 6.6.2.1 of the 2000 Canadian Highway Bridge Design Code (CHBDC) suggests that design be based on the following geotechnical resistance factors:

Ground Anchors (Soil or Rock)

Static Analysis (Tension)	0.4
Static Test (Tension)	0.6

The length of the tie-backs and the placement of the anchored zone should be assessed based on the stability of the retained soil mass, as outlined in Section 27.1.3.5 of the Canadian Foundation Engineering Manual (CFEM). However, as a minimum guideline, all anchors should be installed with their free length extending *beyond* a line projected upward at 60 degrees to the horizontal from the base of the shoring by an amount equal to 15 percent of the wall height, as indicated on Figure 27.6 of the CFEM.

Helical anchors may be economic since an open hole is not required. These anchors are also probably more easily removed following construction, should this be a requirement. However, the capacity of the anchors is less than that achievable for grouted earth anchors. One form of these anchors is that manufactured by A.B. Chance and distributed in Ontario by E.B.S. Construction and Engineering. For preliminary design purposes, helical anchors may be designed using a factored ULS geotechnical resistance of about 40 to 100 kilonewtons depending on the depth of installation, number and size of helixes, and the capacity of the rod. However, the actual capacity should be determined by the supplier and installer.

Because the anchor capacity (in particular the ground-to-anchor bond strength for grouted soil anchors) is highly dependent upon the installation technique, the complete temporary anchor design should be the responsibility of the Contractor, who should be held to an anchor

performance specification, enforced by proof tests on all anchors. Requirements for temporary anchor design and performance include the following:

- The sustained working load is not to be greater than 60 percent of the guaranteed ultimate tensile strength of the anchor tendon or bar
- Ten per cent of all anchors are to be subjected to a performance test in which they are stressed to a sustained load equal to 1.33 times the design (working stress).
- All anchors are to be proof-loaded to 1.33 times the design load and locked off at 1.1 times the design (working stress) load.

Vertical and lateral monitoring of the shoring wall should be carried out on a daily basis using precise methods.

5.3.4 Ground Movements

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground will occur as a result of excavation, installation of shoring, and deflection of the ground support system, including bending of the walls, compression of the struts and/or extension of the tie-backs, the quality of the work, as well as deformation of the soil in which the toes of the walls are embedded. The ground movements induced could affect the performance of surface structures or underground utilities adjacent to the excavation. Therefore, the magnitude and extent of ground movement and the specific impact on surrounding structures must be assessed. In particular, the impacts of the potential ground movements on the adjacent travelled lanes of Highway 417 must be considered. Some lateral settlement of the nearest lane(s) must be expected. Temporary padding of the lane could be required, as could reductions in the posted speed limits. Following construction and backfilling of the retaining walls, the nearest lane(s) may need to be resurfaced to return the roadway to the correct cross-fall.

As a preliminary guideline, typical settlements behind soldier pile and lagging shoring are less than about 0.2 percent of the excavation depth, provided good construction practices are used, voids are not left behind the lagging, and the groundwater level is lowered in advance of excavation so that soil does not flow through the shoring over time. However this is only a preliminary guideline and is provided only to assist the owner's designers in carrying out an initial assessment of the expected settlements. A more detailed assessment of the expected settlements should be undertaken by the contractor. Should the preliminary assessment carried out using this estimated settlement indicate unacceptably large settlements to adjacent structures, roadways, or utilities, then a more detailed assessment should be carried out at the design stage (prior to tender) to better assess the shoring requirements or a more rigid form of shoring should be selected.

The magnitude of both lateral and vertical movements should be monitored during the construction period. The expected levels of deformation should be established by the contractor and alert levels should be set at which the designers should review the deformation and consider modifications to the design and/or construction procedures.

5.4 Proposed Transitway and Ramp Storm Sewers

A 675 millimetre diameter storm sewer is proposed under the Transitway pavement structure. Between Stations 7+070 and about 7+170 this sewer will be constructed within bedrock. From about Station 7+170 to 7+310 the sewer will be constructed in a predominantly layered sandy silt or glacial till subgrade. At the ramp structure, the sewer alignment will be under the north boulevard and have a minimum cover of about one metre.

A new section of 300 millimetre diameter storm sewer for the ramp will drain west between Stations 17+285 and 17+320 and outlet into an existing drainage course. A new section of 300 millimetre diameter ramp storm sewer will also be provided between Stations 17+407 and 17+452 and will outlet into the existing 300 millimetre diameter Highway 417 storm sewer.

5.4.1 Excavation

The recommendations provided in Section 5.2 of this report should be followed for excavation of the storm sewers. All pipe laying activities in the overburden should be carried out within a properly designed and constructed trench box as the overburden at this site is prone to sloughing.

East of about Station 7+170, the subgrade of the Transitway storm sewer changes from bedrock to frost susceptible layered sands, silts or glacial till. Within this area the sewer is also within the zone of seasonal frost penetration (i.e., 2.1 metres below grade). To prevent differential frost heave at this location, all frost susceptible material should be removed to a maximum depth of 2.1 metres or to the bedrock surface whichever is reached first and be replaced with OPSS Granular B Type II. From this point eastward a 10 horizontal to 1 vertical frost taper should then be provided from the 2.1 metres depth up to the bottom of the pavement structure (subgrade level). To prevent transverse longitudinal differential frost movement, the frost susceptible subgrade should be removed full width of the Transitway.

Beneath hard surfaced areas only, where the ramp storm sewer will be located within the zone of frost penetration (i.e., 2.1 metres below finished grade under pavement areas) frost tapers should be provided at slopes and depths indicated in Ontario Provincial Standard Drawing (OPSD) 803.030 and 803.031. These transitions should be provided both transverse and longitudinal to the sewer.

5.4.2 Pipe Bedding

The pipe bedding and backfill for the storm sewers should be in accordance with OPSD 802.030 to 802.033. The bedding material should consist of granular material meeting OPSS requirements for Granular A. The storm sewer pipe should be covered with 300 millimetres of granular material meeting OPSS Granular A or Granular B Type I having a maximum size of 25 millimetres. All granular bedding and backfill materials must be compacted in maximum 150 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment. To allow adequate compaction of the bedding material, the soil subgrade surface should be undisturbed and any groundwater inflow should be pumped from the excavation. If subgrade disturbance does occur, it may be necessary to firm up the subgrade with an additional 150 to 300 millimetres of well graded, coarse crushed stone, such as that meeting OPSS Granular B Type II (100 millimetre minus crushed stone), as a sub-bedding. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

5.4.3 Trench Backfill

As the storm sewers exist beneath future facilities, the trench backfilling should be carried out in a manner which is compatible with these uses. In order to provide an acceptable subgrade in these areas, and minimize distress due to differential frost heaving and differential settlement between areas underlain by service trench backfill and non-disturbed soil and rock, it is important that the excavated native soil and well shattered rock materials be used in the frost zone (between the roadway subgrade level and 2 metres below the proposed future finished roadway grade). The portion of the trench backfill in the frost zone should match that exposed on the adjacent trench walls.

Consequently, the excavated soil suitable for use as backfill should be separated for reuse within the frost zone portion of the service trench. The trench backfill soils should be placed in maximum 225 millimetre thick lifts and should be compacted to at least 95 percent of standard Proctor maximum dry density using suitable vibratory equipment.

The native silty soils and glacial till at this site are sensitive to changes in moisture content. Therefore, some drying of materials and/or extra compactive effort may be required to compact these materials to the required density. All organic soils and wet soils should be wasted. Any large boulders (i.e., greater than 300 millimetres) within the glacial till or other overburden soils should also be wasted from the site.

Backfill within the zone of frost penetration below the bedrock surface should consist of compacted, well shattered, broadly graded, rock material (maximum recommended size of 300 millimetres).

Backfill below the zone of seasonal frost penetration could consist either of acceptable native soil, well shattered bedrock, or imported sand or sand and gravel. All imported backfill material should be placed in maximum 225 millimetre thick lifts and should be compacted to at least 95 percent of standard Proctor maximum dry density using suitable vibratory equipment.

Most of the native soils which exist at this site are highly frost susceptible and are prone to significant ice lensing with consequent heaving and settlement upon thawing. In order to carry out the work during freezing temperatures and have the trench backfill perform acceptably as a road subgrade, the service trench should be opened for as short a time as possible and excavation should be carried out only in lengths in which the construction operations (including backfilling) can be fully completed in one working day. The sides of the trench should not be exposed to freezing temperatures to allow ice lensing to occur. In addition, the backfill should be excavated, stored and replaced without being substantially disturbed by frost or contaminated by snow or ice.

It is recommended that no road construction be planned for the winter period. In some areas, the subgrade consists of frost susceptible soil which would experience total and differential frost heaving as the work takes place. In addition, the unavoidable introduction of frost, snow and ice into the roadway materials during this surface construction could adversely affect the performance of the road structure. Furthermore, construction traffic would be operating on the subgrade when it is at its weakest state during the spring. This would cause a further breakdown in the subgrade with associated contamination of the granular materials. As such, roadway construction should not be started until the frost is completely out of the ground (mid-May).

5.4.4 Seepage Barriers

In order to avoid the introduction of a more permeable drainage channel along which groundwater could flow and contribute to additional permanent groundwater level lowering in the area of this site, the service trench bedding and backfill should be interrupted with seepage barriers. The seepage barriers should extend from trench wall to trench wall and from the native subgrade level to at least the top of the granular pipe cover material, i.e. to 300 millimetres above the pipe. These could consist of a 1.0 to 1.5 metre wide barrier of imported compacted silty clay or glacial till. However, concrete should only be used beneath the depth of frost penetration. The seepage barriers should be placed at about 150 to 200 metre intervals.

5.5 Long Term Side Slopes

It is understood that the Transitway will be constructed using open cut techniques between Stations 7+070 and 7+140 (i.e., west of the ramp structure to the western project limit).

The subsurface conditions within this section of the Transitway, based on borehole 97-7, are expected to consist of about 2 metres of sand fill overlying about 1.7 metres of sand and gravel

glacial till with the surface of the bedrock at about 3.7 metres depth (about elevation 64.4). The groundwater level is indicated to be at about 2.3 metres depth below the ground surface.

It is understood that the proposed Transitway gradeline within this section is at about elevation 63.5 metres. As such, the excavation, which will be a maximum of about 4.5 metres in depth (based on an average existing ground surface of about elevation 68.0 metres) will be through the sand fill, glacial till and about 1 metre into the underlying bedrock. It is anticipated that the construction of the Transitway will lower the permanent groundwater level in the area of the cut to at least the pavement subgrade level.

Therefore, based on the above, it is considered that for design purposes, 2 horizontal to 1 vertical or flatter slopes are acceptable within the cut section from a geotechnical point of view. This applies to the overburden materials exposed both above and below the measured existing groundwater level provided that adequate groundwater level lowering is carried out prior to excavation and construction of the Transitway.

Final grading of these slopes to their final inclination should be deferred until the groundwater level has stabilized at the lower level. In the interim, the slope may slough if cut at this angle.

5.6 Bridge and Retaining Wall Foundation Options

5.6.1 General

In general, the subsurface conditions on this site consist of a surficial layer of fill material overlying layered sandy silt, silty clay, and clayey silt, which is underlain by sandy soils (along the eastern portion of the site) and glacial till. The surface of the sandstone bedrock exists at about 4 metres depth at the western project limit (Station 7+070 metres) and deepens to about 12.5 metres depth at the eastern project limit (Station 7+310 metres).

It is understood that the bridge structure will be founded on spread footing foundation on the sandstone bedrock surface. It is also understood that the retaining walls, which will be used to form the future 'cut' and will be approximately 6 to 7 metres in height, are to be founded on spread footing foundations from about Stations 7+180 to 7+220 (i.e., in areas of shallow bedrock) and conventional H-pile foundations between Stations 7+220 and 7+310 (i.e., in areas where the surface of the bedrock is deeper).

5.6.2 Axial Geotechnical Resistance - Stations 7+070 to 7+220

Three options that could be considered for the shallow foundations for the retaining walls within this section of the project include:

1. Spread footing on bedrock.
2. Spread footings on lean concrete placed over the bedrock surface.
3. Spread footings placed on a pad of compacted engineered fill placed over the bedrock.

Spread footing on the glacial till are not recommended due to:

1. The lower available bearing resistance.
2. The sensitivity of the overburden material to disturbance due to groundwater inflow or from construction traffic. Foundations constructed on disturbed soil could experience unpredictable settlements.
3. The potential for differential settlements between walls supported on the glacial till versus the adjacent walls supported on unyielding rock.

Provided there are no continuous mud seams below the founding level, footings on the bedrock surface, or on a platform of lean concrete (compressive strength of greater than 5 MPa) extending down to the bedrock surface, may be designed using a factored geotechnical resistance at ULS of 1,500 kPa. SLS resistances do not apply to the design of footings on the sandstone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. The lean concrete could be placed directly against the excavation side slope, which could be excavated as steep as possible.

Boreholes 97-4 and 97-5 indicate that the bedrock contains seams filled with frost susceptible soil (mudseams) to about elevation 60.9 metres. At the borehole locations, the mud seams are about 70 millimetres thick. As such, it is suggested that for footings founded above elevation 60.9 metres, 50 millimetre diameter probeholes on 3 metre centres be advanced along the foundations to verify the presence and extent of the mud seams. If or where the mud seams are present, reduced bearing parameters may be required.

Spread footings placed on a pad of engineered fill consisting of OPSS Granular B Type II which extends down to the bedrock surface may be designed using a geotechnical resistance at SLS of 200 kPa and a factored geotechnical resistance at ULS of 250 kPa. The pad of engineered fill must be placed within the full zone of influence of the foundations. The zone of influence is considered to extend out and down from the edge of the footings at a slope of 1 horizontal to 1 vertical. This requirement will therefore result in a larger excavation, versus bearing directly on the rock or lean concrete. The foundation settlements should be less than 5 to 10 millimetres provided the engineered fill is compacted to at least 95 percent of the materials standard Proctor maximum dry density. The fill material should be placed in the maximum 300 millimetre thick lifts and compacted to at least 100 percent standard Proctor maximum dry density using suitable vibratory compaction equipment.

Should rock anchors be required to resist uplift forces caused by overturning, then the design of the anchors should be based on the parameters provided in Section 5.3.3

5.6.3 Axial Geotechnical Resistance - Stations 7+220 to 7+310

As discussed above, the retaining walls between stations 7+220 to 7+310 are to be founded on H-pile foundations. The factored *structural* axial ULS resistance of a HP310 x 79 steel H pile would be 1,450 kilonewtons. The ULS factored *geotechnical* resistance of the pile should equal or exceed the structural resistance if it is driven to a final set of 10 blows for the last 12 millimetres of penetration using a hammer developing about 28 kilojoules of transferred energy per blow (i.e., this assumes that steel with a yield stress of 350 megapascals).

For piles end-bearing on or within bedrock, Serviceability Limit States (SLS) generally do not govern the design since the stresses required to induce 25 millimetre of movement (i.e., the typical SLS criteria) exceed those at ULS. Accordingly, the post-construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

The pile tips for vertical piles should have the flanges suitably reinforced while battered piles should be provided with standard Titus bearing points to ensure adequate seating.

Relaxation of the piles following the initial set could result from several processes, including:

- the dissipation of negative excess pore water pressures in the silty soils; and
- the driving of adjacent piles.

Provision should therefore be made for re-striking all of the piles at least once to confirm the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed no sooner than 48 hours of the previous set.

It is recommended that dynamic monitoring and capacity testing be carried out (e.g., by the contractor) at an early stage in the piling operation to verify both the transferred energy from the pile driving equipment and the load carrying capacity of the piles. Further guidelines can be provided on the testing frequency to be included in the specification once the foundation design has been finalized. However, as a preliminary guideline, the specification should require that at least 5 percent of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing. As well, CAPWAP analyses should be carried out for at least one third of the piles tested, with the results provided no later than one week

following testing. The final report should be stamped by an engineer licensed in the province of Ontario.

The foundation and piling specifications should be reviewed by Golder Associates prior to tender and the contractor's submission (i.e., shop drawings, equipment, procedures, and set criteria) should be reviewed by the geotechnical consultant prior to the start of piling.

Piling operations should be inspected on a full time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

For piles which are less than about 3 metres in length (i.e., "short piles"), the shallow depth of soil may not provide adequate resistance to lateral movement and the pile may not be stable. For this circumstance, to improve the stability of the pile, consideration could be given to providing structural fixity between the pile and the pile cap as well as between the pile and bedrock surface. The structural fixity at the pile cap would be designed by the structural engineer but would likely involve increasing the embedment length of the pile into the pile cap. At the bedrock surface, the piles could be socketed into the bedrock so that rotation will be prevented. The piles would be installed in a borehole drilled into the bedrock. A cased hole would first be drilled through the overburden, a socket drilled into the bedrock, the borehole flushed and filled with grout, and the pile would be inserted. With this arrangement, there should be no technical restriction on the minimum pile length.

5.6.4 Frost Protection

All foundation elements (pile caps and footings) should be provided with a minimum of 1.8 m of soil cover for frost protection.

The requirements for frost protection can be waived provided that the bedrock beneath the foundation does not contain soil filled seams or joints. However, this can only be confirmed at the time of construction by drilling 50 millimetre diameter probeholes on 3 metre centres along the footing alignment.

5.6.5 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

5.6.6 Lateral Earth Pressures

The lateral earth pressures acting on the retaining wall, abutment stems and any associated wing walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type II should be used as backfill behind the walls. The fill should be compacted in lifts having a loose thickness no greater than 300 millimetres. Each lift should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory plate or light roller vibrating compaction equipment.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3501.00 and 3504.00. Since the excavation will be made below the groundwater level, continuous seepage into the drainage system should be expected. To avoid continuous seepage through the wall drains and onto the Transitway, the perforated longitudinal drain pipe should be connected directly to the storm sewer system, and should be at or slightly below the wall drain level.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.9.3. The fill should be compacted in lifts having a loose thickness no greater than 300 millimetres. Each lift should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory plate or light roller vibrating compaction equipment. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC*). The position of the shoring will determine which case applies.

- For Case I, the pressures are based on the native soils behind the excavation/shoring and the following parameters (unfactored) may be used:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	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 static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.2. Based on experience, for the subsurface conditions at this site, a 15 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.2g to 0.23g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.23$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.34$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.12$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v .

Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case ii) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.33	0.33
Non-yielding wall	0.82	0.68	0.68

Note: These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio of this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.23. This corresponds to displacements of up to about 60 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K \gamma d + (K_{AE} - K) \gamma (H-d)$$

where $\sigma_h(d)$ is the lateral earth pressure at depth, d, (kPa)
 K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
 K_{AE} is the seismic active earth pressure coefficient;
 γ is the unit weight of the backfill soil (kN/m^3), as given previously;
 d is the depth below the top of the wall (m); and
 H is the total height of the wall (m).

5.6.7 Sliding Resistance – Spread Footings

Resistance to lateral forces / sliding resistance between the concrete footings and the rock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For footings on bedrock, a coefficient of friction of 0.7 may be used. This is an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If greater resistance is required, the footings could be provided with shear keys or prestressed rock anchors could be used to increase the normal stress level across the interface.

Shear keys in competent rock located in the centre portion of the footing can be proportioned using a ULS factored horizontal geotechnical resistance of 250 kilopascals. This bearing resistance assumes that the shear key has a depth of at least 0.75 metres and that the bearing surface is vertical and undisturbed. For a shear key depth of 0.6 metres, the ULS factored horizontal geotechnical resistance could be taken as 200 kilopascals. The geotechnical resistance should be reduced if the rock is unavoidably disturbed, or excavation is planned in rock within about 7 metres of the wall. A minimum shear key depth of 0.6 metres is suggested.

To minimize disturbance to the rock, it is suggested that the vertical cuts for the shear key be carried out using rock sawing equipment or line drilling and mechanical rock removal where the space between the edges of the line drilled holes does not exceed 125 millimetres. The rock between the cuts should then be carefully removed without disturbing the adjacent rock.

Based on previous experience in the Ottawa area, saw cutting a maximum shear key depth of 1.0 metres with a preferred depth of 0.75 metres can be obtained using relatively small rock sawing equipment; greater depths (up to about 3.0 metres) can be obtained using backhoe mounted rock sawing equipment.

Guidelines on rock anchor design were previously discussed in Section 5.3.3 is also applicable here.

It is understood that battered piles will be used to resist horizontal loads.

5.6.8 Resistance to Lateral Loads – Piled Foundations

Lateral loading could be resisted fully or partially by the use of battered steel H-piles.

If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h .

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

n_h is the constant of horizontal subgrade reaction (use 2.2 MPa)

z is the depth (m)

B is the pile diameter/width (m)

A maximum lateral resistance of 100 kN at ULS, and a maximum lateral resistance of 25 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 79 piles. These values are based on the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in Table C6.8.7.1(a) of the *Commentary* to the CHBDC.

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 as follows:

<i>Pile Spacing in Direction of Loading (d = Pile Diameter)</i>	<i>Reduction Factor</i>
8d	1.0
6d	0.7
4d	0.4
3d	0.25

Additional lateral resistance can be provided by socketting the piles into the bedrock.

5.7 Noise Attenuation Barrier Foundations

It is understood that a noise attenuation barrier will be provided along the north side of the Transitway alignment. It is also understood that the barrier will be founded on 762 millimetre diameter concrete piers which will be in the order of 2.8 to 3.4 metres long.

These concrete piers are typically constructed by augering a hole, casing the hole with a light liner, and filling the liner with concrete.

A wall constructed of relatively stiff materials such as cast-in-place concrete or precast concrete is considered to be somewhat sensitive to post-construction total and differential settlement. Such a wall should therefore derive its support from the inorganic native soils below any fill present.

The pier foundations should be provided with 1.5 metres of earth cover for frost protection. To avoid frost jacking (frost adhesion and heaving) of the piers, oversize pier holes should be drilled to allow non frost susceptible sand fill to be placed between the pier and the frost susceptible soil.

Alternatively, the outside of the pier form in the frost zone could be provided with a smooth, impervious coating to prevent frost adhesion.

The design of the foundations for the noise attenuation barrier is the responsibility of the supplier. However, the following generalized soil parameters should be used for the design of the noise attenuation barrier foundations:

Cohesion c (kPa)	Friction Angle ϕ (degrees)	Unit Weight γ (kN/m³)
0	28	19

5.8 Pavement Design

It is understood that the proposed pavement structure, in keeping with the present Transitway roadway design, will be a flexible pavement structure. Based on the information provided, the Transitway will have an urban cross section as shown in the typical sections on Harmer Podolak Drawing T-13, typical sections.

The pavement subgrade will have to be carefully crowned or shaped to promote drainage and to ensure a uniform and consistent granular subbase thickness. The granular pavement materials should extend uninterrupted to the subdrains.

Two design loading conditions were used in determining the pavement structure. The Transitway Manual loading of 2700 Design Traffic Number (DTN) which is the number of equivalent 80 kilonewton single axle loads per day in both directions and the estimated bus volumes from current traffic studies of 790 buses per day.

Based on an analysis of axle loading on half and fully loaded standard and articulated buses used by OC Transpo, the truck factors were found to range from 5 to 21.5. The type of bus and passenger volume has a significant impact on the DTN. Assuming that the split between regular and articulated buses is 50 percent and that half of the buses will be fully loaded and half will be half loaded, the DTN of 790 buses per day would be 9,320 or about 3.5 times the volume of the Transitway Manual.

Based on a 20 year design life with a DTN of 9,320, the design ESALs for the Transitway would be 36.6 million or Traffic Category Level E.

5.8.1 AASHTO Pavement Analysis

The following design parameters were used in the analysis for pavement design purposes:

Initial Serviceability	-	4.3
Terminal Serviceability	-	3.2
Reliability Level	-	95%
Overall Standard Deviation	-	0.45

The following table summarizes the structural and drainage coefficients used in the AASHTO pavement analysis:

Component Layer	Structural Coefficients	Drainage Coefficient
Hot Laid Asphaltic Concrete	0.44	1.0
Granular O Base	0.14	1.2
Granular B Type II Subbase	0.14	1.0

The subgrade soil will be variable consisting of silty sand, sandy silt, clayey silt, glacial till and bedrock. Where the subgrade consists of soil, a resilient modulus of 30,000 kilopascals was used in the AASHTO analysis. This value represents a good subgrade not disturbed by construction traffic.

Based on the above parameters and ESALs, the design structural number, S_N , is 212 mm.

5.8.2 Pavement Structure

Transitway

Based on the above parameters, the composition of the Transitway pavement should consist of the following:

- 150 millimetres of hot mix hot laid asphaltic concrete
- 200 millimetres of OPSS Granular O base
- 800 millimetres of OPSS Granular B, Type II subbase

In areas underlain by bedrock the above subbase should be replaced with a minimum 300 millimetre thick layer of well graded rock shatter to provide a drainage path above the bedrock subgrade. This rock shatter should be blinded with a minimum of 300 millimetres of OPSS Granular B Type II to prevent loss of Granular O into the voids within the rock shatter. Alternatively, a class II woven geotextile meeting the requirements of OPSS 1860 should be provided over the rock shatter. The geotextile should meet the following physical properties:

Property	Value
Grab Tensile Strength	≥ 890 Newtons
Grab Tensile Elongation	≥ 15 percent
Mullen Burst	≥ 2750 kPa
Puncture	≥ 400 Newtons
Trapezoid Tear	≥ 330 Newtons

An LINQ GTF 200 or Amoco 2002 geotextile, or approved equivalent, would be suitable for this purpose.

E - N/S Ramp

The composition of the E-N/S Ramp should consist of the following:

150 millimetres of hot mix hot laid asphaltic concrete
 150 millimetres of OPSS Granular O base
 600 millimetres of OPSS Granular B, Type II subbase

Raised Boulevards

The raised boulevards should be provided with the following pavement structure:

90 millimetres of asphaltic concrete
 150 millimetres of OPSS Granular O base
 300 millimetres (minimum) of OPSS Granular B, Type II subbase

The thickness of the subbase layer has been increased from 150 millimetres in the Functional Design to 300 millimetres to improve the strength at the edge of the boulevard to prevent edge cracking.

The granular material used on site should meet the requirements of the City of Ottawa S.P. F3147.

The granular material used on site should be placed and compacted in accordance with Ontario Provincial Standard Specifications (OPSS) 501 Method A.

5.8.3 Pavement Type and Depths

Transitway

The composition and thickness of the asphaltic concrete along the Transitway should consist of the following:

Superpave 12.5 FC1 – surface course	40 mm
Superpave 19 base course	50 mm
Superpave 19 base course	60 mm

Raised Boulevards

The composition and thickness of the asphaltic concrete along the raised boulevards should consist of the following:

40 millimetres Superpave 12.5 surface course
50 millimetres Superpave 19.0 base course

E-N/S Ramp

The composition and thickness of the asphaltic concrete along the E-N/S Ramp should consist of the following:

Superpave 12.5 FC1 – surface course	40 mm
Superpave 19 base course	50 mm
Superpave 19 base course	60 mm

The composition and thickness of the asphaltic concrete for the paved shoulders should consist of the following:

Superpave 12.5 FC1 surface course	40 mm
Superpave 19.0 base course	50 mm

The asphaltic concrete should meet the requirements of City of Ottawa specification F-3106.

5.8.4 Asphalt Cement Grade

The asphaltic concrete used on this project should to be made with PG 64-34 asphalt cement on all lifts.

5.8.5 Traffic Category

Based on the design ESALs of 36.6 million, the Transitway traffic category is a Level E. The traffic category for the ramp should be Level C.

5.8.6 Embankment Borrow

Although the E N/S ramp will be at or near the existing grade, some embankment construction will be carried out requiring the use of borrow material for the ramp and the detour. The maximum borrow fill thickness for the ramp is in the order of about 0.5 metres between about Stations 17+210 and 17+225. The maximum borrow fill thickness for the detour is about 1.5 metres between Stations 0+165 to 0 + 270.

As the topsoil layer is not uniform under the full footprint of the ramp, in preparation for construction of the ramp all organic material should be removed from the ramp embankment footprint. This will prevent differential frost heave in areas underlain by topsoil and areas not underlain by topsoil. On sloping ground benching should be carried out in accordance with OPSD 208.01. The borrow fill below the pavement structure should consist of OPSS Select Subgrade Material, or acceptable compactable earth borrow fill material. The native silty sand and sandy silt glacial till from above the groundwater table would make suitable borrow material provided the water content of the material is at or near optimum. Silty sand and sandy silt glacial till from below the groundwater table would probably be too wet.

All embankment fill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.8.7 Transition Treatments

Transition from bedrock to earth subgrade required in the vicinity of Station 7 +140 should be carried out in accordance to OPSD 205 series. The transition depth "t" should be taken as 1.8 metres.

At the transition gore of the ramp from Highway 417 the asphaltic concrete and the granular thicknesses should be tapered either up or down to match the pavement structure of Highway 417 at a slope of 10 horizontal to 1 vertical. For ease of placing and compaction of granular and asphaltic concrete the minimum width of the gore should be 1.5 metres.

At Station 17+135 the pavement structure of the new ramp should be tapered up or down at a slope of 10 horizontal to 1 vertical as required to match that of the existing ramp.

5.8.8 Subexcavation

Although the above granular thicknesses are considered adequate for structural design, experience has shown that they may not be sufficient to carry out the short term heavy axle loads of construction traffic over the subgrade, especially if construction is carried out during adverse weather conditions. It is suggested that allowance be made for proof-rolling of subgrades, for subexcavation of soft and wet areas to an additional depth of up to 300 millimetres in cut areas, and for replacement with clean compacted earth borrow meeting OPSS select subgrade fill or Granular B. Proof-rolling may be carried out using either a loaded truck or large rubber-tired smooth drum roller and should consist of at least one pass coverage of the full subgrade width carried out under the supervision of an experienced geotechnical engineer. If the subgrade is soft and wet then consideration could be given to placing a geotextile over the subgrade to provide added support of the pavement structure during construction and to prevent pumping of the subgrade. The geotextile should consist of a woven Class II geotextile as per OPSS 1860.

5.9 Corrosion of Buried Steel and Cement Type

Three samples of groundwater (one from each of boreholes 97-1, 97-3, and 97-6) were submitted to Accutest Laboratories Ltd. for basic chemical analysis related to potential corrosion of buried steel elements and sulphate attack on buried concrete elements. The results of this testing are provided in Appendix C.

The testing indicate that the sulphate content of the groundwater range from about 52 to 79 milligrams per litre; these results indicate that concrete made with Type 10 Portland cement should be acceptable for substructures.

The measured pH ranged from about 7.1 to 7.5, the conductivity ranged from 733 to 2194 $\mu\text{mhos/cm}$, and the chloride contents ranged from 10 to 436 milligrams per litre. These results indicate an aggressive environment for exposed ferrous metals. Buried ferrous elements should therefore be protected against corrosion by use of corrosion inhibiting coatings or equivalent means.

6.0 CONSTRUCTION CONSIDERATIONS

Various construction sequences could be considered for this design assignment. From a geotechnical point of view, however, the preferred alternative is to have the Transitway storm sewer and Transitway bulk excavation carried out concurrently to about 0.3 to 0.5 metres above the pavement subgrade. The excavation for the storm sewer should then be carried out prior to the construction of Transitway structures and before excavation of the Transitway to subgrade elevation as excavation for the storm sewer could jeopardize structures by undermining the foundations if carried out after structures were in place.

As indicated previously, the subgrade soils beneath the Transitway are highly sensitive to disturbance due to truck traffic, ponded water, and frost. Several options are available to prevent disturbance to the proposed roadway subgrade. A temporary road could be constructed above the design subgrade level to allow access to the various construction sites. In this case, it is suggested that the subgrade for the haul road be kept at least 1 metre above the design subgrade level; the granular thickness requirements for the road could then be the responsibility of the contractor; however, the contractor should understand that at least 1 metre of crushed stone fill will be required for construction vehicle access. This technique could also be employed for construction of the storm sewer. Alternatively, consideration could be given to excavating to the design roadway subgrade level and constructing a temporary access road at this level. In this case, it is suggested that at least 1 metre of OPSS Granular B Type II (100 millimetre minus crushed stone) be installed over a woven OPSS Class II geotextile to provide support for the construction traffic. Any granular material that becomes contaminated would have to be removed and, if necessary, replaced to provide the recommended thickness of Granular B Type II subbase material. Heavy and full vibratory compaction of the granular materials used for haul roads should precede use by construction vehicles.

During construction, haul vehicles should operate on temporary roads having a thickness of at least 1 metre of compacted crushed stone fill or as required to prevent disturbance to the subgrade. The subbase and base materials should be spread to the design levels with track mounted equipment and then compacted. The minimum of 1 metre of compacted crushed stone as a temporary road for construction traffic may be waived for paving operations only, provided a trial loading test using a loaded tandem truck does not indicate excessive deflections of the subsurface of the granular base. This test should not be carried out following periods of precipitation.

7.0 ADDITIONAL CONSIDERATIONS

Due to the complexity and magnitude of the proposed works within this Transitway section, and considering that several components of the proposed work are being investigated and reported by others, it is recommended that for continuity of information, the geotechnical consultant be involved throughout the design process in order to be able to address details which may affect the geotechnical guidelines provided in this report.

It is recommended that the design drawings and specifications for the proposed West Transitway be examined and accepted by the geotechnical consultant prior to the tendering stage or start of construction. This is to ensure that geotechnical related guidelines have been correctly interpreted.

It is recommended that inspections be carried out on all excavations to ensure that competent subsoils have been reached and so that any problem areas can be identified and adequate control procedures implemented as quickly as possible. Since the bedding and backfilling of the storm sewer will be fundamental to the successful future Transitway performance, control on the placing and compaction of granular and native soil backfill material will be essential. It is also recommended that the Geotechnical Engineer monitor the proof-rolling and approval of all roadway subgrade and that control on the placing and compaction of the granular roadway materials be carried out to ensure that the materials being used conform to specifications from both a grading and compaction point of view.

The subgrade surface for the roadway and storm sewers should be inspected by geotechnical personnel to determine if acceptable, undisturbed soil has been reached.

All load testing of the anchor/piles should be carried out under the direct supervision of the geotechnical consultant.

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT



SCALE 1:25,000
DATE AUG. 2006
DESIGN
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TITLE

KEY PLAN

FILE No. 061120203-1000-01
PROJECT No. 06-1120-203 REV. 0

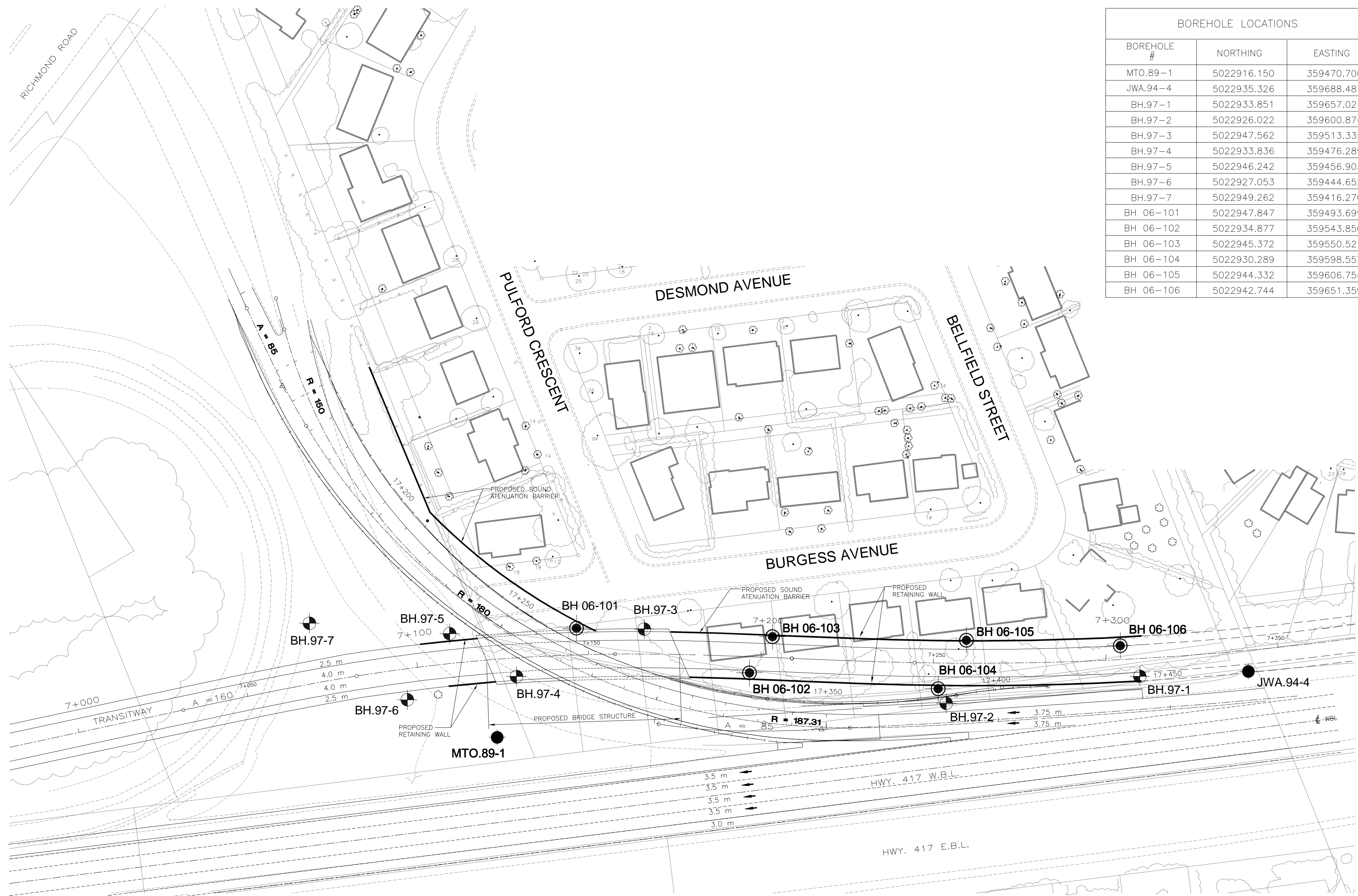
CHECK T.M.S.
REVIEW M.I.C.

PROPOSED E-N/S RAMP, WEST TRANSITWAY
OTTAWA, ONTARIO

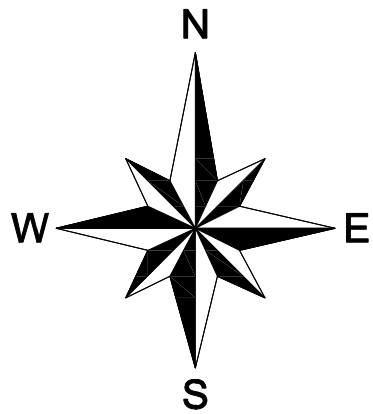
FIGURE

1

Drawing file: 06-1120-203-1000-02.dwg May 03, 2007 - 10:51am



BOREHOLE LOCATIONS		
BOREHOLE #	NORTHING	EASTING
MTO.89-1	5022916.150	359470.700
JWA.94-4	5022935.326	359688.481
BH.97-1	5022933.851	359657.021
BH.97-2	5022926.022	359600.876
BH.97-3	5022947.562	359513.335
BH.97-4	5022933.836	359476.289
BH.97-5	5022946.242	359456.903
BH.97-6	5022927.053	359444.652
BH.97-7	5022949.262	359416.270
BH.06-101	5022947.847	359493.699
BH.06-102	5022934.877	359543.850
BH.06-103	5022945.372	359550.521
BH.06-104	5022930.289	359598.557
BH.06-105	5022944.332	359606.756
BH.06-106	5022942.744	359651.359



LEGEND

- APPROXIMATE BOREHOLE LOCATION IN PLAN (PRESENT INVESTIGATION BY GOLDER ASSOCIATES LTD.)
- APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS 1997 INVESTIGATION BY GOLDER ASSOCIATES LTD.)
- APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION BY OTHERS)

REFERENCE:

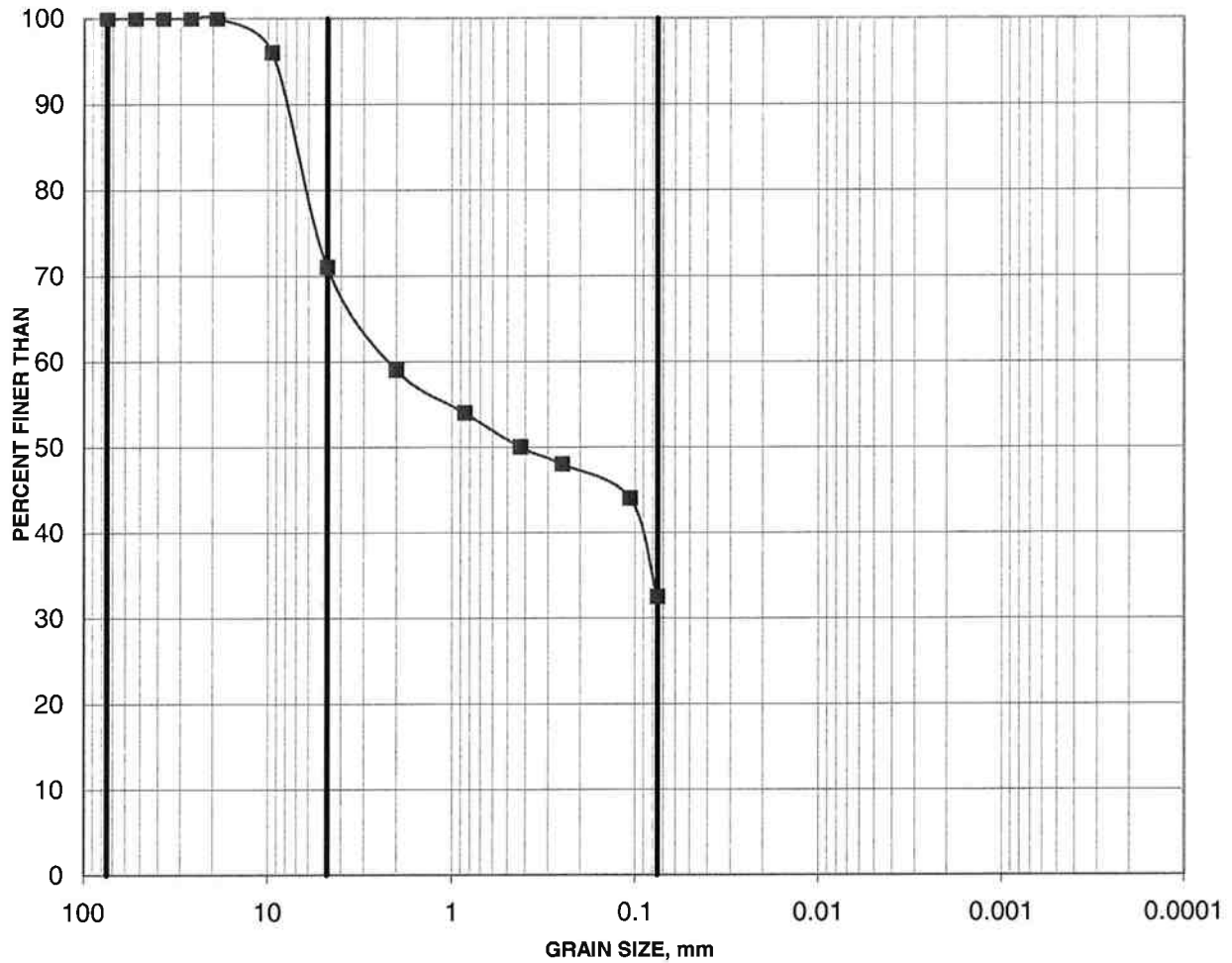
BASE PLAN SUPPLIED BY HARMER PODOLAK ENGINEERING CONSULTANTS INC.

15 0 15 30
SCALE 1:750 METRES

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	R/W
PROJECT PROPOSED WEST TRANSITWAY EXTENSION OTTAWA, ONTARIO						
TITLE SITE PLAN						
PROJECT No. 06-1120-203			FILE No. 061120203-1000-02.dwg			
DESIGN	CADD	N.B.H.S.	SEPT. 2006	SCALE	1:750	REV.
CHECK	T.M.S.	01 MAY 07				
REVIEW	M.I.C.	01 MAY 07				
Golders Associates Ottawa, Ontario			FIGURE 2			

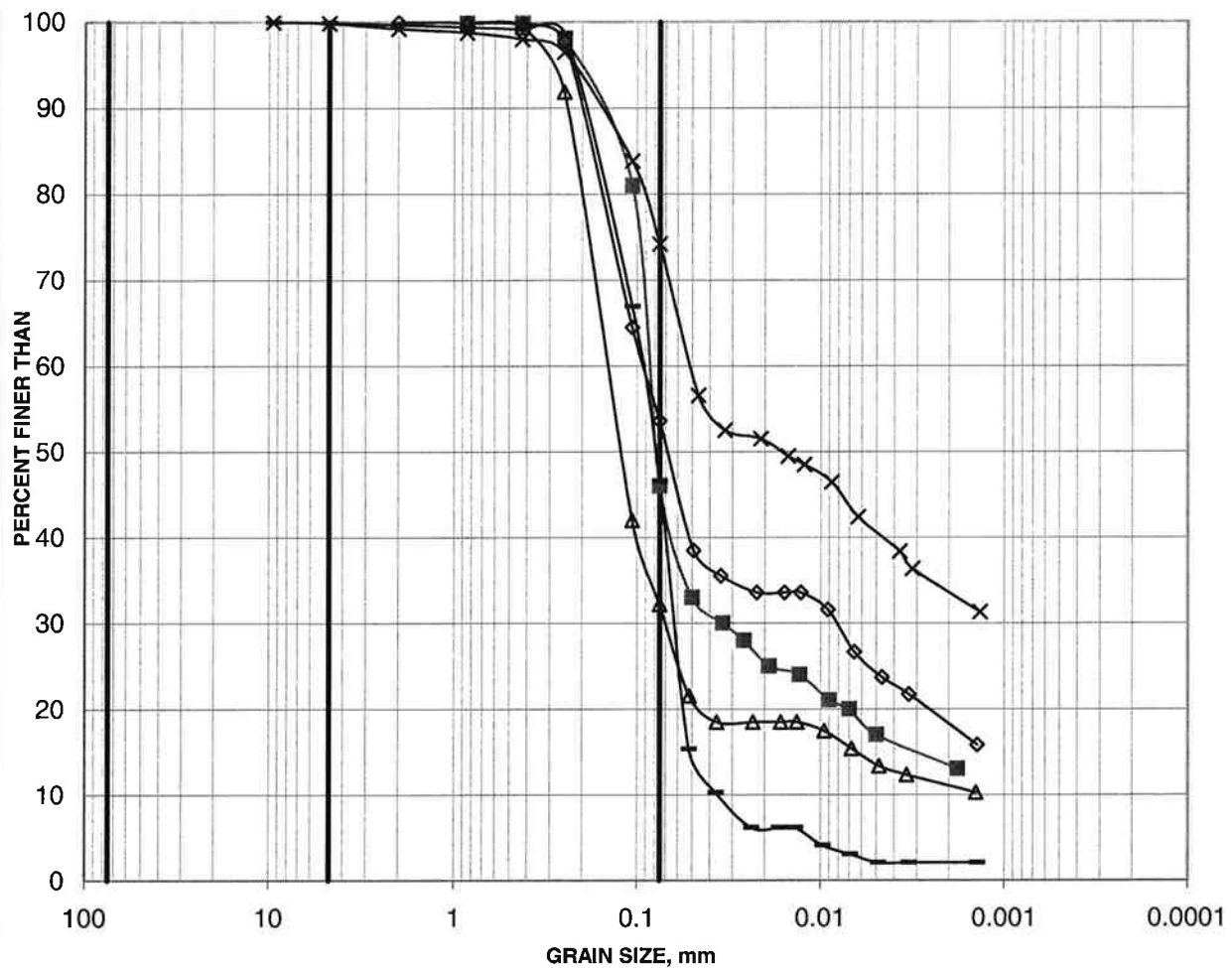
Silty Sand and Gravel (Fill)



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample
—■— 97-7	2

Layered Sandy Silt, Silty Sand, and Clayey Silt

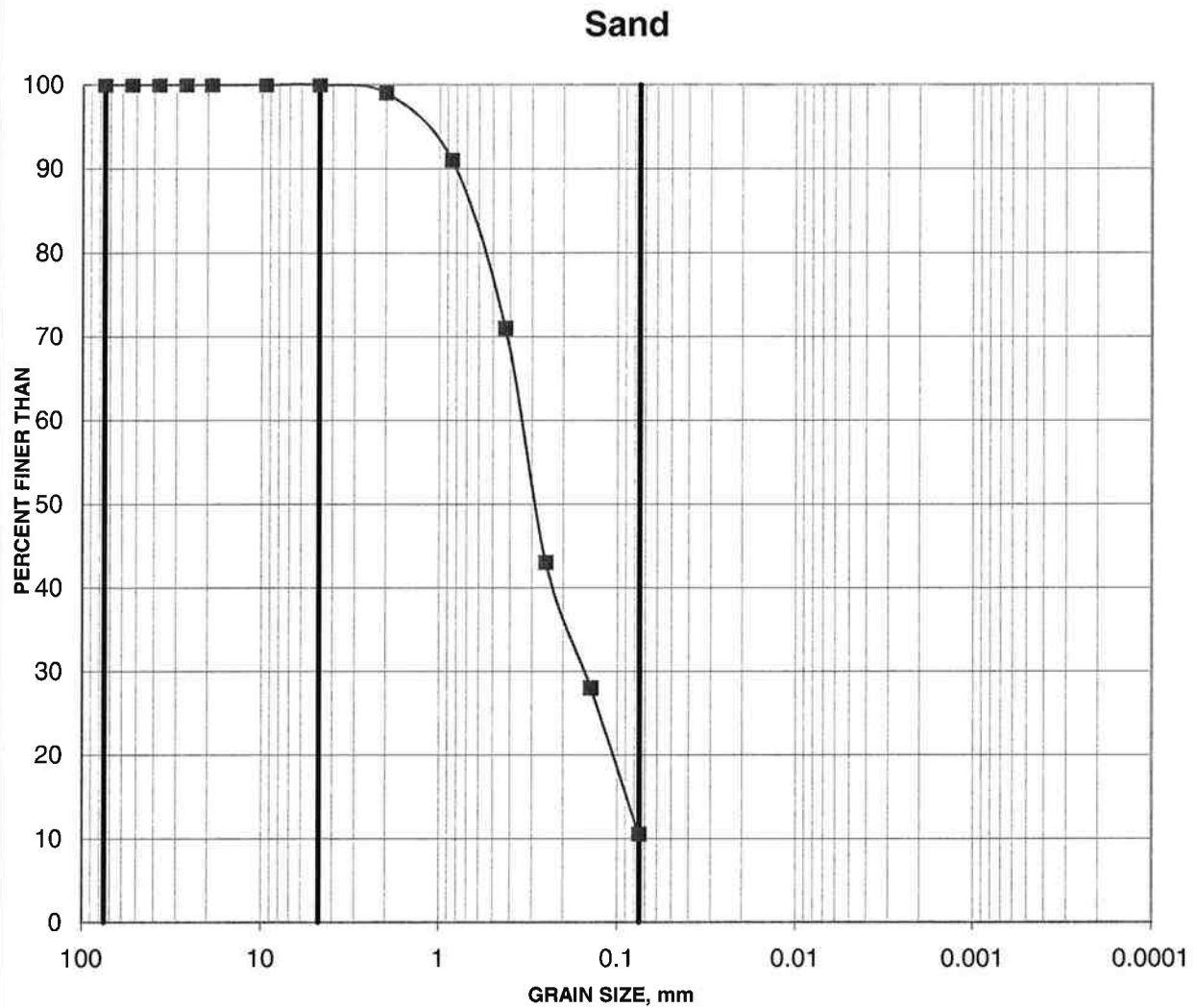


Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
97-2	6	
06-102	3	2.1-2.7
06-103	6	5.2-5.8
06-105	8	7.6-8.2
06-106	6	4.6-5.2

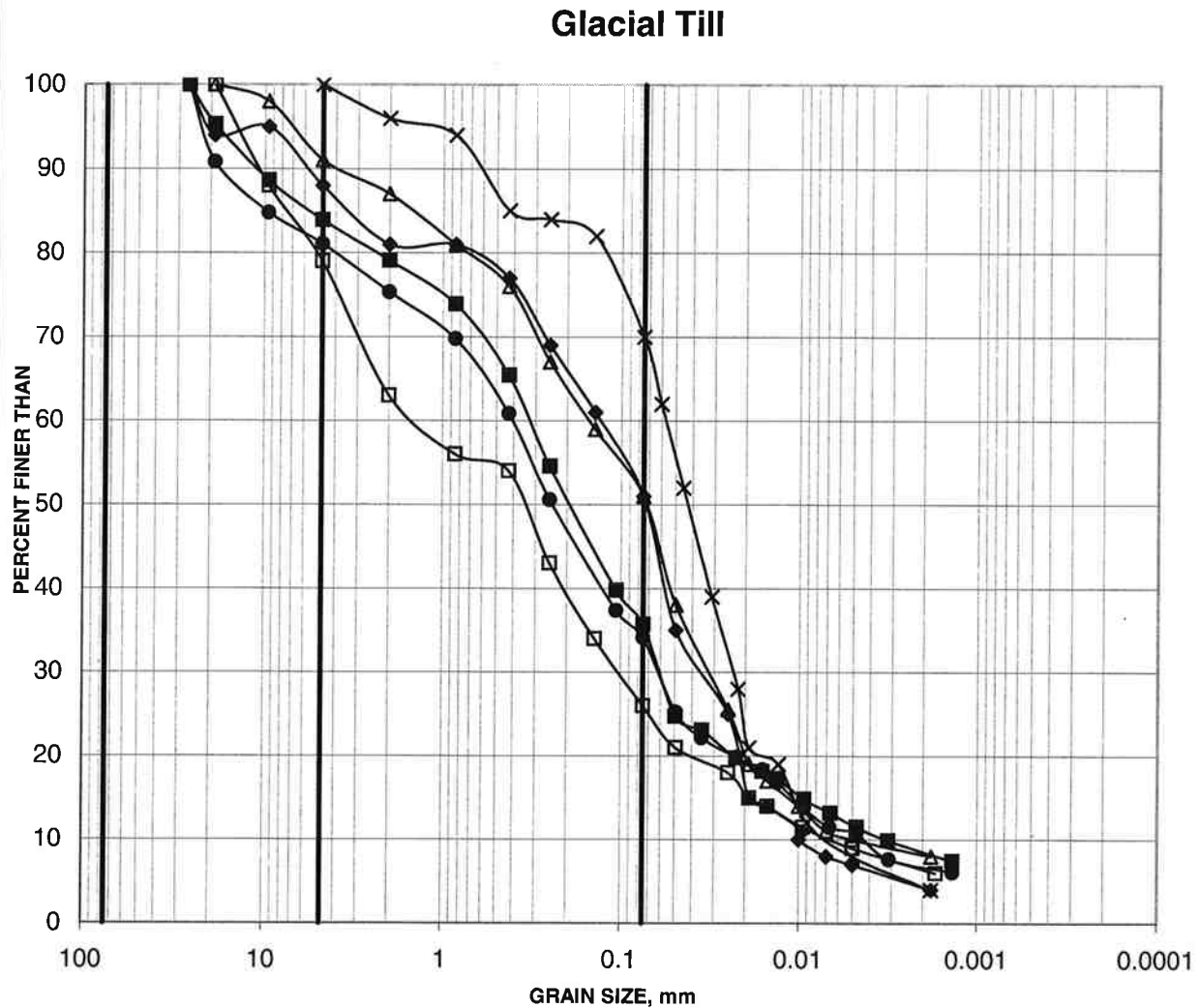
GRAIN SIZE DISTRIBUTION

FIGURE 5



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
—■ 97-2	9	6.8-7.5



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
—X—	97-1	8
—△—	97-3	7
—□—	97-3	8
—◆—	97-4	5
—■—	06-101	6
—●—	06-104	9

APPENDIX A

ABBREVIATIONS AND SYMBOLS RECORD OF BOREHOLE SHEETS PRESENT AND PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES

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
DO	Drive open
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 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

Peizo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm² 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	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 limited
w_L	liquid limit
C	consolidaiton (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 test (LV-laboratory vane test)
γ	unit weight

Note:

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

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 \times acceleration due to gravity)

(a) Index Properties (cont'd.)

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)

(b) 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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	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 $= \sigma'_p/\sigma'_{vo}$

(d) 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 $= \tan \delta$
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 $= (\text{Compressive strength})/2$

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

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

GRAIN SIZE

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

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

O:\ Templates\Rock Description Terminology

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 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 information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B –	Bedding	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

PROJECT: 06-1120-203

RECORD OF BOREHOLE: BH 06-101

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: August 14, 2006

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
								nat V. + Q - rem V. ⊕ ⊖ ○				Wp — W — Wl					
								20	40	60	80	10 ⁻⁶	10 ⁻⁵			10 ⁻⁴	10 ⁻³
0	Power Auger 200mm Diam. (Hollow Stem)	Ground Surface		68.82													
		Dark brown sandy topsoil, trace gravel (FILL)		68.52													
		Brown silty sand (FILL)		68.36													
		Dark brown silty TOPSOIL		68.06													
		Brown SILTY fine SAND															
1		Grey brown CLAYEY SILT		67.86	1	50 DO	11										
		Compact brown SILTY fine SAND		1.16													
				67.14													
2		Very stiff grey brown layered CLAYEY SILT and SANDY SILT with sand seams		1.68	2	50 DO	4										
			66.02	3	50 DO	3											
3	Stiff grey layered CLAYEY SILT and SANDY SILT with sand seams		2.80														
				4	50 DO	1											
4																	
				64.10													
5		Compact grey SANDY SILT, some gravel, trace clay, occasional sandy silt seam and cobble (GLACIAL TILL)		4.72	5	50 DO	40										
					6	50 DO	20										
6					7	50 DO	>50										
				62.48													
				6.34													
7		Fresh light grey fine to medium grained SANDSTONE BEDROCK with numerous dark grey green thinly laminated to laminated shale interbeds			8	NQ RC	DD	96	90	79							
8				9	NQ RC	DD	100	93	67								
9																	

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: T.M.S.

BOREHOLE 06-1120-203-1000.GPJ HYDROGEO.GDT 2/21/07

PROJECT: 06-1120-203

RECORD OF BOREHOLE: BH 06-102

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: August 14, 2006

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
							20	40	60	80	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³		
0	Power Auger 200mm Diam. (Hollow Stem)	Ground Surface		69.11												
		Topsoil (FILL)		0.15												
		Brown clayey silt, some sand (FILL)														
1				68.20	1	50 DO										
		Dark brown silty TOPSOIL		67.95												
		Compact brown stratified SILTY fine SAND		1.16												
				67.28	2	50 DO										
2			Very stiff grey brown layered CLAYEY SILT and SANDY SILT with sand seams		1.83											
						3	50 DO									
3																
			Stiff grey layered CLAYEY SILT and SANDY SILT with sand seams		65.76	4	50 DO									
				3.35												
4					5	50 DO										
5																
6		Loose to dense grey SANDY SILT, some gravel, trace clay with silty sand and fine sand layers, occasional cobbles (GLACIAL TILL)		63.01	6	50 DO										
				6.10												
7					7	50 DO										
8					8	50 DO										
		Fractured SANDSTONE BEDROCK Fresh light grey fine to medium grained SANDSTONE BEDROCK with numerous thinly laminated to very thinly bedded shale interbeds		60.91	9	NQ RC										
				8.29												
9					10	NQ RC										
10	Rotary Drill NQ Core															
11					11	NQ RC										
		End of Borehole		57.80												
				11.31												
12																
13																
14																
15																

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: T.M.S.

BOREHOLE 06-1120-203-1000.GPJ HYDROGEO.GDT 2/21/07

PROJECT: 06-1120-203

RECORD OF BOREHOLE: BH 06-103

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: August 15, 2006

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
							20	40	60	80	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴			10 ⁻³
0		Ground Surface		69.08												
		Topsoil (FILL)		68.90												
		Brown silty sand, trace roots (FILL)		0.18												
				68.35												
1		TOPSOIL		68.17	1	50										
		Loose to compact brown SILTY fine SAND, occasional sandy silt layer		0.91		DO										
				67.25	2	50										
2		Very stiff grey brown layered CLAYEY SILT and SANDY SILT with sand seams		1.83		DO										
					3	50										
						DO										
3		Stiff grey layered CLAYEY SILT and SANDY SILT with sand seams		65.88	4	50										
				3.20		DO										
4		Very loose grey SILTY fine SAND, occasional silty clay layer		64.81												
				4.27	5	50										
5						DO										
					6	50										
						DO										
6		Compact grey SANDY SILT, some gravel, trace clay, occasional sand and sand and gravel layers (GLACIAL TILL)		63.14												
				5.94	7	50										
						DO										
7					8	50										
						DO										
8					9	50										
						DO										
						</										

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: T.M.S.

BOREHOLE 06-1120-203-1000.GPJ HYDROGEO.GDT 2/21/07

PROJECT: 06-1120-203

RECORD OF BOREHOLE: BH 06-104

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: August 16, 2006

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
								20 40 60 80				10 ⁻⁵ 10 ⁻⁴ 10 ⁻³					
								nat V. + Q - ● rem V. ⊕ U - ○				Wp ——— W ——— WI					
								20	40	60	80	20	40	60	80		
0	Power Auger 200mm Diam. (Hollow Stem)	Ground Surface		68.70													
		Brown and dark brown sandy silt, trace gravel and topsoil (FILL)		0.00													
				67.97													
1		TOPSOIL		0.88	1	50 DO	16										
		Loose to compact brown SILTY fine SAND, occasional fine sand and sandy silt seam															
2		Very stiff grey brown layered CLAYEY SILT and SANDY SILT with sand seams		66.87 1.83	2	50 DO	3										
						3	50 DO	2									
3		Stiff grey layered CLAYEY SILT and SANDY SILT with sand seams		65.50 3.20	4	50 DO	1										
4						5	50 DO	1									
5			Very loose to compact layered SILTY SAND, SANDY SILT, and CLAYEY SILT		63.21 5.49												
					6	50 DO	1										
7					7	50 DO	15										
8					8	50 DO	5										
9		Compact grey SANDY SILT, some gravel, trace clay (GLACIAL TILL)		60.01 8.69	9	50 DO	27										
					10	50 DO	>50										
10	Rotary Drill NQ Core	Fresh light grey fine to medium grained SANDSTONE BEDROCK with numerous dark grey to green thinly laminated to very thinly bedded shale interbeds		59.25 9.45													
					11	NQ RC	DD	100	96	91							
11						12	NQ RC	DD	100	98	87						
12						13	NQ RC	DD	100	96	81						
13		End of Borehole		55.78 12.92													
14																	
15																	

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: T.M.S.

BOREHOLE 06-1120-203-1000.GPJ HYDROGEO.GDT 2/21/07

PROJECT: 06-1120-203

RECORD OF BOREHOLE: BH 06-105

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: August 16, 2006

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION					
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT PERCENT									
								Cu, kPa				nat V. + Q - ● rem V. ⊕ U - ○					Wp — W — Wl				
								20	40	60	80	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴			10 ⁻³	20	40	60	80
0	Power Auger 200mm Diam. (Hollow Stem)	Ground Surface		69.47																	
		Grey crushed stone (FILL)																			
		CONCRETE		0.21																	
		Brown silty sand, trace gravel (FILL)		68.77																	
1		TOPSOIL		0.70 68.46																	
		Compact brown SILTY fine SAND, occasional clayey silt seams		1.01	1	50 DO	10														
2					2	50 DO	14														
		Very stiff grey brown layered CLAYEY SILT and SANDY SILT with sand seams		67.18 2.29	3	50 DO	2														
3																					
		Stiff grey layered CLAYEY SILT and SANDY SILT with sand seams		66.24 3.23	4	50 DO	1														
4																					
5					5	50 DO	1														
6																					
					6	50 DO	1														
7		Loose grey SANDY SILT, some gravel, trace clay		62.46 7.01	7	50 DO	WH														
8		Compact grey SILTY fine SAND, occasional sandy silt layer		61.91 7.56	8	50 DO	16														
9					9	50 DO	30														
10		Compact grey SANDY SILT, some gravel, trace clay, occasional cobbles (GLACIAL TILL)		60.02 9.45	10	50 DO	66														
					11	50 DO	26														
11	Rotary Drill NQ Core	Fresh light grey fine to medium grained SANDSTONE BEDROCK with numerous dark grey to green thinly laminated to laminated shale interbeds		58.86 10.61	12	NQ RC	DD	100	95	91											
12					13	NQ RC	DD	100	98	96											
13																					
						14	NQ RC	DD	100	98	79										
14		End of Borehole		55.54 13.93																	
15																					

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: T.M.S.

BOREHOLE 06-1120-203-1000.GPJ HYDROGEO.GDT 2/21/07

PROJECT: 06-1120-203

RECORD OF BOREHOLE: BH 06-106

SHEET 2 OF 2


LOCATION: See Site Plan

BORING DATE: August 17, 2006

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT PERCENT					
								nat V. + Q - ● rem V. ⊕ U - ○									
								20	40	60	80	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴			10 ⁻³
15	Rotary Drill NQ Core	Fresh light grey fine to medium grained SANDSTONE BEDROCK with numerous dark grey to green thinly laminated to laminated shale interbeds (continued)		53.49	17	NQ RC	DD	T.C.R. (%) 100	S.C.R. (%) 97	R.Q.D. (%) 86							
16		End of Borehole		15.94													
17																	
18																	
19																	
20																	
21																	
22																	
23																	
24																	
25																	
26																	
27																	
28																	
29																	
30																	

Standpipe

Water level in
standpipe at
elev. 62.95m on
Aug. 18, 2006.

BOREHOLE 06-1120-203-1000.GPJ HYDROGEO GDT 2/21/07

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: T.M.S.

PROJECT: 971-2082

LOCATION: See Plan

SAMPLER HAMMER, 63.6kg; DROP, 760mm

RECORD OF BOREHOLE 97-1

BORING DATE: Aug. 19, 1997

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, K, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (m)	NUMBER	TYPE	SHEAR STRENGTH Cu, kPa	WATER CONTENT, PERCENT Wp — W — Wl				
0		Ground Surface	67.88								
		TOPSOIL	0.08								
		Compact brown fine to medium sand (FILL)									
1		Very loose grey silty fine sand, trace clay and gravel (FILL)	67.12 0.76	1	50 DO 4						
2				2	50 DO 3						
			65.59 2.29	3	50 DO 1						
3		Very loose grey SANDY SILT with interbedded layers of silty sand and silty clay		4	50 DO WH						
4				5	50 DO WH						
5				6	50 DO WH						
6			61.93 5.95	7	50 DO 1						
		Loose to compact grey silty sand with gravel, trace clay and occasional cobbles (GLACIAL TILL)		8	50 DO 8						
7				9	50 DO 32						
		End of Hole	60.41 7.47								
8											
9											
10											

DATA INPUT: O 97-1-082.d/S.L

Bentonite Seal

 Native Backfill

 W.L. in Screen at Elev. 65.32m Sept. 4, 1997

 Top of pipe at Elev. 68.72m

Granular Filter

 38mm PVC #10 Slot Screen

 MH

W.L. in Screen at Elev. 65.32m Sept. 4, 1997

 Top of pipe at Elev. 68.72m

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: S.B

CHECKED:

PROJECT: 971-2082

RECORD OF BOREHOLE 97-2

SHEET 1 OF 1

LOCATION: See Plan

BORING DATE: Aug. 19, 1997

DATUM: Geodetic

SAMPLER HAMMER, 63.6kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, K, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION			
		DESCRIPTION	STRATA PLOT	ELEV.	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa			nat.V - + rem.V - ⊕ Q - ● U - ○	WATER CONTENT, PERCENT Wp — W — Wl 20 40 60 80	
				DEPTH (m)									
0	Power Auger 200mm Diam (Hollow Stem)	Asphalt Surface		88.27							Asphalt Seal		
ASPHALTIC CONCRETE		0.09										Bentonite Seal	
		Compact brown sand and gravel (FILL)		87.68 0.61									
1		Loose brown and grey fine sand, some silt, trace clay and gravel (FILL)			1	50 DO	7					Native Backfill	
2					2	50 DO	5						
3			Very loose grey SANDY SILT with interbedded layers of grey silty sand and silty clay		85.98 2.29	3	50 DO	2					
4				4	50 DO	2							
5				5	50 DO	WH							
				6	50 DO	WH							
6				7	50 DO	1							
		Loose fine SAND, trace silt and gravel		62.17 6.10	8	50 DO	4						Granular Filter
7				61.41 6.66	9	50 DO	7						
		Loose fine to medium SAND, trace silt											
8				60.03 8.24	10	50 DO	5						
		End of Hole											
9													
10													

DATA INPUT: 037-2-082 dr/S.L

W.L. in
Standpipe at
Elev. 65.10m
Sept. 19, 1997

DATA INPUT: O-97-2-082.drl/S/L

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: S.B

CHECKED:

SAMPLER HAMMER, 63.6kg; DROP, 760m

BORING DATE: Aug. 19, 1997

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k_v , cm/s	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV.	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH Cu, kPa	WATER CONTENT, PERCENT Wp — W — Wi
				DEPTH							
				(m)							
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface		88.80							
		TOPSOIL		0.00							
				0.18							
		Dense brown and grey fine to medium sand with cobbles (FILL)							Bentonite Seal		
1					1	50	52			Native Backfill	
				67.28							
				1.52							
2					2	50	18				
					3	50	17				
3			Loose to compact grey silty sand with gravel, trace clay (GLACIAL TILL)								
					4	50	6				MH
4											
					5	50	8				
				6	50	7					
5											
				7	50	8				MH	
6											
			8	50	25						
7											
			9	50	>50						
		End of Hole Auger Refusal		61.85							
				6.95							
8											
9											
10											

DATA INPUT: 037-3-082.drt/s L

W.L. in
Screen at
Elev. 65.95m
Sept. 4, 1997

Top of pipe
at Elev. 68.80m

DATA INPUT: O:07-3-002.d#1/S.L

1 to 50

Golder Associates

CHECKED: **MLC**

PROJECT: 971-2082

LOCATION: See Plan

SAMPLER HAMMER, 63.6kg; DROP, 760mm

RECORD OF BOREHOLE 97-4

BORING DATE: Aug. 18&19, 1997

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, K, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION								
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa		WATER CONTENT, PERCENT													
								nat.V - + rem.V - @	Q - ● U - ○	Wp	W												
0	Power Auger 200mm Diam (Hollow Stem)	Asphalt Surface		88.51										Asphalt Seal									
		ASPHALTIC CONCRETE		0.00											Native Backfill								
		Compact sand and gravel (FILL)		0.12												MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
1																		Asphalt Seal					
		Loose to compact black and grey silty sand, trace clay and gravel (FILL)		67.90	1	50 DO	13												Native Backfill				
				0.81																MH			
2																					W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997		
																						Asphalt Seal	
		Gray layered SANDY SILT with interbedded layers of grey silty sand and silty clay		66.22	2	50 DO	5																Native Backfill
			2.29										MH										
3														W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
			65.46	3	50 DO	3									Asphalt Seal								
			3.05													Native Backfill							
4																	MH						
	Compact grey silty sand with gravel, trace clay, occasional cobbles (GLACIAL TILL)			4	50 DO	10												W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997					
																			Asphalt Seal				
5																				Native Backfill			
																					MH		
																						W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997	
													Asphalt Seal										
														Native Backfill									
6	Rotary Drill NO Core	Slightly weathered to fresh light grey fine to medium grained very thinly to medium bedded SANDSTONE with numerous grey green fine grained thinly laminated to very thinly bedded shale interbeds. Mud seam @6.52 to 6.59m depth (ROCKCLIFFE FORMATION)	63.02	5	50 DO	26									Bentonite Seal								
				5.49	6	50 DO	23																
																							Standpipe
7						7	50 DO	> 100										W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997					
																			Asphalt Seal				
															Native Backfill								
8				8	NO RC	1	T.C.R. 100% S.C.R. 78.6% R.Q.D. 71.4%									MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									
															MH								
9				9	NO RC	1	T.C.R. 100% S.C.R. 78.3% R.Q.D. 63.3%									W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997							
																	Asphalt Seal						
													Native Backfill										
														MH									
															W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997								
																Asphalt Seal							
																	Native Backfill						
													MH										
10				10	NO RC	1	T.C.R. 100% S.C.R. 65% R.Q.D. 51.7%							W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
															Asphalt Seal								
																Native Backfill							
																	MH						
													W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997										
														Asphalt Seal									
															Native Backfill								
																MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									
															MH								
																W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997							
																	Asphalt Seal						
													Native Backfill										
														MH									
															W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997								
																Asphalt Seal							
																	Native Backfill						
													MH										
														W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
															Asphalt Seal								
																Native Backfill							
																	MH						
													W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997										
														Asphalt Seal									
															Native Backfill								
																MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									
															MH								
																W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997							
																	Asphalt Seal						
													Native Backfill										
														MH									
															W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997								
																Asphalt Seal							
																	Native Backfill						
													MH										
														W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
															Asphalt Seal								
																Native Backfill							
																	MH						
													W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997										
														Asphalt Seal									
															Native Backfill								
																MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									
															MH								
																W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997							
																	Asphalt Seal						
													Native Backfill										
														MH									
															W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997								
																Asphalt Seal							
																	Native Backfill						
													MH										
														W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
															Asphalt Seal								
																Native Backfill							
																	MH						
													W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997										
														Asphalt Seal									
															Native Backfill								
																MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									
															MH								
																W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997							
																	Asphalt Seal						
													Native Backfill										
														MH									
															W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997								
																Asphalt Seal							
																	Native Backfill						
													MH										
														W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
															Asphalt Seal								
																Native Backfill							
																	MH						
													W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997										
														Asphalt Seal									
															Native Backfill								
																MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									
															MH								
																W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997							
																	Asphalt Seal						
													Native Backfill										
														MH									
															W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997								
																Asphalt Seal							
																	Native Backfill						
													MH										
														W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997									
															Asphalt Seal								
																Native Backfill							
																	MH						
													W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997										
														Asphalt Seal									
															Native Backfill								
																MH							
																	W.L. in Standpipe at Elev. 64.97m Sept. 19, 1997						
													Asphalt Seal										
														Native Backfill									

DATA INPUT: 0:97-4-982.07/S L

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: S.B

CHECKED: MIC

PROJECT: 971-2082

LOCATION: See Plan

SAMPLER HAMMER, 63.6kg; DROP, 760mm

RECORD OF BOREHOLE 97-5

BORING DATE: Aug. 19, 1997

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, K, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT, PERCENT	
								Cu, kPa	nat.V - + rem.V - @ Q - ● U - ○			Wp	W
0	Power Auger 200mm Diam (Hollow Stem)	Asphalt Surface		68.65							Asphalt Seal Bentonite Seal		
		ASPHALTIC CONCRETE		0.00									
		Compact brown sand and gravel (FILL)		0.15									
				68.10 0.55									
1		Compact brown fine to medium sand, trace gravel (FILL)			1	50 DO	14					Native Backfill	
				87.13 1.52									
2		Compact dark grey mottled silty sand, trace clay and gravel (GLACIAL TILL)			2	50 DO	3						
				66.36 2.29									
3		Loose to dense grey silty sand with gravel, occasional cobble, trace clay (GLACIAL TILL)			3	50 DO	6						
4				4	50 DO	30							
5					5	50 DO	31						
6	Rotary Drill NQ Core	Slightly weathered to fresh light grey fine to medium grained very thinly to medium bedded SANDSTONE with numerous grey green fine grained thinly laminated to very thinly bedded shale interbeds. Mud seam @ 7.70 to 7.77m depth (ROCKCLIFFE FORMATION)		63.77 4.88						Bentonite Seal			
					6	50 DO	>50						
7					7	NQ RC	-	T.C.R. 100% S.C.R. 83.3% R.Q.D. 78.6%					
											Granular Filter		
8				8	NQ RC	-	T.C.R. 100% S.C.R. 88.7% R.Q.D. 78.3						
9				9	NQ RC	-	T.C.R. 100% S.C.R. 78.5% R.Q.D. 75%				Standpipe		
		End of Hole											
10				60.51 8.14							Screen Dry to Elev.60.41m Sept. 19, 1997		

DATA INPUT: O.97-5-082.d/S.L

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: S.B

CHECKED: MIC

PROJECT: 971-2082

LOCATION: See Plan

SAMPLER HAMMER, 63.6kg; DROP, 760mm

RECORD OF BOREHOLE 97-6

BORING DATE: Aug. 25, 1997

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 780mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, K, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa nat.V. + rem.V. -	Q - ● U - ○	WATER CONTENT, PERCENT Wp W Wl 20 40 60 80		
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface	68.27								Bentonite Seal
		TOPSOIL	0.00								
			0.15								
1		Loose brown and grey fine sand, some sand and gravel (FILL)		1	SO	7					
			66.90								
2	Rotary Drill HQ Core	Loose brown silty sand, some gravel, trace clay (GLACIAL TILL)	1.37	2	SO	6					Native Backfill
			65.98								
			2.29								
3		Loose to compact grey silty sand with gravel, trace clay (GLACIAL TILL)		3	SO	7					
			64.12								
4	Rotary Drill HQ Core		4.15	4	SO	9					Granular Filter
				5	SO	26					
				6	HQ RC		T.C.R. 70% S.C.R. 20% R.Q.D. 0%				
5		Slightly weathered to fresh light grey fine to medium grained very thinly to medium bedded SANDSTONE with numerous grey green fine grained thinly laminated to very thinly bedded shale interbeds. (ROCKCLIFFE FORMATION)		7	HQ RC		T.C.R. 98.3% S.C.R. 88.3% R.Q.D. 88.6%				
			62.08								
6		End of Hole	6.19								38mm PVC #10 Slot Screen
7											
8											
9											
10											

DATA INPUT: O:97-6-082.d/S/L

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: S.B

CHECKED: m/c

PROJECT: 971-2082

LOCATION: See Plan

SAMPLER HAMMER, 63.6kg; DROP, 760mm

RECORD OF BOREHOLE 97-7

BORING DATE: Aug. 25, 1997

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k _t cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH C _u , kPa	nat.V - + rem.V - ⊕			Q - ● U - ○	WATER CONTENT, PERCENT W _p — W — W _t 20 40 60 80
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface		68.07							M	<div>Bentonite Seal</div> <div>Native Backfill</div> <div>Granular Filter</div> <div>38mm PVC #10 Slot Screen</div> <div>Bentonite Seal</div>	
		TOPSOIL		0.00 0.09									
1		Compact brown fine to medium sand (FILL)			1	50 DO	14						
2	Rotary Drill HQ Core	Very dense brown and grey silty sand and gravel (FILL)		66.55 1.52 66.09 1.98	2	50 DO	59		○				
3		Loose to very loose grey sand and gravel, occasional cobbles and shale fragments (GLACIAL TILL)			3	50 DO	>50						
4	Rotary Drill HQ Core			64.35 3.72	4	50 DO	8		○				
5		Slightly weathered to fresh light grey fine to medium grained very thinly to medium bedded SANDSTONE with numerous grey green fine grained thinly laminated to very thinly bedded shale interbeds. (ROCKCLIFFE FORMATION)			5	HQ RC	-	T.C.R. 97.1% S.C.R. 85.3% R.Q.D. 85.3%					
6	Rotary Drill HQ Core			62.09 5.98	6	HQ RC	-	T.C.R. 100% S.C.R. 85.2% R.Q.D. 79.6%					
7		End of Hole											
8													
9													
10													

DATA INPUT: C:\97-7-082.ct\SL

W.L. in Screen at Elev.65.67m Sept. 4, 1997

Top of pipe at Elev.68.80m

DATA INPUT: O:97-7-082.d/S.L

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: S.B

CHECKED:

APPENDIX B

RECORD OF BOREHOLE SHEETS PREVIOUS INVESTIGATION BY OTHERS

RECORD OF BOREHOLE No 89-1

METRIC

W P 120-87-00 LOCATION Co-Ord. N 5,022,910.0; E 359,479.0 (HML C-1) ORIGINATED BY R.H.
 DIST 9 HWY 417 N 416 BOREHOLE TYPE Hollow Stem Auger, BX Rock Core COMPILED BY R.H.
 DATUM Geodetic DATE July 11, 1989 CHECKED BY G.J.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100			
68.0	Ground Surface													GR SA SI CL
0.0	Fill, Sand and Silt trace organics: Brownish grey		1	SS	5	67								
66.5	Loose													
1.4	Sandy, Gravelly, Silt to Sand, traces of silt and gravel. Light grey. Loose to dense		2	SS	9	66								
			3	SS	40	63								31 28(41)
63.2			4	SS	20/1 Scm	64								
4.8	Bedrock Shale with lenses of Sandstone. Grey to light grey Very poor to good		5	RC REC BX 94Z		63								RQD=0%
			6	RC REC BX 100Z		62								*(27) RQD=88%
			7	RC REC BX 96Z		61								RQD=54%
60.2														
7.3	End of Borehole *q _u (MPa) obtained from point load test.													

OFFICE REPORT ON SOIL EXPLORATION

BOREHOLE RECORD

94-4

Sta. 7+341 6.0m RT

CLIENT McCormick Rankin Consulting Engineers

PROJECT No. 10494

LOCATION West Transitway

BOREHOLE No. 94-4

DATES: BORING 94-06-28

WATER LEVEL 94-07-13

DATUM Geodetic

ELEVATION (m)	SOIL DESCRIPTION	STRATA	PLOT	WATER LEVEL	SAMPLES				UNDRAINED SHEAR STRENGTH - kPa									
					TYPE	NUMBER	RECOVERY	N-VALUE OR Rqd	50	100	150	200	WATER CONTENT & ATTERBERG LIMITS					
67.69							mm											
67.5	150 mm Asphalt				SS	1	325	6										
67.4	Loose, grey sand and gravel: BASE																	
66.5	Very loose to loose, brown to grey SILTY SAND, trace black organics				SS	2	400	9										
	Very loose, grey SILTY SAND, trace to some clay				SS	3	600	2										
					SS	4	600	2										
					SS	5	600	3										
					SS	6	600	3										
					SS	7	600	2										
					SS	8	600	2										
					SS	9	600	3										
					SS	10	600	2										



Proposed Pipe Invert

Continued Next Page

- Field Vane Test, kPa
- Remoulded Vane Test, kPa
- Pocket Penetrometer Test, kPa





APPENDIX C

RESULTS OF CHEMICAL ANALYSIS ACCUTEST REPORT No. A7-6213

