REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
MONTREAL RIVER BRIDGE REPLACEMENT AT MATACHEWAN
HIGHWAYS 66 & 566
G.W.P 83-86-00, SITE NO. 47-017
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 14, NEW LISKEARD

Submitted to:

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GEOCRES NO. 41P-35

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January 2006 04-1111-056

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Lea Consulting Ltd. (Lea) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detailed design of the bridge structure replacing the existing structure carrying Highways 66 and 566 across the Montreal River in Matachewan, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P41-1530, dated August 12, 2004, that forms part of the Consultant's Agreement (Number P.O. 5005-A-000428) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated November 22, 2004. The general arrangement drawing for the bridge structure at Highways 66 and 566 was provided to Golder by Lea in August 2005 and updated in November 2005, when a retaining wall was added to the northeast corner of the bridge structure.

2.0 SITE DESCRIPTION

The site is situated about 60 m north (upstream) of the existing Highway 66/566 structure that crosses the Montreal River in Matachewan, Ontario. The proposed bridge site is located in the Montreal River valley whose side slopes reach up to about 18 m in height across the valley. The area immediately surrounding the bridge site is undulating with rocky outcrops and forested areas to the north and west of the site. The surrounding land is mainly used for commercial and residential development with grass and tree cover extending beyond the limits of the town. The river is used mainly for recreation and is approximately 90 m wide at the crossing location.

The existing bridge, located 60 m downstream, was built in 1936 and consists of a timber beam structure supported on eight timber pile bents.

The ground surface on the banks of the river is at about Elevation 302 m; Riverside Drive runs parallel to the river on the east side. The river banks are about 2 m in height and slope down to the river's edge. The west bank is vegetated with mature trees and the east bank is vegetated with grass. The water level in the river was measured at Elevation 299.5 m (December 2004) and the high water level is reportedly Elevation 301.7 m as indicated on the General Arrangement drawing. The lowest water level during the period of this subsurface investigation was measured in August 2005 at about Elevation 298.9 m.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work at the bridge site was carried out in three stages: between July 20 and July 27, 2005, eight (8) boreholes, numbered 1 to 8, were advanced on land; between August 23 and August 25, 2005, one (1) additional borehole, number 9, was advanced within the water channel; and on November 30, 2005, BH 10 was advanced in the general location of the retaining wall. The locations of these boreholes are shown on Drawing 1 and noted on the respective borehole logs.

The on-land portion of the field investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Ltd. (Marathon) of Ottawa, Ontario. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers as well as wash boring methods using ‘NW’ casing. Tri-cone methods were used to advance the boreholes at some locations. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures with an automatic hammer. Dynamic Cone Penetration Testing was carried out in selected boreholes when auger and casing advance was terminated in the boreholes. Samples of the boulders in Borehole 1 below a depth of about 26.5 m below existing ground surface were obtained using an ‘NQ’ size rock core barrel.

The on-water portion of the field investigation was carried out using raft-mounted portable drilling equipment also supplied and operated by Marathon. The borehole was advanced by wash boring methods using NW and BW casing with further borehole advance using BQ size rock core barrel. Soil samples were obtained at random intervals ranging from 0.15 m to 1.1 m in depth, from the tip of the core barrel and/or casing. A single sample at a depth of about 3.2 m below water surface was obtained using a 50 mm outer diameter (O.D.) split-spoon sampler using a half-weight (i.e., 70 lb.) manual hammer. The ‘N’ value presented in the borehole log was converted to be in general accordance with SPT procedures.

The boreholes on-land were advanced to depths ranging from 1.2 m (Borehole 8) to 31.4 m (Borehole 1) below the existing ground surface. The borehole advanced in the river channel was terminated at a depth of approximately 6.4 m below water surface due to the presence of cobbles and boulders and the limited power of portable drilling equipment.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in selected boreholes to permit monitoring of the groundwater level at these locations. The piezometers consist of a 50 mm outside diameter rigid PVC tubing with a 1.5 m long slotted screen that is sealed at a selected depth within the boreholes. The boreholes and piezometers, after the last water level was obtained, were backfilled with bentonite and/or cement bentonite grout as per Ontario Reg. 128 (amendment to O. Reg. 903). The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content and grain size distribution) was carried out on selected samples.

The proposed locations of the foundation elements were surveyed and laid out in the field by Lea using the NAD 83 MTM co-ordinate system and the geodetic datum for elevation. The northings, eastings and elevations of the as-drilled boreholes were measured in the field relative to the staked locations by members of Golder. The borehole in the river was referenced to the river water level at the time of drilling, which was referenced to the stakes surveyed by Lea, located at the river's edge.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

From published geologic information, the site is located in the physiographic region known as the Cobalt Plain that forms part of the Canadian Shield (Ontario Geological Survey, Northern Ontario Engineering Geology Terrain Study 83, Elk Lake Area, 1979). The area is subdivided into clayey and sandy lowlands interrupted by rocky uplands. Most uplands are marked by prominent north-trending patterns of narrow valleys that are controlled by faults, such as the Montreal River Fault system. The surficial geology in the immediate area of the Montreal River valley indicates that the valley was filled with glacial sand, gravel and boulders. This valley was infilled by glaciofluvial deposits, forming an outwash plain of up to 60 m in thickness.

The site is located in the geologic region known as the Abitibi Greenstone belt that forms the largest greenstone belt within the Canadian Shield (Geology of Ontario; OGS Special Volume 4). Faults and joints in the bedrock are numerous and the presence of many faults is reflected in the terrain. While the geology of the Abitibi Greenstone region is complex, the bedrock geology of the immediate area of the Montreal River Bridge is characterized by intrusive rocks of the Precambrian era, including granodiorite and monzonite and geophysical interpretations indicate the existence of diabase dykes.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the bridge location is shown on Drawings 1 and 2.

In general, the subsoils at the site consist of localized surficial layers of sand and/or fill underlain by an extensive deposit of sand and gravel containing cobbles and boulders. Occasional seams of sand and silty sand were encountered within the deposit. In all of the boreholes, the presence of cobbles and boulders within the sand and gravel deposit made it difficult to advance the boreholes. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Sand to Sand and Gravel (Fill)

Fill consisting of sand to sand and gravel was encountered at the existing ground surface in Boreholes 2, 3 and 4. The surface of the fill ranges between Elevation 302.0 m and 302.5 m and the fill thickness ranged from 1.5 m to 4.3 m. Trace rootlets and occasional cobbles were noted within the fill materials. In some instances, the sample hammer was observed to be bouncing during sampling.

SPT ‘N’ values measured within the fill ranged between 7 and 40 blows per 0.3 m of penetration, indicating a loose to dense relative density. The grain size distribution curve for a select sample obtained from Borehole 2 is shown on Figure 1.

The natural water content measured on samples of the fill was about 4 percent.

4.2.2 Sand

A deposit of sand was encountered at the existing ground surface in Boreholes 1 and 7. The surface of the sand deposit ranges between Elevation 302.5 m and 302.0 m and the thickness of the sand deposit varied between 1.3 m and 1.4 m, respectively. Within this deposit, trace gravel was noted in Borehole 1 and occasional cobbles were noted in Borehole 7; trace organics were noted in both boreholes.

SPT ‘N’ values measured within the sand deposit were 8 and 6 blows per 0.3 m of penetration, indicating a loose relative density.

4.2.3 Sand and Gravel containing Cobbles and Boulders

A deposit of sand and gravel was encountered either at the existing ground surface or below the fill or sand deposit in Boreholes 1 to 8 and 10. In Borehole 9, this deposit was encountered below the river bed cobble and boulder deposit (refer to Section 4.2.4). All boreholes were terminated within this deposit, which was proven to a thickness of 29.9 m in Borehole 1. Cobbles and/or boulders were encountered in each of the boreholes within the sand and gravel deposit as indicated by resistance to augering, casing advance, dynamic cone penetration as well as by fragments obtained in split spoon samples and the core barrel. The surface of the deposit ranges from Elevation 298.2 m to 310.6 m.

At the borehole locations, measured SPT ‘N’ values ranged between 9 and 119 blows per 0.3 m of penetration, indicating a loose to very dense relative density. Typically, the deposit is compact to dense with ‘N’ values ranging between 20 and 40 blows per 0.3 m of penetration. As noted in Section 4.2.5, in Boreholes 1, 4 and 5 sand and gravel was noted to flow into the hollow-stem augers and/or casing due to water pressures and this required an external source of water to be pumped into the borehole to maintain a constant head of water in order to obtain SPT values. The first sample taken upon reaching the high water pressures zones may have been impacted (i.e., lower) than the actual SPT 'N' value, although the 'N' values were not inconsistent with 'N' values recorded at the same elevation in these three boreholes. Grain size distribution curves for samples of the sand and gravel deposit are shown on Figure 2.

Sand and silty sand seams were encountered within this deposit in Boreholes 4, 5, and 6. In general, these seams were less than 1.2 m in thickness and were encountered near the top of the deposit, although in Borehole 1 a sand seam was encountered at 23.9 m of depth, and was approximately 1.5 m in thickness. Grain size distribution curves for samples of the sand and silty sand seams are shown on Figure 3.

The natural water content measured on samples of the sand and gravel deposit range from 1 to 24 percent, with a typical range between 4 and 15 percent.

Based on recovery of casing/core samples and the observations of the drill and sampler operations, the cobbles and boulders fraction are estimated to make up between 10 and 30 percent of the deposit. Although it was possible to obtain SPT samples from within this deposit, it was noted that in some instances the sample hammer was bouncing on an inferred cobble/boulder. Boulders up to 750 mm diameter were present during the investigation as cored near the bottom of Borehole 1. The depths and corresponding elevations at which the hollow stem augers could not penetrate further in the boreholes due to either the cobbles and boulders or heaving sands are summarized in the table below. Boreholes 1, 2, 4 and 5 were continued with casing after difficulty during augering was encountered. Boreholes 6, 7 and 8 were continued using a Dynamic Cone Penetration Test. Additional drilling observations and comments are summarized in Table 1, following the text of this report.

Auger Refusal		
Borehole	***Depth (m)***	***Elevation (m)***
1	15.2	287.5
2	2.7	299.8
4	4.6	297.4
5*	10.5	295.3
6	4.4	300.2
7	3.7	298.3

Auger Refusal		
Borehole	***Depth (m)***	***Elevation (m)***
8	1.2	309.3
9	n/a	n/a
10	n/a	n/a

*Sand heaving inside augers (refusal to further auger penetration)

Photos of the site conditions are presented on Figures 4 and 5 showing evidence of boulders at the ground surface at the west pier and west abutment. Based on visual observations of the gravel, cobbles and boulder fragments obtained from the borehole samples, the rock fragments encountered consisted of different types of granite, diorite, gabbro, siltstone, diabase, and quartz.

Dynamic Cone Penetration Testing (DCPT) was carried out in the deposit in Boreholes 2, 4, 5, 6 and 7. Increased resistance to penetration with depth in each of the DCPTs suggests that the deposit may be laterally extensive beneath the site. In several instances, the DCPT results were greater than 100 blows per 0.3 m of penetration, indicative of cobbles and boulders in the deposit. The dynamic cones were terminated at depth where greater than about 1 m of 100 blow material or refusal was encountered.

4.2.4 Cobbles and Boulders

As indicated previously, Borehole 9 was advanced within the river channel. The river bed was 0.6 m below the water surface at the time of drilling. From the river bed surface, at Elevation 298.3 m, cobbles and boulders were encountered for the upper 2.6 m of the borehole. The cobbles and boulders were surrounded by a sand and gravel matrix which was evident based on examination of the wash water. The bottom 0.8 m of the deposit was considered to consist of gravel and cobbles containing some sand. Casing advance was very slow and difficult in the borehole due to gravel and/or cobble fragments continuously becoming stuck in the casing and core barrel. The borehole was advanced an additional 3.2 m by wash boring and coring methods through the sand and gravel deposit containing cobbles and boulders and was terminated due to coring/casing refusal.

Based on visual observations of the gravel, cobbles and boulder fragments obtained from the samples in Borehole 9, the rock types are generally consistent with those found in Boreholes 1 to 8, consisting of granite, diorite, gabbro, siltstone, diabase, and quartz.

4.2.5 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed in Boreholes 1 and 5. Both piezometers have their screened zones within the sand and gravel deposit. Details of the piezometer installations are shown in the Record of Borehole Sheets following the text of this report. The water levels in the piezometers and open holes upon completion of drilling are summarized in the table below:

Location	*Bore-hole*	*Installations*	*Ground Surface Elevation (m)*	*Water Level Depth (m)*	*Water Level Elevation (m)*	*Date*
West Approach	3	Open Borehole	302.5	Dry	-	Upon Completion of Drilling
West Abutment	1	Piezometer	302.7	4.3	298.4	August 23, 2005
	2	Open Borehole	302.5	3.2	299.3	Borehole caved to this depth.
West Pier	9	Open Borehole	298.9*	0	298.9	August 23, 2005
East Pier	4	Open Borehole	302.0	3.4	298.6	Upon Completion of Drilling
	7	Open Borehole	302.0	n/a	-	-
East Abutment	5	Piezometer	305.8	7.5	298.3	August 23, 2005
	6	Open Borehole	304.6	n/a	-	-
East Approach	8	Open Borehole	310.5	n/a	-	-
Northeast Retaining Wall	10	Open Borehole	310.6	n/a	-	-

* Borehole 9 was drilled from a raft in the river channel. Water surface is the reference elevation.

In general, the samples taken in the boreholes were noted to be moist to wet with free water evident within several sand to sand and gravel samples. In Boreholes 1, 4 and 5, sand and gravel was noted to flow into the hollow-stem augers and/or casing due to water pressures and this required an external source of water to be pumped into the borehole to maintain a constant head of water in order to obtain SPT samples. In Borehole 9, water pressures were encountered during casing advance.

The above groundwater levels are consistent with the adjacent river water level. The water level in the river was measured at Elevation 299.5 m (December 2004) and the high water level is Elevation 301.7 m as noted in the General Arrangement drawing. The lowest water level during the period of this subsurface investigation was measured in August 2005 at about Elevation 298.9 m. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.3 Closure

The field technician supervising the land portion of this drilling program was Mr. Suresh Bainey of Golder's Mississauga office. The field engineer for the in-water portion of this drilling was Mr. André Bom of our Sudbury office. This report was prepared by Miss Sarah Poot, P.Eng., a senior geotechnical engineer; the technical aspects were reviewed by and quality control provided by Mr. Jorge Costa, P. Eng., a Designated MTO Contact for Golder.

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PART B

FOUNDATION DESIGN REPORT
MONTREAL RIVER BRIDGE REPLACEMENT AT MATACHEWAN
HIGHWAYS 66 & 566
G.W.P 83-86-00, SITE NO. 47-017
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 14, NEW LISKEARD

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed bridge structure carrying the re-aligned Highway 66/566 over the Montreal River. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

The bridge is proposed to be a three-span structure, 140.8 m in total length and 11.6 m in width. The proposed embankments are between 3 m and 6 m in height above the existing ground surface. The grade of the proposed Hwy 66/566 bridge is between Elevation 305.5 m and 310.5 m, while the existing ground surface varies from about Elevation 302 m to 311 m and the river bed is at about Elevation 298.3 m. The river level during the drilling in August 2005 was at Elevation 298.9 m, some 0.6 m lower than that measured in December 2004 as shown on the General Arrangement drawing provided to Golder by Lea.

5.1 General Bridge Foundation Options

It is understood that consideration is being given to the use of an integral abutment configuration for support of the bridge. In this case, a single row of vertical piles would be required for support of the abutments. For the piers, different foundation alternatives would need to be considered. For this site, semi-integral abutment bridge construction should be considered in combination with spread footings, caissons and/or piles for the pier, as discussed in the following sections.

The subsoils at this site consist of sand and gravel containing cobbles and boulders. The extent of cobbles and boulders appears to be much greater within the river channel (west pier) compared to on the land (east pier and abutment locations). The presence of cobbles and boulders should be taken into consideration for both the design of the foundations as well as the constructability of the foundation elements.

In general, driven piles are feasible at the east pier and the abutments. However, at the west pier, where the presence of cobbles and boulders significantly impacted the advance of the borehole, driven piles are not as technically feasible unless special techniques are employed to advance the piles through the upper portion of the granular deposit containing numerous boulders and cobbles.

Specialized construction techniques such as pre-drilling and casing advance in the upper 4 m of the deposit will likely be required to install the piles at the west pier.

Caissons are technically feasible at this site, however, the cobbles and boulders, as well as the high groundwater pressures, would require specialized construction techniques.

Spread footings may be considered at this site for semi-integral abutments or the piers, but other considerations such as scour and erosion protection measures will be required.

A summary of the advantages, disadvantages, relative costs and risks/consequences for each of the foundation alternatives is given in Table 1 following the text of this report. The preferred alternative from a foundation point of view is to drive piles. A brief discussion of the founding alternatives at the foundation elements is presented below.

5.2 Shallow Foundations

It is understood from Lea that there is a high potential for scour at this site due to the fast flowing river current in this area and that the scour elevation extends some 2.4 m below the river bed. For this reason, shallow spread footings for the piers may not be feasible due to the need to adequately control the groundwater in order to excavate a deeper/larger dry foundation area below the scour elevation and the presence of numerous cobbles and boulders within the river channel which restrict the drivability of closed sheet-piling. However, special techniques such as line drilling will facilitate the installation of a sheeted cofferdam (as discussed in Section 5.9). Such measures are not required if the footings for the abutments can be perched within the embankment above the water level.

If spread footings are used at the abutments and the piers are founded on deep foundations, the anticipated differential settlement would not exceed 25 mm. The benefit of using shallow spread footings at the abutments is that it eliminates the need to advance foundations through the bouldery subsoils at these locations.

5.2.1 Geotechnical Resistance

Spread footings placed on undisturbed sand and gravel may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa, assuming a footing width of 3 m. A geotechnical resistance at Serviceability Limit States (SLS) of 300 kPa may be used for 25 mm of settlement. The settlement of the abutment footings, and hence the SLS capacity, will be governed by the embankment loading.

The footing should be maintained as high as possible and should not be any lower than Elevation 300.5 m (1 m above the typical river water level to avoid dewatering requirements). In this regard, construction should be carried out at times of low water level and scour protection measures must also be provided. We understand that this founding elevation is not feasible with the currently proposed vertical profile of the bridge structure.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary.*

5.2.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the sand and gravel deposit should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ, may be taken as 0.62 between the base of the concrete footings and the properly prepared sand and gravel subgrade. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.2.3 Frost Protection

The footings should be provided with 2.2 m of soil cover (above the base of the footing) to protect against frost.

5.3 Steel H-Pile Foundations

Two different piling alternatives can be considered for the bridge foundations depending on the structural requirements of the bridge (i.e., integral vs. semi-integral, etc.) and the construction considerations at this site. Consideration could be given to the use of HP310x132 piles, which would be more damage resistant in the bouldery soils, or standard HP310x110 piles. If an integral abutment design is being considered, the HP310x132 piles may not provide adequate flexibility in the upper zone, although pile sections as heavy as HP310x152 have been used in integral abutment design. Further discussion on the advantages and disadvantages of each pile type is given below.

We recommend that steel HP310x132 piles be driven from inside a pre-augered/drilled cased hole extending through the upper 4 m of the sand and gravel containing boulders and cobbles. This pre-augered/drilled hole is intended to provide a guide for the pile and to prevent deflection on boulders within this zone. These heavier pile sections will reduce the potential for damage to the pile during driving through the sand and gravel deposit containing cobbles and boulders. In addition, the pile tips should be reinforced with Titus Rock Injector points, or equivalent. The appropriate NSSP for pile tip protection should be included in the Contract Documents; an example is included in Appendix A for reference.

Alternatively, if a standard (i.e., lighter) pile section is desired, such as HP310x110, it will be necessary to pre-drill a cased hole (especially at the west pier and west abutment) to a minimum of 1 m above the design tip elevation, into which the pile could be inserted and driven for the last 1 m to seat the pile into the overburden.

The following section of the report discusses construction aspects regarding driving of piles through the bouldery soils at this site, which should be taken into account during the design of the piles considering the flexibility in pile location, the reduced pile capacity due to boulder interference and/or pile damage and contractual issues.

5.3.1 Construction Considerations

Due to the presence of cobbles and boulders within the sand and gravel deposit at this site, pile driving using conventional construction techniques may prove problematic in the field (as indicated by the difficulty in advancing the foundation boreholes using conventional drilling techniques).

For the alternative of using HP310x132 piles, it is recommended that the sand and gravel deposit containing cobbles and boulders be pre-augered/drilled with a 600 mm I.D. casing to 4 m below the elevation of the intended start of pile driving, to assist the piles to maintain their batter and direction during driving by ensuring that the pile does not get deflected by a cobble/boulder in the first few metres of driving. The hole should be cased and pre-augered/drilled and this zone should be backfilled with sand once the piles have been driven to the design elevation. Alternatively, the holes could be backfilled and the casing removed prior to pile driving. The backfill should be placed in 1 m layers and rodded in place or placed hydraulically to reduce the void space. The sand backfill should meet the gradation given in Table 1 of OPSS 1002 (April 2004) for fine aggregate for concrete. The Contractor must be prepared and have appropriate equipment on site to install the above noted casing, on a batter if required, and be able to penetrate through cobbles and boulders as the casing is installed. We understand that Special Provision SP903S01 is being modified by Lea to include appropriate language.

For the alternative of using HP310x110 piles, a pre-drilled hole will be required along essentially the full length of the pile for approximately a 20 m depth. Specialized dual rotary systems have been successfully employed on other MTO projects, to advance cased holes in similar soil deposits. "Symmetrix (ROTEX)"and "ODEX" are two such systems that are available in Ontario, however, the cost of installing piles using this system could be about twice that of conventional pile driving. In addition, the cased hole would have to be backfilled with concrete as the casing is removed using tremie methods below the groundwater level. If this alternative is chosen, a special provision would be required in this regard. It should be noted that these systems are air track driven, and create significant noise, dust and cuttings during drilling; however, measures can be taken to reduce these impacts.

5.3.2 Axial Geotechnical Resistance

As noted in Section 5.3.1, the presence of cobbles and boulders within the subsurface soils at this site prevented auger advance below depths for which refusal to split spoon sampling was not recorded. Dynamic Cone Penetration Testing was advanced in selected boreholes (e.g., BH 4 to BH 7) to depths below the auger refusal and terminated within the deposit indicating greater than 100 blows per 0.3 m of penetration.

The piles should be terminated within the dense, lower portion of the sand and gravel deposit within which the Dynamic Cone Penetration Testing indicated greater than 100 blows per 0.3 m of penetration. Estimated design tip elevations for driven piles at this site are given in the table below. Also given in the table are the minimum (i.e., highest) acceptable pile tip elevations. These design elevations should be incorporated into the contract drawings in an appropriate note.

Foundation Location	*Design Tip Elevation in Sand and Gravel Deposit (m)*	*Acceptable Range of Final Pile Tip Elevations (m)[1,2]*
West Abutment	273	273 to 276
West Pier (1)	274	274 to 278
East Pier	275	275 to 277
East Abutment	277	277 to 281

Notes: 1. Elevation for the West Pier (in the water) is based on the results of the boreholes from the west abutment and east pier.
2. Pile tip elevation after driving is complete.

For steel HP310x132 piles driven through the bouldery material to the estimated tip elevation noted above which derive axial resistance primarily from shaft friction, a factored axial geotechnical resistance at ULS of 1100 kN and an axial geotechnical resistance at SLS of 900 kN may be used for design. In this case, the pile capacity must be verified in the field by the use of the Hiley formula (Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an ultimate capacity of 2750 kN. The following note should be shown on the Contract drawing assuming that a resistance factor of 0.4 (in accordance with MTO Foundations requirements) is applied to the use of the Hiley:

- "Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 2,750 kN per pile but must be driven to between the elevations given in the table above."

For steel HP310x110 piles driven 1 m to the design tip elevation through the bottom of cased, pre-drilled hole and therefore derive axial resistance from end bearing, a factored axial geotechnical resistance at ULS of 1100 kN and an axial geotechnical resistance at SLS of 900 kN may be used for design.

The calculation of pile capacity for the HP310x132 piles was carried out using the Meyerhof (1976) method which takes into account both the end-bearing (i.e., toe) resistance and frictional resistance (i.e., skin friction) along the piles, based on the SPT 'N' values and DCPT values obtained in the boreholes. The resulting calculation indicates approximately the same pile capacity for both pile sections. The pile capacity was then divided by a value of 4 as recommended by Meyerhof (1976) to account for potential errors in obtaining the 'N' values at the anticipated toe elevation of the pile. The calculated pile capacity was also compared to actual pile load test data from other sites, including pull out test data which indicates that about 17% of the capacity is related to end-bearing resistance and the rest to skin friction. The resulting pile resistance values recommended are tempered values between those calculated using 'N' values and those calculated using pile load test data.

For HP310x110 piles advanced by driving 1 m below the base of a pre-drilled hole and within the zone where DCPT values greater than 100 blows per 0.3m of penetration were recorded, the piles essentially derive their resistance from end-bearing. However, due to the uncertainty of the type/quality of the bearing stratum at the tip elevation (i.e., possible cobbles and boulders) the same resistances were also recommended for these piles driven in this manner.

In both cases, the above values assume that the pile is not "hanging-up" on a boulder above the highest acceptable tip elevation presented above. Since it is possible that one or more piles may hang-up on a boulder, the design should be flexible enough to allow for the addition of extra piles within the footing area and for the removal/recovery of piles that are not driven to the required depth. Since integral abutments are not flexible in their pile locations, we recommend that the bridge design not consist of integral abutments.

5.3.3 Resistance to Lateral Loads

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 evaluation of the piles subjected to lateral loads (e.g., ice loads) should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The pile should be modelled as a beam-column supported by springs equivalent to the passive soil reaction distributed along the shaft. The passive resistance developed for lateral deformations typical of bridge foundations is generally much less than the passive pressure associated with a full passive resistance. This full passive resistance is calculated from earth pressure theories assuming unlimited deformation of the soil. The lateral resistance of the pile may be limited by the factored structural flexural resistance of the pile rather than the resistance of the soil.

Therefore, in order to develop the full passive resistance, the pile would have to deflect a 'large' amount. For piles 'fixed' within the pile cap, the magnitude of possible deflection is further reduced and the horizontal geotechnical resistance of the pile is some fraction of the full passive resistance occurring at relatively small horizontal displacements.

It can be assumed based on the shear strength of the soil, that the pile can be considered a laterally supported compression member. The horizontal load capacity of vertical piles may be limited in three different ways:

- The capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
- The bending moments may generate excessive bending stresses in the pile material, resulting in structural failure of the piles; or
- The deflections of the pile heads may be too large to be compatible with the superstructure.

CFEM (1992) gives two methods by which to assess the lateral capacity of a pile. The first is Brom's Method (1964), which examines failure criteria (i.e., ultimate horizontal resistance) for two types of piles – 'short piles' where the lateral capacity of the soil adjacent to the pile is fully mobilized and 'long piles' where the bending resistance of the pile is fully mobilized.

The second method examines the lateral deflections of the pile by using the horizontal subgrade reaction theory where the soil around a pile is modelled using a series of springs. The spring constant is called the coefficient of horizontal subgrade reaction, k_h (kN/m^3). The value of k_h is used as an input parameter into the elastic soil-structure interaction model. For cohesionless soils, such as at the Montreal River Bridge location, the coefficient of horizontal subgrade reaction to a vertical pile can be estimated using the following formula:

$$k_h = \frac{n_h z}{B}$$

where:

k_h = coefficient of horizontal subgrade reaction (MPa/m)
n_h = the constant of horizontal subgrade reaction, as given below (MPa/m)
z = depth (m)
B = pile diameter/width (m)

Based on the above discussion, it is considered that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case. For the proposed piles (HP310x132) driven into the very dense sand and gravel deposit, the horizontal resistance at Ultimate Limit States (ULS) will be controlled by structural limitations such as the yield moment (M_{YIELD}) of the pile (i.e., Brom's 1964 method). At Serviceability Limit States (SLS), the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil.

The following ranges for the value of n_h may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments; and the requirement for lateral support in the case of non-integral abutments and the piers.

Soil Unit	***Elevation***	***n_h (MPa/m)***
Backfill around piles and CSPs (assumed to be compacted granular fill below the groundwater level).	Where applicable	5 to 10
Compact to dense sand to sand and gravel deposits (i.e. N < 50 blows).	Ground surface to: Elev. 276 m (W. Abut) Elev. 280 m (W. Pier) Elev. 284 m (E. Pier) Elev. 281 m (E. Abut)	6 to 10
Very dense sand to sand and gravel deposits (i.e. N > 50 blows).	Below the elevations given above.	10 to 12

A maximum lateral resistance of 130 kN at ULS and 50 kN at SLS is recommended for HP310x132 piles.

The upper zone of soil (down to a depth below the pile cap equal to about 1.5 x B after Brom's 1964, where B = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also 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 horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

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

5.3.4 Frost Protection

The pile caps should be provided with a minimum of 2.2 m of soil cover for frost protection. If the required soil cover cannot be provided, consideration could be given to the use of rigid polystyrene foam insulation below the footings. As a guideline, one inch of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover.

5.4 Caissons

As an alternative to pile foundations, the use of caissons could be considered for support of the bridge abutments and piers at this site. The advantage of using caissons is that a fewer number are generally required compared to piles; however, there will be difficulty advancing large diameter caissons through the sand and gravel containing cobbles and boulders at this site using conventional augering techniques. The following section of the report discusses construction concerns regarding caisson installation through the sand and gravel containing cobbles and boulders at this site, which should be taken into account during the design of the caissons.

5.4.1 Construction Considerations

Due to the presence of cobbles and boulders within the sand and gravel deposit at this site, especially at the west pier and to some extent at the west abutment, augering for caissons using conventional construction techniques may prove problematic in the field (as indicated by the difficulty in advancing the foundation boreholes using conventional drilling techniques). Specialized construction techniques would be required at these locations.

If caissons are constructed using conventional augering techniques, coring, churn drilling and/or down hole hammer techniques will be necessary to advance the casing through any boulders that are encountered. These techniques are costly and time consuming, ultimately requiring additional time and materials. Caissons could be installed using the same dual rotary systems as discussed in Section 5.3.1 but using a larger diameter casing. This method would be much faster but the overall costs would likely be much greater than that of conventional augering (assuming no cobbles and boulders).

Temporary liners would be a requirement at this site since the groundwater levels are high and the soils would be susceptible to caving and sloughing below the groundwater level. Since down-hole inspection of the caisson would not be possible and because of the high groundwater pressures and the potential for basal heave, lower end-bearing capacities would be necessary for design. It may be possible, however, to use an underwater, down hole camera to inspect the base of the caissons, depending on the construction techniques used. Tremie concrete methods would be necessary for placing concrete below the water level as outlined in Special Provision SP902S01.

It should be noted that if a caisson cap is required below the river water level at the east and west piers, cofferdams will be needed to control water in order to build the caisson cap in the dry. A steel sheeted cofferdam will be needed to allow construction of the caisson cap (as discussed in Section 5.9). In this regard, if the caissons extend up to the underside of the bridge deck, the caisson alternative may eliminate the need for a caisson cap in the river and therefore eliminate the requirement for a cofferdam at this location. Additional considerations with respect to ice loading and the like would have to be included in the design for this option.

5.4.2 Axial Geotechnical Resistance

The estimated design base elevations for caissons founded within the very dense sand and gravel deposits can be derived from the table in Section 5.3.2.

The caissons will derive their axial resistance from both shaft friction and end-bearing resistances. The factored axial geotechnical resistance at ULS and axial geotechnical resistance at SLS that may be used for design are given in the table below for the various caisson diameters:

Caisson Diameter (m)	***Axial Resistance (kN)***	
	Very Dense Sand and Gravel	
	ULS	**SLS**
0.9	2,000	1,600
1.5	3,250	2,600

It should be noted that blow up of the base of the caisson could occur during installation and sufficient head should be maintained at all times to balance the hydrostatic water pressures.

5.4.3 Resistance to Lateral Loads

The resistance to lateral loading for the caissons should be in accordance with Section 5.3.3, with the upper limit as determined through the use of the horizontal subgrade reaction formulas. The recommended maximum lateral resistance for the caissons is as follows:

Caisson Diameter (m)	***Factored Lateral Resistance at ULS (kN)***	***Lateral Resistance at SLS (kN)***
0.9	400	150
1.5	650	250

5.4.4 Frost Protection

Caisson caps should be provided with a minimum of 2.2 m of soil cover for frost protection or sufficient insulation as described in Section 5.3.4.

5.5 Retaining Wall

The northeast retaining wall extends eastward from behind the east abutment, on the north side of the proposed bridge for a total length of about 16 m. The height of the wall above the finished grade ranges from about 3 m at the abutment to less than 2 m at the east end of the wall. However, due to the existing sloping ground in this area and the new embankment configuration, the height of the wall above the proposed footing elevations varies from 5.5 m near the abutment to 4.8 m at the east end of the wall.

Based on the results from BH 6 and BH 10 at the west and east ends of the proposed wall, the subsoils consist of sand and gravel containing cobbles and boulders. The measured SPT ‘N’ values within the zone of influence of the retaining wall footing range from 16 blows to 46 blows per 0.3 m of penetration, indicating that the deposit has a compact to dense relative density. The groundwater level in the boreholes (across the site) was encountered below Elevation 299.5 m, approximately coincident with the water level in the Montreal River.

Based on the subsoil conditions at the site, consideration could be given to two different wall types: a conventional concrete retaining wall on shallow spread footings, and a retained soil system (RSS) wall. In this case, since the existing ground is sloping upwards to the east, the footing has to be stepped to accommodate this change in grade and minimal excavation would be required for either wall type. From a foundations perspective, we recommend an RSS type wall be used since this type of wall is more economical to construct than a concrete wall.

Based on the design drawings provided to us by Lea, the proposed bottom of footing ranges from Elevation 305.2 m at the west end of the wall, stepping up to Elevation 306.4 m at the east end of the wall. The top of the wall ranges from Elevation 310.1 m to 311.2 m at the west and east ends of the wall respectively.

5.5.1 Conventional Concrete Gravity Wall

Consideration could be given to constructing a conventional concrete retaining wall supported on shallow spread footings. For preliminary design, a factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 200 kPa (for 25 mm of settlement) may be used for design of footings placed on the undisturbed sand and gravel at the given footing elevations, assuming a footing width of 3.5 m.

A minimum of 2.2 m of soil cover should be provided to the base of the concrete footing to provide adequate protection against frost penetration or sufficient insulation as described in Section 5.3.4.. Given that a minimum of 2.2 m of fill will be present in front of the wall, there will be sufficient passive resistance in front of the wall to resist overturning.

The resistance to lateral forces / sliding resistance between the compacted granular fill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan φ’, between the concrete wall footing and the sand and gravel subgrade materials may be taken as 0.68. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.5.2 Retained Soil System (RSS) Wall

Consideration could be given to the use of a mechanically-reinforced retained soil system (RSS) which consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or geo-grids. A facing material, typically pre-cast concrete panels supported on a strip footing and mechanically fastened to the reinforcing strips or geo-grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material.

Normally, the pre-cast concrete panel facing footing is placed on a granular levelling pad with a minimum thickness of 150 mm. In this case, since the subgrade material consists of sand and gravel, it is recommended that both the facing footing and the reinforced soil mass be constructed directly on the subgrade. The subgrade should be inspected and proof-rolled after the desired foundation elevation is reached. It is noted that the base of RSS walls and their facing footings do not have to be provided with frost cover, therefore, consideration could be given to raising the footing level, although the sloping ground in front of the wall would have to be taken into consideration.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, a factored geotechnical resistances at ULS of 300 kPa and a geotechnical resistance at SLS of 200 kPa may be used for design of the reinforced soil mass founded on the properly prepared subgrade.

There is sufficient soil mass to provide adequate resistance against overturning, and the global stability of the wall has been checked and found to have an adequate factor of safety. The internal stability of the RSS wall should be checked by the designer of the RSS wall.

The resistance to lateral forces / sliding resistance between the new compacted granular fill of the RSS wall and the sand and gravel subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan φ', may be taken as 0.60. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining 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 Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. 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 taper should be in accordance with OPSD 3501.00 and 3504.00.
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3505.000.
- 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. Compaction equipment should be used in accordance with OPSS 501.06. 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 2.2 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 1.5 horizontal to 1 vertical (1.5H: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).*
- For Case I, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill or rock fill:

	Earth Fill	Rock Fill
Soil unit weight:	21 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.35

- For Case II, the pressures are based on the rock fill as above or 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. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

- rotation of approximately 0.002 about the base of a vertical wall;
- horizontal translation of 0.001 times the height of the wall; or
- a combination of both.

A restrained structure is typically culverts or rigid frame bridge where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHDBC. In this regard, the following should be taken into account in the lateral earth pressures.

- 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 1. The site-specific zonal acceleration ratio for Matachewan is 0.05. Based on experience, for the subsurface conditions at this site, a 30 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.05g to 0.065g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.065.
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.10$). 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 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**	
	Earth Fill	**Rock Fill**	**Granular A**	**Granular B Type II**
Yielding wall	0.30	0.22	0.26	0.26
Non-yielding wall	0.34	0.25	0.30	0.30

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 for 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 0.065. This corresponds to displacements of up to 16 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:

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

Where K = either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} = the seismic active earth pressure coefficient;
γ' = the effective unit weight of the soil (kN/m^3)
- taken as soil unit weights given above for fill materials;
- taken as 21 kN/m^3 above Elev. 299 m for the native materials and 11 kN/m^3 below Elev. 299 m

d = the depth below the top of the wall (m); and
H = the height of the wall above the toe (m).

5.7 Liquefaction Potential and Seismic Analysis

5.7.1 Analysis Methods

The liquefaction potential of the granular soils below the immediate approach embankments and under seismic loading has been considered using the empirical method outlined in Section C.4.6.2

of the *CHBDC Commentary*, which correlates the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR) of the soils with their normalized penetration resistance and fines content for granular soils. The cyclic resistance ratio (CRR) has been determined using the empirical method suggested by the CHBDC based on papers by Seed et al (1984) using SPT 'N' values and accounting for fines content. The method used to determine the CSR will be the simplified procedure suggested by Seed and Idriss (1971) relating to the peak ground acceleration and effective overburden stress.

In general, geologically young, loose deposits of sand and non-plastic silty sands with low fines content (less than 5 percent passing No. 200 sieve) which are below the water table are potentially susceptible to liquefaction.

5.7.1.1 Liquefaction Induced Settlements

Where liquefaction is identified to be a problem either in clayey soils or in granular soils using the methods described above, vertical deformation of the soil under the earthquake loading may occur due to the contraction of the sand deposit using a relationship developed by Tokimatsu and Seed (1987). This deformation can be estimated using relationships proposed by Makdisi and Seed (1978). If deformation is anticipated, soil improvement methods should be considered and could include densification, removal and re-compaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled.

5.7.1.2 Stability Under Seismic Conditions

The susceptibility of granular deposits underlying the proposed roadway embankments and the consequent stability of the embankment under seismic loading conditions for this site has been assessed. The peak zonal acceleration for this site (Matachewan) is 0.065g, which is based on a zonal acceleration of 0.05g multiplied by an amplification factor of 30 percent for the types of soils found in this area. Typically, the seismic loading will be applied to the long-term (drained) conditions.

If liquefaction of the subsoils under the embankment loading is not anticipated, a factor of safety of 1.0 is typically used to assess the stability under magnitude 7.0 earthquake events.

Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual strength parameters in the liquefied layers using the correlation proposed by Seed and Harder (1990) which is correlated to SPT 'N' values. If under these conditions, the embankment is estimated to have a factor of safety less than 1.0 under static conditions, the embankment is considered to be susceptible to a flow slide. Flow slides are characterized by very large lateral and vertical displacements of the embankment. If under residual strength conditions, the static factor of safety is greater than 1.0, lateral displacements may still occur, and are estimated using the Newmark method, which compares the design ground acceleration to that necessary to induce a factor of safety equal to 1.0 in the embankment (i.e. yield acceleration). If the yield acceleration is greater than the maximum acceleration for this site, then no remedial measures are required. If the yield acceleration is less than the maximum acceleration, soil improvement methods may be necessary to improve soil conditions.

5.7.2 Results of Analysis

Using the methods outlined in Section 5.6.1, the soils at this site are not considered to be liquefieable. A factor of safety of greater than 1.0 is obtained for magnitude 7.0 earthquake events.

5.8 Approach Embankment Design and Construction

The ground surface at the west bridge approach is at about Elevation 302.5 m and ground surface at the east bridge approach is about Elevation 311 m. While the ground surface at the west approach is generally fairly level, the ground surface at the east approach generally slopes downwards towards the river and is at about Elevation 305 m at the location of the east abutment. The proposed grade of the new bridge will be between Elevation 305.5 m and 310.5 m at the west and east abutments, respectively, resulting in overall embankment heights of about 3 m at the west approach and between 0 and 6 m at the east approach.

The subsoils at the site consist of loose to very dense sand and gravel containing cobbles and boulders. Surficial loose to dense granular fill and/or sand was also encountered in the boreholes at the approaches. The groundwater level is generally at the same elevation as the river water level and for analysis, both the high water level at Elevation 301.7 m and the lowest water level of Elevation 298.9 m (August 2005) were used. The results of stability and settlement analysis for the new approach embankments are presented in the following sections.

5.8.1 Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.18), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at these sites considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the various embankment heights, excavation depths and geometries.

It is understood that consideration is being given to the use of earth fill or rock fill for construction of the approach embankments. Rock fill is assumed to have side slopes at 1.25 horizontal to 1 vertical (1.25H:1V) and the earth fill is assumed to have side slopes at 2H:1V. For the soils at this site, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of in situ SPT and are given in the table below:

Soil Type	***Unit Weight (kN/m^3)***	***Angle of Internal Friction***
Rock Fill	19	40^o
Earth Fill (Granular Material)	21	35^o
Loose to very dense sand and gravel	21 above water level 11 below water level	28^o

The approach embankments (including the slopes in front of the abutments) at this site constructed out of earth fill material with 2H:1V side slopes or rock fill side slopes with 1.25H:1V side slopes are anticipated to be stable up to 6 m in height. It should be noted that where the existing native slopes are used as part of the overall embankments (such as at the east approach), these sides slopes should be stable at 2H:1V.

5.8.2 Settlement

The immediate compression of the existing fill and native sand to sand and gravel deposits under the proposed embankment loading were modelled by estimating an elastic modulus of deformation based on the SPT 'N' values. The following correlation proposed by Bowles (1984) was employed, and the results tempered by engineering judgement considering experience in similar soil conditions:

$$Es = 18{,}000 + 750N \text{ (Bowles, 1984) (kPa)}$$

where:

N = average SPT 'N' value in the sand and gravel deposit

The estimated settlement, analyzed using the method outlined in the Canadian Foundation Engineering Manual, 3rd Edition (CFEM), are 20 mm and 45 mm at the west and east approaches, respectively.

In addition to settlement of the subsoils, settlement of the embankment fill will also occur. The settlement of rock fill is governed by the data contained in the document entitled “Rockfill in the Foundation Design of Highway Structures” by the Ministry of Transportation and Communications, Research and Development Branch, dated 1982, which was used to establish the relative percentages for varying of rock fill embankment heights. For rock fill compacted in 1.5 m lifts in accordance with SP206S03, the rock fill settlement is expected to be 10 mm and 25 mm for 3 m and 6 m high embankments, respectively. About 60% of the rock fill settlement is expected to occur during the first year.

If earth fill is used for embankment construction, the magnitude and rate of settlement will be governed by the quantity of fines. For properly placed and compacted granular fill, the settlement is expected to be less than 25 mm and will occur during construction. If earth fill containing a large amount of fines is used in the construction of the embankments, the settlement would also be up to about 25 mm but this settlement will occur over the long-term. In this case, it would be recommended that paving be delayed for at least 6 months.

The following table summarizes the estimated settlement at this site, assuming that if earth fill is used, it is comprised of granular material:

Soil Type	***Estimated Settlement***			
	West Approach (H = 3 m)		**East Approach (H = 6 m)**	
	Rock Fill	Earth (Granular) Fill	Rock Fill	Earth (Granular) Fill
Embankment Fill	10 mm	25 mm	25 mm	25 mm
Native Soils (and existing fill)	20 mm	20 mm	40 mm	45 mm
Total	**30 mm**	**45 mm**	**65 mm**	**70 mm**

Because these settlements are expected to occur in the short term, essentially during construction, and because the soils are cohesionless, downdrag on the deep foundation elements are not considered to be a concern. As a precaution, embankment construction should be carried out in advance of pile driving. If this is the case, the size of rock fill used in the abutment area should be limited to 75 mm maximum particle size.

5.9 Subgrade Preparation and Embankment Construction

The existing subsoils are considered to be appropriate subgrade for the proposed approach embankments; however, all softened/ loosened soils should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement. Topsoil was noted in the boreholes advanced on the west side of the river and it should be noted that vegetation exists on both sides of the river. This indicates that organic materials may be encountered elsewhere during subgrade preparation. Provided that the organic materials are not extensive, sub-excavation of the topsoil/vegetation would not be required as part of subgrade preparation unless the soils containing organics are softened/loosened.

The embankment heights at this site are less than 6 m and therefore do not require a mid-height berm (in accordance with Northern Region Directives).

If rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision SP206S03 (dated January 2004). The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with SP206S03. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

In order to minimize differential settlement between the existing east slope and the newly placed embankment fill, the new fill should be keyed into the existing slope as per OPSD 208.01.

The front slope of the west abutment will be constructed with its toe immediately adjacent to the river and erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of rip-rap (300 mm diameter), rock protection or concrete slope paving. The toe of the east abutment front slope will be located above the high water level and erosion protection is therefore not required. The potential for scour below the footings and pile/caisson caps must be taken into account in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be carried out as soon as possible. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil and seeding. The requirement to vegetate the embankment side slopes does not apply to rock fill slopes.

5.10 Design and Construction Considerations

5.10.1 Excavations and Groundwater Control

It is anticipated that excavations for the pile/caisson caps will extend through the sand to sand and gravel fill and native sand and sand and gravel deposits. Excavations through these deposits will encounter cobbles and boulders of various types and sizes. For those excavations made in open cut, temporary excavation side slopes above the groundwater level (Elevation 299 m) should be made at no steeper than 1.5H:1V. Excavations in open cut below the groundwater level should be made at no steeper than 3H:1V. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The loose to dense sand to sand and gravel fill and the native loose sand and compact to dense sand and gravel deposits at the site are classified as Type 3 soils, according to the OHSA.

For excavations made for the pile/caisson caps distant from the river (i.e. more than 20 m away), it is expected that the groundwater may generally be controlled by pumping from well-filtered sumps at the base of the excavation, provided that the elevation of the base of the excavation is above the groundwater level (Elevation 299 m). For excavations in the river or in close proximity to the river (i.e. both piers) and for those made below the groundwater level, additional groundwater control measures, such as a cofferdam, may be required for excavations and these are discussed below. Surface water should be directed away from the excavations at all times.

At the piers and the west abutment, it will be necessary to excavate below the river water level and therefore, cofferdams will be required at these locations. It is understood that there is a special provision for cofferdams that is typically used in MTO Contracts for this purpose. The design is the responsibility of the contractor. Since the soil deposits consist of sand and gravel to depths of over 30 m, it may not be possible to achieve a cut-off to the groundwater in these excavations. In addition, due to the presence of cobbles and boulders in the soil, installing standard sheet-piling to a sufficient depth to achieve water cut-off will not be possible unless specialized construction techniques are employed. Such techniques could involve line drilling ahead of and to facilitate driving of steel sheeting or installing the sheet piles in alternating pre-drilled cased holes. Alternatively, a pre-made cofferdam could be constructed and then pushed into place as excavation within the cofferdam advances. There will likely be some sloughing of the sand and gravel into the excavation during installation of the cofferdam.

The requirements of SP902S01 clause 902.07.03 indicate that concrete for footings shall be placed in the dry, unless otherwise specified in the contract. However, a cofferdam of steel sheeting in this material will likely not be able to be installed to sufficient depth to provide full water cut-off. Concrete should therefore be placed by tremie methods to balance the hydrostatic head within the sheeted enclosure, coupled with pumping from sumps located in the corners of the enclosure. Assuming a 1 in 10 year flood Elevation of 301.3 m (as given to us by Lea), and given the footing thicknesses shown on the design drawings, the tremie plug thicknesses are given in the table below, and utilize a factor of safety against uplift of 1.2.

Location	***B/F Elevation (m)***	***Height of Water above B/F (m)***	***Thickness of Tremie Plug (m)***
West Abutment	299.1	2.2	1.0
West Abutment	296.0	5.3	2.0
West Pier	298.5	2.8	1.2

where B/F indicates bottom of footing.

The thickness of the tremie plugs at the above noted locations is based on the following assumptions:

- The sheet piles/cofferdam walls do not extend beyond the base of the tremie plug; the Contractor will be responsible for determining the actual length of the sheet piles/walls. It should be noted that it will likely be difficult to install sheet piles in the compact to dense sand and gravel containing cobbles and boulders encountered at this site.

- The H-piles will act as anchors to resist uplift of the tremie plug, assuming they are driven prior to placement of the tremie plug.

The Contractor's designer should check these thicknesses against his design. A note should be included on the contract drawing to this effect. In addition, a NSSP should be included in the contract to address the issue of cobbles and boulders and is included in Appendix A for reference.

Excavation support for protection of the existing roadway may be required at this site at the east abutment. Where required, the temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

5.10.2 Obstructions

Cobbles and boulders will present significant challenges to construction at this site. Difficulties with augering, piling, excavation and dewatering will be encountered and a NSSP should be included in the contract document to alert the contractor to such potential construction difficulties. An example NSSP is included in Appendix A for reference.

5.11 Closure

This report was prepared by Miss Sarah Poot, P.Eng., a senior geotechnical engineer and the technical aspects were reviewed by Mr. Murty Devata, P. Eng., a Consultant to Golder Associates Ltd. Mr. Jorge Costa, P. Eng., Principal with Golder and the Designated MTO Contact, conducted a quality control review of the report.

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REFERENCES

Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, 2nd Edition. McGraw-Hill Book Company, New York.

Canadian Foundation Engineering Manual. 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations, 512p.

Commentary on CAN/CSA-S6-00, Canadian Highway Bridge Design Code 2001. CSA, Toronto.

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Makdisis, F.I., and Seed, H.B. 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. ASCE Journal of the Geotechnical Engineering Division, V. 104, GT7, pp 849-867.

Meyerhof, G. G. (1976). "Bearing Capacity and Settlement of Pile Foundations"; The Eleventh Terzaghi Lecture, Nov. 6. 1975. J. of Geotech. Engrg. Div., ASCE, Vol. 102, GT3, pp. 195-228. Discussions in Vol. 103, GT3 and GT4, Closure in Vol. 103, GT9.(presented in CFEM 3rd Edition 1992).

Ontario Geological Survey, Northern Ontario Engineering Geology Terrain Study 83, Elk Lake Area, 1979

Tokimatsu, K., and Seed, H. 1987. Evaluation of Settlements in Sands Due to Earthquake Shaking. ASCE Journal of Geotechnical Engineering, V.113, N.8.

TABLE 1
DRILLING SUMMARY
MONTREAL RIVER BRIDGE REPLACEMENT AT MATACHEWAN
HIGHWAYS 66 & 566, G.W.P 83-86-00, SITE NO. 47-017

Borehole/ Location	*Ground Surface Elevation (m)*	*Depth (Elevation) of Borehole (m)*	*Drill Method*	*Observations*
3 West Approach	302.5	5.2 (297.3)	Hollow Stem Auger	• SPT sampling spoon bouncing at 1.0 m depth (inferred cobble/boulder).
1 West Abutment	302.7	31.4 (271.3)	Hollow Stem Auger, NW Wash Boring, NQ Core Barrel	• Sand heaving inside augers at about 9 m depth. • Several instances of grinding during auger advance (inferred cobbles/boulders). • Difficult augering due to high water pressure below 15.2 m depth, switch to NW casing. • Switch to NQ core barrel below 26.5 m depth to advance through boulders.
2 West Abutment	302.5	11.3 (291.2)	Hollow Stem Auger, NW Wash Boring, DCPT	• Auger refusal at 2.7 m depth, switch to NW casing. • Casing advance terminated at 11.3 m depth due to worn out casing shoe, DCPT attempted at this depth but no penetration possible (inferred boulder). • DCPT driven from ground surface in adjacent hole; hammer bouncing at 1.8 m depth (inferred boulder).
9 West Pier	298.9 (River Water Level)	6.4 (292.5)	BW Wash Boring, BQ Core Barrel, (Portable)	• Gravel, cobbles continuously becoming stuck in BQ core barrel. • Difficulty advancing boreholes, drilling very slow, grinding throughout – refusal to casing/coring at 6.4 m depth; depth of water = 0.6 m on August 23, 2005.
4 East Pier	302.0	26.8 (275.2)	Hollow Stem Auger, Diamond Rotary, DCPT	• Difficult auger advance below 4.6 m depth, switch to NW casing. • Boulder encountered between 5.3 and 5.9 m depth • Difficult casing advance below 11.3 m due to heaving sand and cobbles and boulders, drive DCPT below 16.8 m depth.
7 East Pier	302.0	25.9 (276.1)	Hollow Stem Auger, DCPT	• Auger refusal at 3.7 m depth, start of DCPT.

TABLE 1
DRILLING SUMMARY
MONTREAL RIVER BRIDGE REPLACEMENT AT MATACHEWAN
HIGHWAYS 66 & 566, G.W.P 83-86-00, SITE NO. 47-017

Borehole/ Location	***Ground Surface Elevation (m)***	***Depth (Elevation) of Borehole (m)***	***Drill Method***	***Observations***
5 East Abutment	305.8	28.7 (277.2)	Hollow Stem Auger, Diamond Rotary, DCPT	• Sand heaving inside augers at 10.5 m depth, switch to NW casing. • Casing advance stopped at 22.0 m depth due to worn out casing shoe, start of DCPT.
6 East Abutment	304.6	24.1 (280.5)	Hollow Stem Auger, DCPT	• Auger refusal at 4.4 m depth, start of DCPT.
8 East Approach	310.5	1.2 (309.3)	Hollow Stem Auger	• Auger refusal at 1.2 m depth.
10 Northeast Retaining Wall	310.6	10.7 (299.9)	Hollow Stem Auger	• Spoon bouncing on cobble at 2.4 m depth. • Very hard augering below 2.4 m depth due to cobbles and boulders.

Note: Wash boring methods (casing) in the deposits were very slow, resulting in numerous delays and worn casing bits.

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES
MONTREAL RIVER BRIDGE REPLACEMENT AT MATACHEWAN
HIGHWAYS 66 & 566
G.W.P 83-86-00, SITE NO. 47-017

Options	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Steel H-piles terminated in sand and gravel overburden	• Can found piles below the scour elevation. • If special techniques are used for pile installation, rate of pile installation would be increased over caissons.	• Advancing driven piles through bouldery overburden will be difficult; specialized equipment may be required to pre-auger a pilot hole for the driven pile. • Additional piles may be required if piles “hang-up” on boulders. • Additional cost for dewatering pile caps in the river using specialized construction techniques to remove/penetrate cobbles and boulders.	• Typical pile cost = \$200/m • Additional cost using specialized equipment = \$200/m	• Damage to piles. • Not getting required resistance; changes to design during construction phase (adding and/or moving piles).
Caissons terminated in sand and gravel overburden	• Can found caissons below the scour elevation. • Reduced number of deep elements compared to piles. • Specialized equipment would be necessary to advance caissons through boulders; cofferdam could be eliminated at west pier if caisson cap not required.	• Advancing augered caissons through bouldery overburden will be difficult; coring, churn drilling and/or down-hole hammering will be required. • May take several days to complete one caisson. • Temporary liners would be required for groundwater control; high water pressures may increase difficulties during augering. • Concrete for caissons would have to be placed by tremie methods below the water level. • Additional cost for dewatering caisson caps in the river using specialized construction techniques to remove/penetrate cobbles and boulders.	• Typical caisson cost = \$4,900/m (plus \$95,000 mobilization) • Typical caisson cost considering additional time to deal with boulders and/or special equipment = greater than \$4,900/m plus mobilization	• Not getting to the required depth due to poor advance rates.
Spread Footings on sand and gravel subgrade (Abutments only)	• Difficulties of advancing deep foundations through bouldery deposits is avoided.	• May not be able to extend footings to below scour elevation. • Dewatering required; require specialized techniques to drive closed sheet-piling for cofferdam to construct footings in the dry.	• Difficulties with installing cofferdam may increase costs substantially.	• Design may not be feasible at piers unless scour requirements are met.

NOTES:

1. This table should be read in conjunction with Section 5.0 of the Foundation Investigation and Design Report.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

π	3.1416
in 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 stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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 (over-consolidated 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	over-consolidation 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 δ
c'	effective cohesion
c_u, S_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

Golder Associates

RECORD OF BOREHOLE No 1

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00
LOCATION N 5311276.5 ;E 330881.2
ORIGINATED BY SB
DIST HWY 66/566
BOREHOLE TYPE 108 mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE July 20, 2005
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.7	GROUND SURFACE					
0.0	TOPSOIL					
0.2	Fine to medium SAND, trace gravel, trace rootlets Loose Brown Moist	1	SS	8		
301.2 1.5	SAND and GRAVEL, containing cobbles and boulders Loose to dense Brown Moist	2	SS	34	~9	
		3	SS	30/0.1		
	Augers grinding at 2.1 m and 4.5 m depths	4	SS	38	~8	
	Wet at 3.8 m depth	5	SS	36		
		6	SS	9	~17	40 58 (2)
		7	SS	12		
		8	SS	21	~19	
	Sand heaving inside augers between 9.1 m and 11.3 m depths.	9	SS	26		
		10	SS	13	~21	
		11	SS	18		
		12	SS	28		

ELEVATION SCALE: 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290, 289, 288

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m^3

Groundwater level (▽) at approx. elevation 298.5.

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 1

 METRIC

PROJECT 04-1111-056			
W.P. 83-86-00	LOCATION N 5311276.5 ;E 330881.2		ORIGINATED BY SB
DIST	HWY 66/566	BOREHOLE TYPE 108 mm ID Hollow Stem Augers	COMPILED BY AB
DATUM Geodetic	DATE July 20, 2005		CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, containing cobbles and boulders Loose to dense Brown Moist	13	SS	16	287	o	25 75 (1)
	Difficulty advancing augers, switch to NW casing at 15.2 m depth				286		
		14	SS	13			
	Becoming grey below 17.8 m depth				285		
		15	SS	15	284	o	
					283		
		16	SS	22			
					282		
		17	SS	24	281	o	
					280		
		18	SS	44			
					279		
	Sand seam between 23.9 m and 25.4 m depth						
		19	SS	18	278	o	4 93 (3)
					277		
		20	SS	49			
	Casing advance stopped at 26.5 m depth, switch to NQ casing to continue advance of borehole	21	NQRC		276		
					275		
		22	NQRC				
					274		
	Large boulders below 29.7 m depth				273		

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; STRAT PLOT; GROUND WATER CONDITIONS.

Continued Next Page

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

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RECORD OF BOREHOLE No 1

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00
LOCATION N 5311276.5 ;E 330881.2
ORIGINATED BY SB
DIST
HWY 66/566
BOREHOLE TYPE 108 mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE July 20, 2005
CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
271.3	SAND and GRAVEL, containing cobbles and boulders Loose to dense Brown Moist						272				
31.4	END OF BOREHOLE Notes: 1. Water level in piezometer after installation at 4.3 m below ground surface (Elev. 298.4 m). 2. Water level in piezometer at 4.4 m depth (Elev 298.3 m) on July 27, 2005. 3. Water level in piezometer at 4.3 m depth (Elev 298.4 m) on August 23, 2005.										

MISS_MTO 04111056.GPJ ON_MOT.GDT 30/12/05

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 2

 METRIC

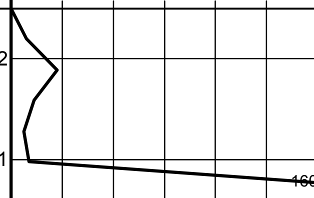
PROJECT 04-1111-056

W.P. 83-86-00 LOCATION N 5311267.1 ;E 330881.8 ORIGINATED BY SB

DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB

DATUM Geodetic DATE July 22, 2005 CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
302.5	GROUND SURFACE												
0.0	TOPSOIL												
0.1	Sand, trace to some gravel, rootlets, occasional cobbles (FILL) Compact to dense Brown Moist					302							
		1	SS	10				o					
						301	160						
		2	SS	21			Hammer bouncing at 1.8 m depth						
		3	SS	24		300		o		40	48	9	3
	Auger refusal at 2.7 m depth, switch to NW casing				∇								
		4	SS	40		299							
298.2 / 4.3	SAND and GRAVEL, containing cobbles Very dense to compact Brown to grey Wet					298							
		5	SS	78				o					
						297							
		6	SS	32		296							
						295							
	Becoming grey at 7.6 m depth	7	SS	28				o		33	63	(4)	
						294							
		8	SS	23		293							
						292							
	Casing advance stopped at 11.3 m worn out shoe, unable to advance casing	9	SS	16				o					
291.2 / 11.3	END OF BOREHOLE												



Notes:

1. Borehole caved at 3.2 m depth.

2. Unable to advance dynamic cone test below 11.3 m depth.

2. Dynamic cone test pushed from ground surface in adjacent borehole, 0.6 m North.

Chart scales: Dynamic cone 20 40 60 80 100; Shear strength 20 40 60 80 100 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED); Water content 10 20 30 (PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L). REMARKS & GRAIN SIZE DISTRIBUTION (%).

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

RECORD OF BOREHOLE No 3

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00
LOCATION N 5311270.9 ;E 330857.2
ORIGINATED BY SB
DIST
HWY 66/566
BOREHOLE TYPE 108 mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE July 23, 2005
CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.5	GROUND SURFACE						
0.0	TOPSOIL						
0.1	Fine to medium sand, trace to some gravel (FILL) Compact Brown Spoon bouncing at 1.0 m depth	1	SS	22/.15			
301.0 1.5	SAND and GRAVEL, containing cobbles Very dense to compact Brown to grey Moist	2	SS	92		~3	
		3	SS	33			
		4	SS	20		~6	22 74 (4)
		5	SS	18			
		6	SS	14		~11	
297.3 5.2	END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L
UNIT WEIGHT γ kN/m³
ELEVATION SCALE: 302, 301, 300, 299, 298

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

RECORD OF BOREHOLE No 4

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00
LOCATION N 5311274.8 ;E 330977.7
ORIGINATED BY SB
DIST
HWY 66/566
BOREHOLE TYPE 108 mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE July 23, 2005
CHECKED BY SEP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.0	GROUND SURFACE						
0.0	Sand and gravel, trace rootlets, occasional cobbles (FILL) Loose to compact Moist Brown	1	SS	7			
		2	SS	20			
299.7 / 2.3	SAND and GRAVEL, containing cobbles and boulders Very dense to compact Moist	3	SS	31		~24	25 42 23 10
	Sand seam between 2.3 m and 2.9 m depth	4	SS	119	▽		
	Becoming wet, grey at 3.1 m depth						
	Grey silty fine sand seam encountered between 3.1 m and 4.3 m depth	5	SS	74		~16	39 56 (5)
	Difficulty advancing augers, switch to NW Casing at 4.6 m depth	6	SS	51			
	Boulders encountered between 5.3 m and 5.9 m depth						
		7	SS	21		~19	20 76 (4)
		8	SS	28			
		9	SS	17		~0	
		10	SS	26			
	Difficult casing advance below 11.3 m due to heaving sand and presence of cobbles and boulders						

Elevation scale: 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290, 289, 288

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 4

 METRIC

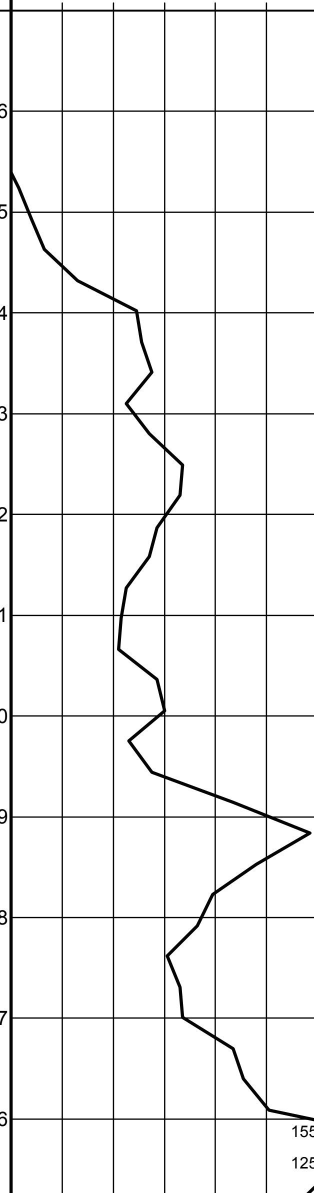
PROJECT 04-1111-056

W.P. 83-86-00 LOCATION N 5311274.8 ;E 330977.7 ORIGINATED BY SB

DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB

DATUM Geodetic DATE July 23, 2005 CHECKED BY SEP

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and GRAVEL, containing cobbles and boulders Very dense to compact Moist						286				
285.2 16.8	START OF DYNAMIC CONE PENETRATION TEST						285 284 283 282 281 280 279 278 277 276	155 125			
275.2 26.8	END OF BOREHOLE Note: 1. Water level in open borehole at 3.4 m depth (Elev. 298.6 m)										



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

RECORD OF BOREHOLE No 5

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00 LOCATION N 5311275.9 ;E 331012.0 ORIGINATED BY SB
DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB
DATUM Geodetic DATE July 26, 2005 CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
305.8	GROUND SURFACE						
0.0	SAND and GRAVEL, containing cobbles Compact to very dense Brown Moist						
					305		
		1	SS	20			
		2	SS	27	304		
		3	SS	40	303	o	55 40 (5)
		4	SS	45		o	
		5	SS	38	302		
	Sand seam encountered between 4.6 m to 5.5 m depth	6	SS	27	301	o	
					300		
		7	SS	57	299		
	Becoming wet below 7.6 m depth	8	SS	23	298	o	
					297		
		9	SS	26	296		
	Sand heaving inside augers at 10.5 m depth, switch to NW casing	10	SS	44	295	o	32 63 (5)
					294		
		11	SS	10	293		
		12	SS	19	292	o	
					291		

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

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RECORD OF BOREHOLE No 5

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00 LOCATION N 5311275.9 ;E 331012.0 ORIGINATED BY SB
DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB
DATUM Geodetic DATE July 26, 2005 CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, containing cobbles Compact to very dense Brown Moist Unable to drive spoon at 15.2 m depth, spoon bouncing Becoming grey at 15.8 m depth				290		
		13	SS	39	289	~18	45 55 (0)
					288		
		14	SS	19	287		
					286		
		15	SS	46	285		
283.9 22.0		16	SS	31	284	~6	
	START OF DYNAMIC CONE PENETRATION TEST				283–278		
277.2 28.7							

Continued Next Page

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

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RECORD OF BOREHOLE No 5

 METRIC

PROJECT	04-1111-056				
W.P.	83-86-00	LOCATION	N 5311275.9 ;E 331012.0	ORIGINATED BY	SB
DIST		HWY	66/566	BOREHOLE TYPE	108 mm ID Hollow Stem Augers
				COMPILED BY	AB
DATUM	Geodetic	DATE	July 26, 2005	CHECKED BY	SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Notes: 1. Water level in piezometer after installation at 7.7 m below ground surface (Elev. 298.1 m). 1. Water level in piezometer at 7.5 m (Elev. 298.3 m) on August 23, 2005.										

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

MISS_MTO 04111056.GPJ ON_MOT.GDT 30/12/05

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RECORD OF BOREHOLE No 6

 METRIC

PROJECT 04-1111-056

W.P. 83-86-00 LOCATION N 5311286.8 ;E 331011.0 ORIGINATED BY SB

DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB

DATUM Geodetic DATE July 27, 2005 CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
304.6	GROUND SURFACE							
0.0	SAND and GRAVEL, occasional cobbles Compact to dense Brown Moist						304	
			1	SS	46			
	Sand seam between 1.5 m and 2.0 m depth		2	SS	16		303	7 80 11 2
			3	SS	40		302	
			4	SS	35		301	
			5	SS	39			
300.2	Auger refusal at 4.4 m depth							
4.4	START OF DYNAMIC CONE PENETRATION TEST						300	
							299	
							298	
							297	
							296	
							295	
							294	
							293	
							292	
							291	
							290	

Continued Next Page

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

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RECORD OF BOREHOLE No 6

 METRIC

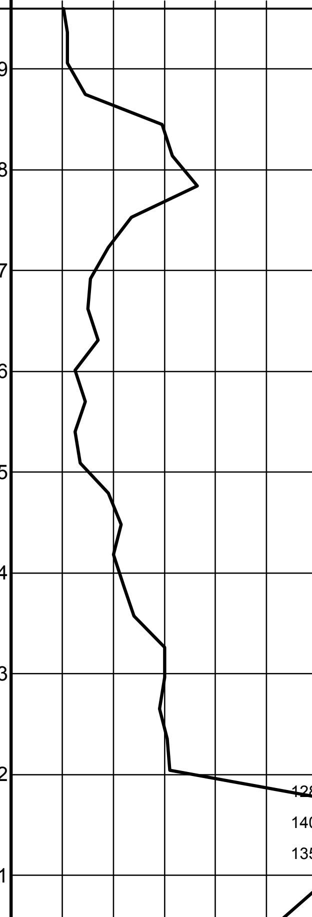
PROJECT 04-1111-056

W.P. 83-86-00 LOCATION N 5311286.8 ;E 331011.0 ORIGINATED BY SB

DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB

DATUM Geodetic DATE July 27, 2005 CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
							289 288 287 286 285 284 283 282 281	128 140 135			
280.5 / 24.1	END OF BOREHOLE										



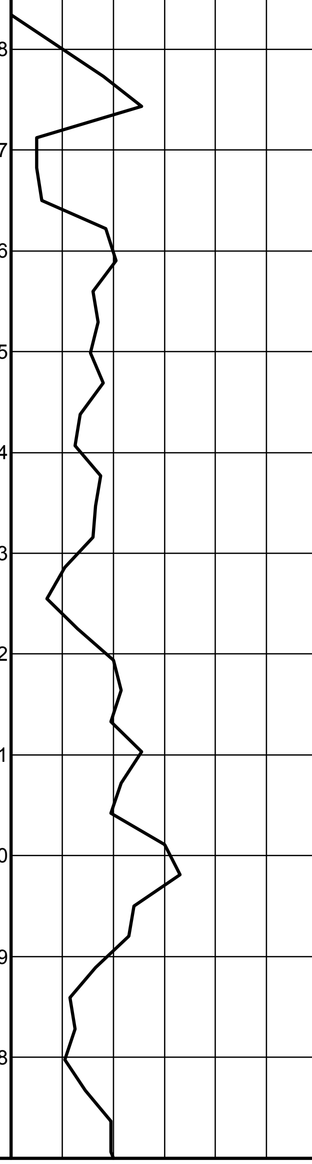
+3, ×3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00
LOCATION N 5311282.8 ;E 330983.8
ORIGINATED BY SB
DIST
HWY 66/566
BOREHOLE TYPE 108 mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE July 27, 2005
CHECKED BY SEP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.0	GROUND SURFACE						
0.0	SAND, some gravel, trace rootlets, occasional cobbles Loose Brown Moist	1	SS	6	301		
300.6 1.4	SAND and GRAVEL, containing cobbles Compact to very dense Brown Wet Becoming grey at 2.3 m	2	SS	18	300	o	43 50 (7)
		3	SS	19	299		
298.3	Auger refusal at 3.7 m depth	4	SS	92		o	54 36 (10)
3.7	START OF DYNAMIC CONE PENETRATION TEST				298 – 288		



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

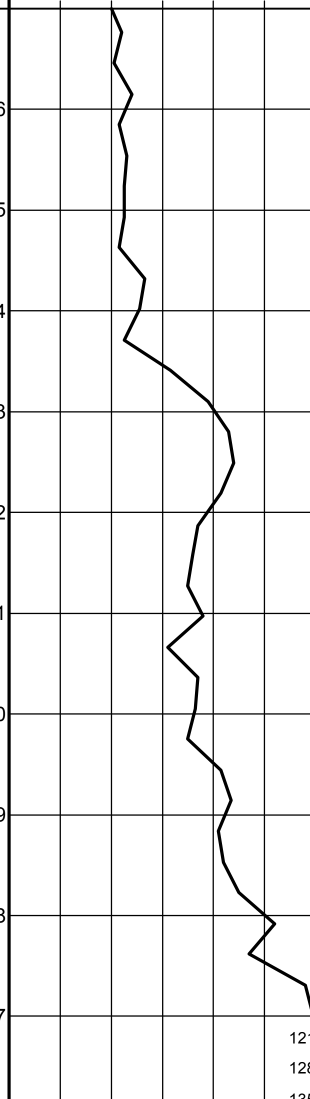
MISS_MTO 041111056.GPJ ON_MOT.GDT 30/12/05

RECORD OF BOREHOLE No 7

 METRIC

PROJECT 04-1111-056			
W.P. 83-86-00	LOCATION N 5311282.8 ;E 330983.8		ORIGINATED BY SB
DIST	HWY 66/566	BOREHOLE TYPE 108 mm ID Hollow Stem Augers	COMPILED BY AB
DATUM Geodetic	DATE July 27, 2005		CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
							286				
							285				
							284				
							283				
							282				
							281				
							280				
							279				
							278				
							277	121, 128, 135			
276.1 / 25.9	END OF BOREHOLE										



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

RECORD OF BOREHOLE No 8

 METRIC

PROJECT 04-1111-056

W.P. 83-86-00 LOCATION N 5311281.3 ;E 331031.9 ORIGINATED BY SB

DIST HWY 66/566 BOREHOLE TYPE 108 mm ID Hollow Stem Augers COMPILED BY AB

DATUM Geodetic DATE July 27, 2005 CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.5	GROUND SURFACE										
0.0	SAND and GRAVEL, occasional cobbles Very dense Brown Moist						310				
			1	SS	50/.25				○		
309.3	Auger refusal at 1.2 m depth										
1.2	END OF BOREHOLE										

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

Golder Associates

RECORD OF BOREHOLE No 9

 METRIC

PROJECT 04-1111-056
W.P. 83-86-00
LOCATION N 5311265.2 ;E 330918.0
ORIGINATED BY AB
DIST HWY 66/566
BOREHOLE TYPE 73 mm OD BW Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE August 23-25, 2005
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
298.9	WATER SURFACE										
0.0	Water										
298.3 / 0.6	River Bed COBBLES and BOULDERS, some sand and gravel matrix						298				
			1	BQRC							
	NW casing advanced to 1.5 m depth; switch to BW casing						297				
	Becoming GRAVEL and COBBLES, some sand below 2.4 m depth		2	BQRC			296				
295.7 / 3.2	SAND and GRAVEL, containing cobbles and boulders Dense		3	SS	9						
			4	WS			295				
			5	BQRC			294				
			6	BQRC							
							293				
292.5	Casing and coring refusal.		7	WS							
6.4	END OF BOREHOLE										

Notes:

1. Gravel, cobbles continuously getting stuck in BQ core barrel; slow drilling progress from river bed surface to bottom of borehole.

2. Alternate BW casing and BQ coring to advance borehole; grinding throughout.

3. Borehole caved to river bed level.

4. Standard Penetration Testing with a 70 lb. hammer. 'N' values above have been converted to a full hammer weight (140 lb.).

MISS_MTO 04111056.GPJ ON_MOT.GDT 30/12/05

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

Golder Associates

RECORD OF BOREHOLE No 10

METRIC

PROJECT	04-1111-056		
W.P.	83-86-00	LOCATION	N 5311287.6 ;E 331034.4
ORIGINATED BY	ID		
DIST		HWY	66/566
BOREHOLE TYPE	108 mm ID Hollow Stem Augers	COMPILED BY	SEP
DATUM	Geodetic	DATE	November 30, 2005
CHECKED BY	SEP		

Column headings: SOIL PROFILE (ELEV / DEPTH; DESCRIPTION; STRAT PLOT) | SAMPLES (NUMBER; TYPE; "N" VALUES) | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION
310.6	GROUND SURFACE
0.0	SAND and GRAVEL, occasional cobbles and boulders Loose to very dense Brown Moist
	Spoon bouncing on cobble at 2.4 m depth Very hard augering below 2.4 m depth
	Trace silt above 3.7 m depth
	Becoming wet below 4.6 m depth
299.9 / 10.7	END OF BOREHOLE

NUMBER	TYPE	"N" VALUES	REMARKS GR	SA	SI	CL
1	SS	9				
2	SS	78				
3	SS	50/0.03				
4	SS	17				
5	SS	18	28	67	(5)	
6	SS	20				
7	SS	57				
8	SS	56				

Elevation scale: 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300

Notes:

1. Open borehole dry upon completion of drilling

2. Borehole caved at 4.1 m depth (Elev. 306.5 m)

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

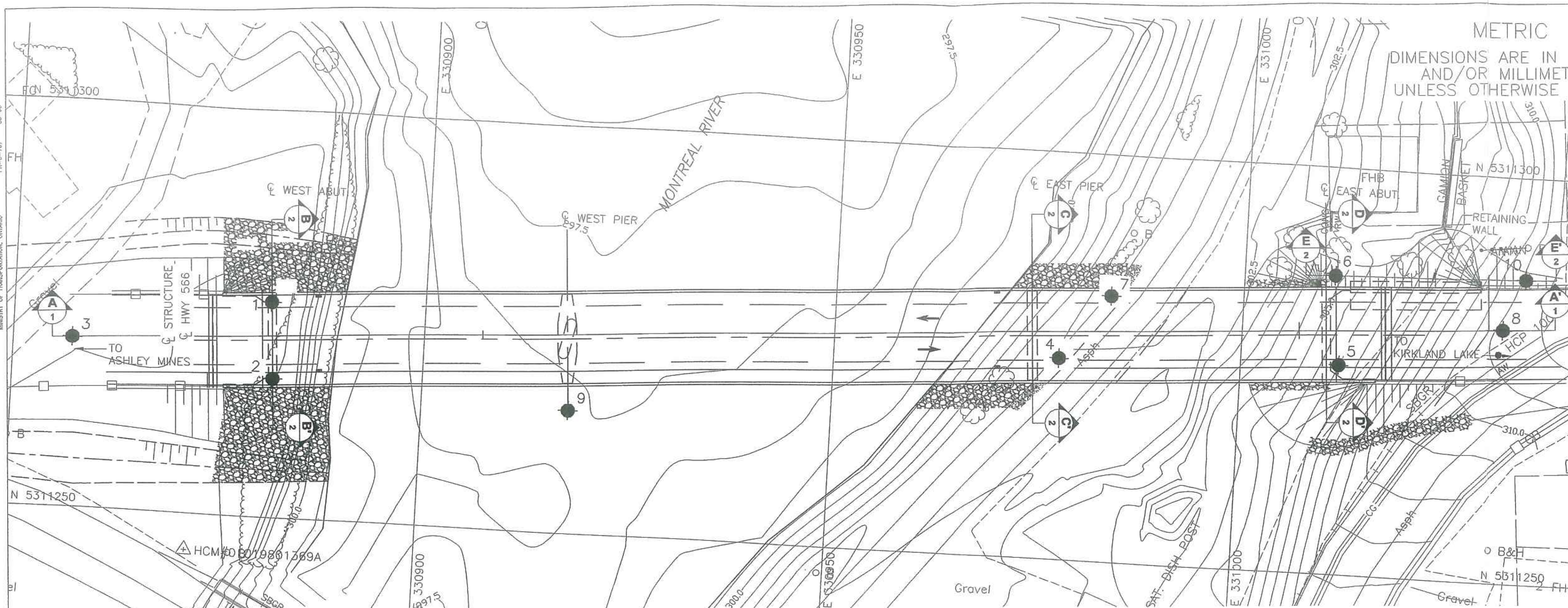
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No 2006-5152
WP No 83-86-00

MONTREAL RIVER BRIDGE
HWY. 566-BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

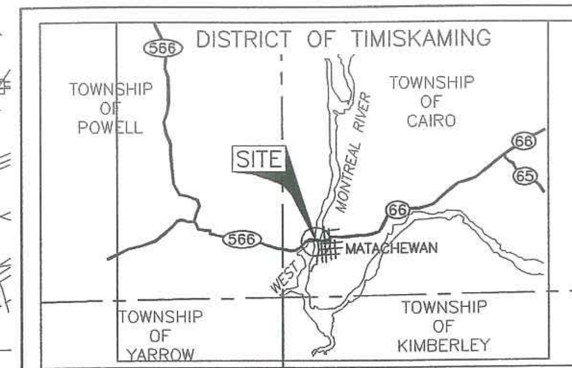
SHEET
—

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



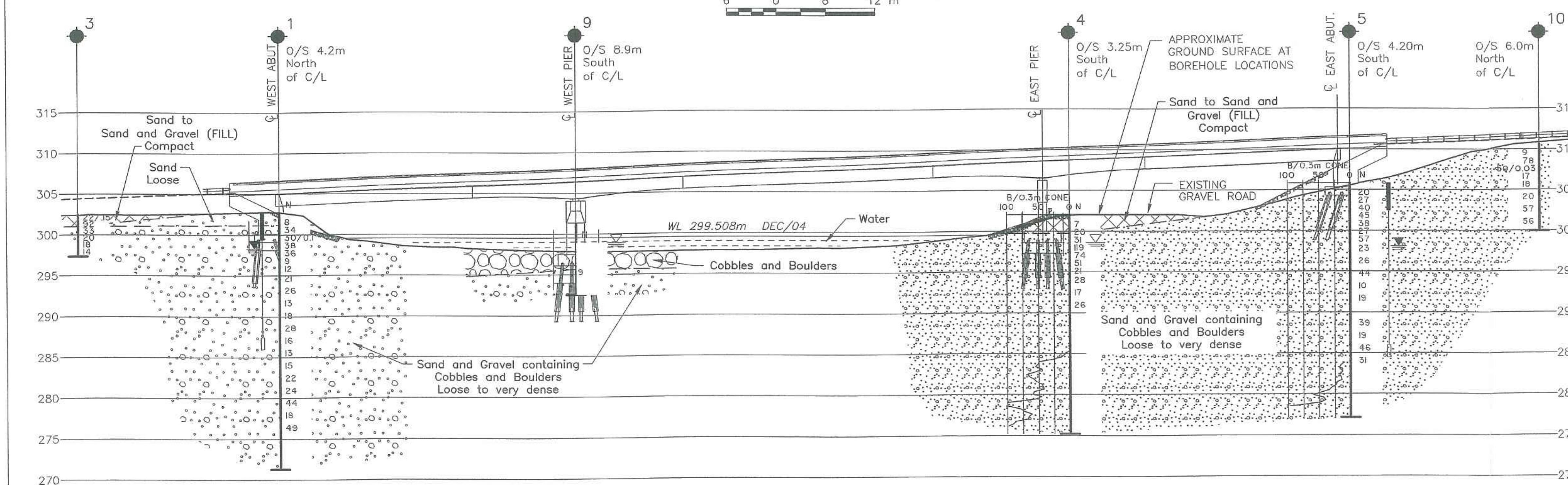
PLAN

SCALE 6 0 6 12 m



KEY PLAN

SCALE 1.6 0 1.6 km



℄ PROFILE MONTREAL RIVER BRIDGE

(A-A' 1)

SCALE 6 0 6 12 m

LEGEND

●	Borehole – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on Aug 23, 2005
	WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
1	302.7	5311276.5	330881.2
2	302.5	5311267.1	330881.8
3	302.5	5311270.9	330857.2
4	302.0	5311274.8	330977.7
5	305.8	5311275.9	331012.0
6	304.6	5311286.8	331011.0
7	302.0	5311282.8	330983.8
8	310.5	5311281.3	331031.9
9	298.9	5311265.2	330918.0
10	310.6	5311287.6	331034.4

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

Base plans provided in digital format by LEA Consulting Ltd., drawing file GA.dwg, recieved November 30, 2005.

Geocres No. 41P-35

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 1/10/06 PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

DESIGN	CHK SEP	CODE CHBDC 00	LOAD ONT CL-625	DATE JANUARY 2006
DRAWN MSM	CHK JMAC	SITE 47-017	STRUCT	SCHEME	DWG 1

DRAWING NAME: T:\Projects\2004\04-1111-056 (LEA, Montreal)\-AD-(FINAL REPORT Client Border)\04111056AD001.dwg
CREATED: OCTOBER 2005 MODIFIED: Jan 10, 2006-11:56am

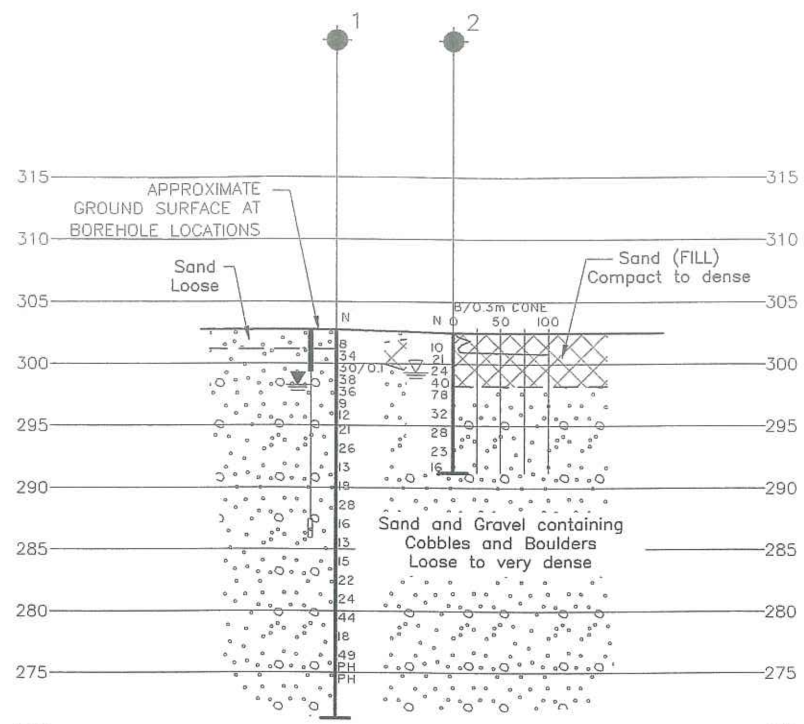
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No 2006-5152
WP No 83-86-00

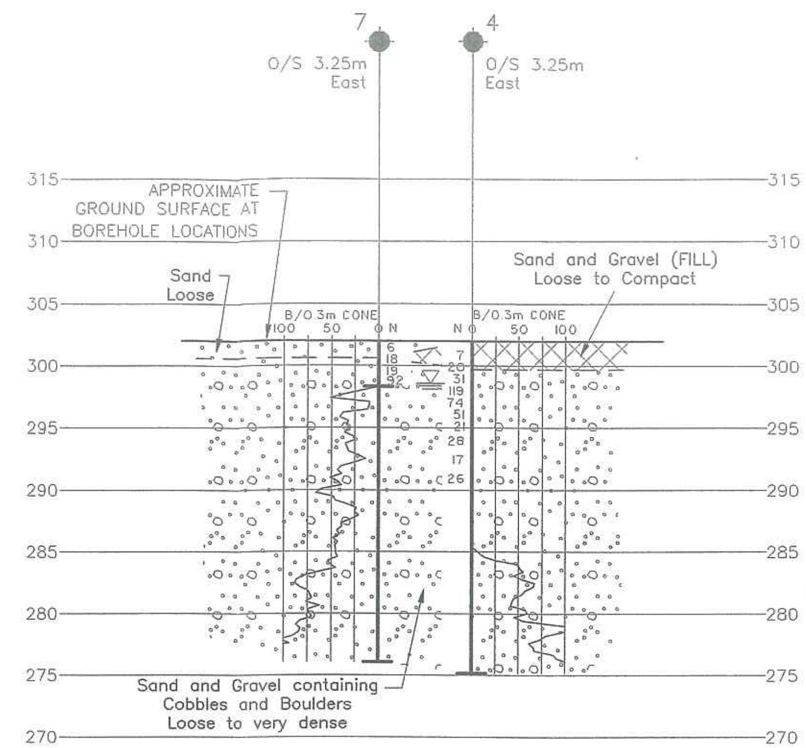
MONTREAL RIVER BRIDGE
HWY. 566-BRIDGE REPLACEMENT
BOREHOLE SOIL STRATA

SHEET —

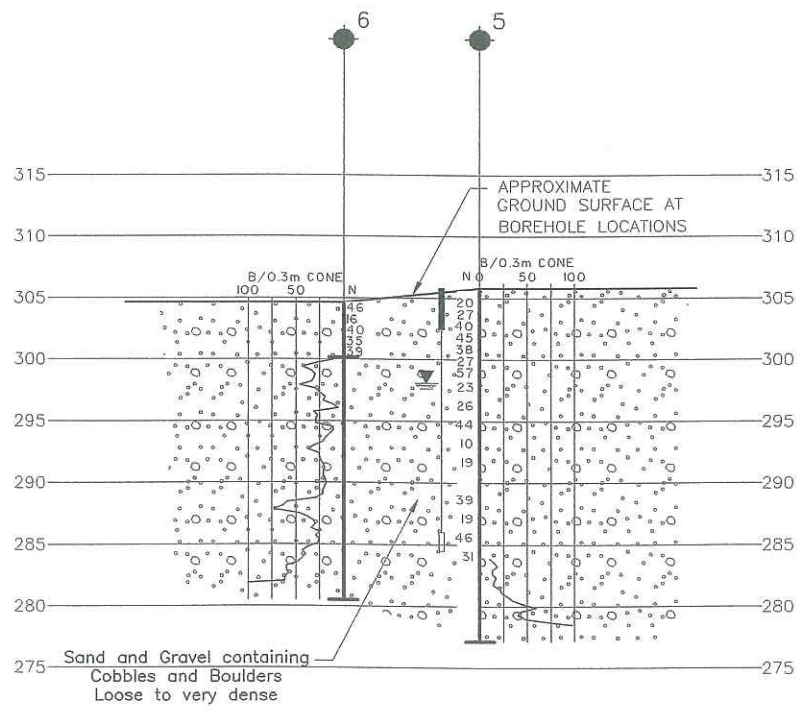
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



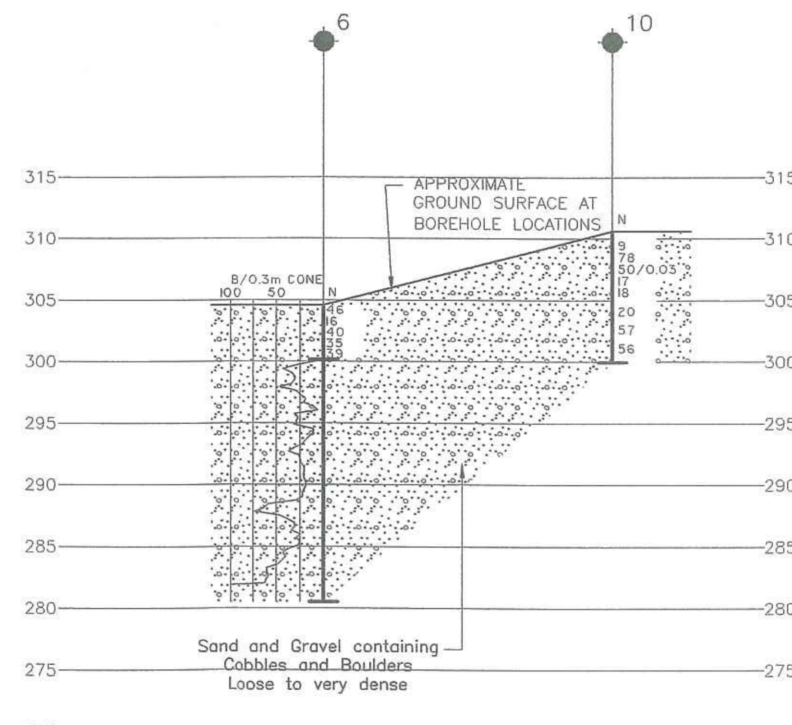
B-B' 1 SECTION ALONG WEST ABUTMENT



C-C' 1 SECTION ALONG EAST PIER



D-D' 1 SECTION ALONG EAST ABUTMENT

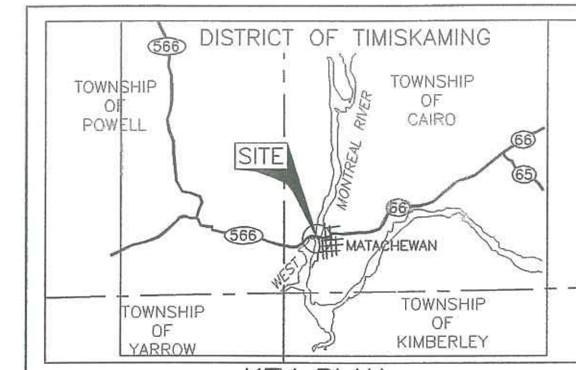


E-E' 1 SECTION ALONG RETAINING WALL

SCALE
6 0 6 12 m

LICENSED PROFESSIONAL ENGINEER M.A. COSTA PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT PROVINCE OF ONTARIO



KEY PLAN
SCALE
1.6 0 1.6 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on Aug 23, 2005
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
1	302.7	5311276.5	330881.2
2	302.5	5311267.1	330881.8
4	302.0	5311274.8	330977.7
5	305.8	5311275.9	331012.0
6	304.6	5311286.8	331011.0
7	302.0	5311282.8	330983.8
10	310.6	5311287.6	331034.4

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REFERENCE

Base plans provided in digital format by LEA Consulting Ltd., drawing file GA.dwg, received August 17, 2005.

Geocres No. 41P-35

REVISIONS	DATE	BY	DESCRIPTION

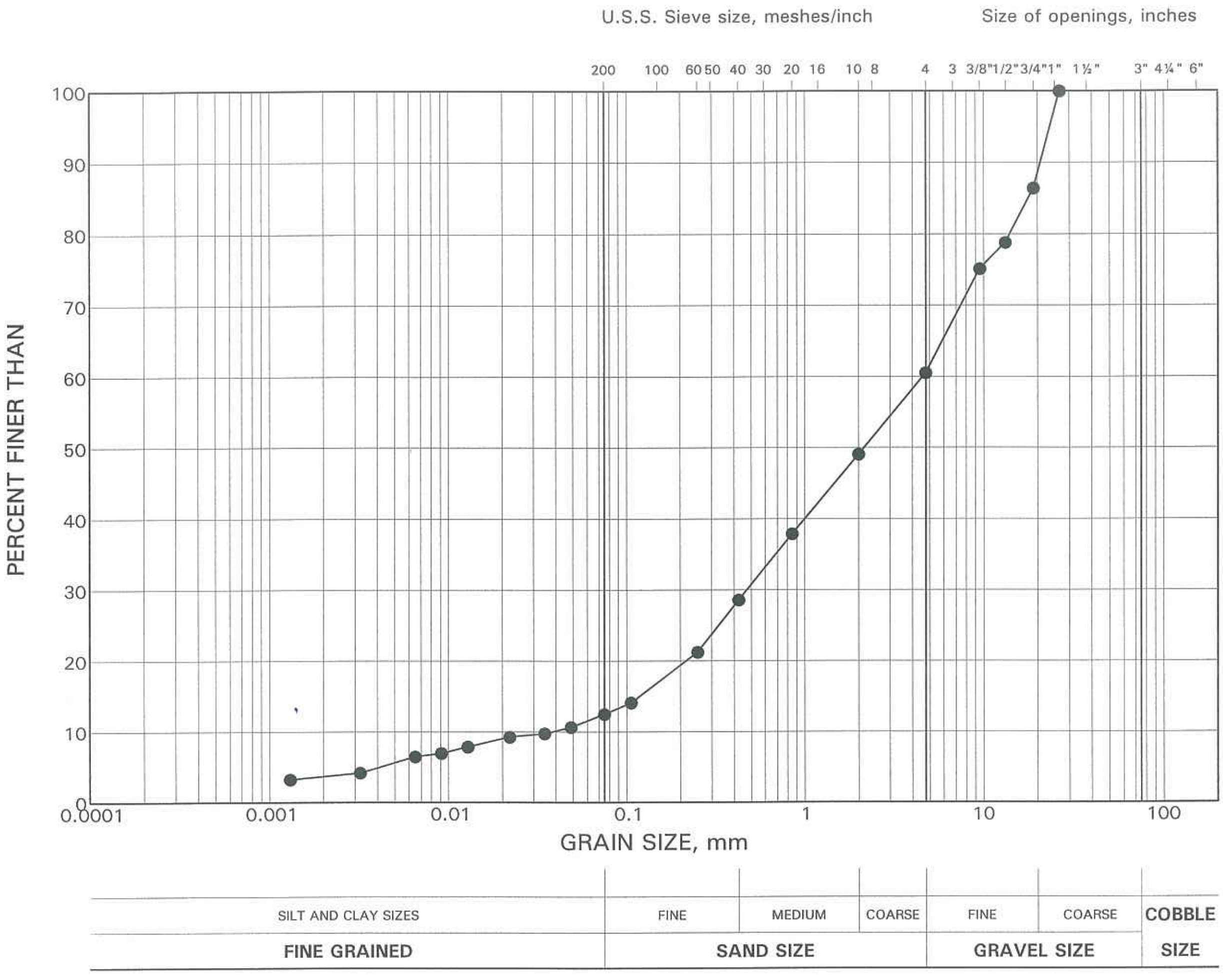
DESIGN	CHK SEP	CODE CHBDC 00	LOAD ONT CL-625	DATE JANUARY 2006
DRAWN MSM	CHK JMAC	SITE 47-017	STRUCT	SCHEME	DWG 2

DRAWING NAME: T:\Projects\2004\04-1111-056 (LEA, Montreal)\-A0-(FINAL REPORT Client Border)\04111105640002.dwg
CREATED: OCTOBER 2005 MODIFIED: Jan 10, 2006-11:55am

MINISTRY OF TRANSPORTATION, ONTARIO

GRAIN SIZE DISTRIBUTION
Sand and Gravel (Fill)

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	2	3	299.9

Date December, 2005

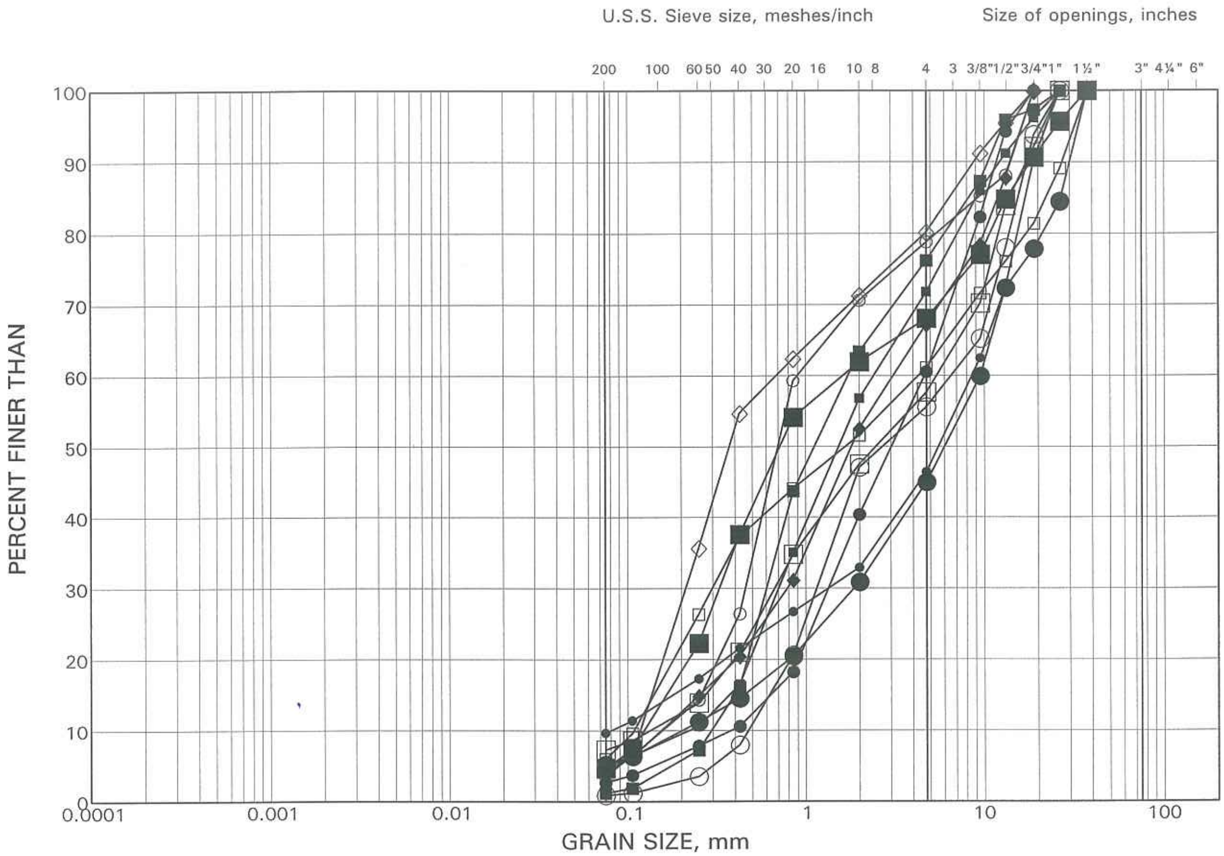
Project 04-1111-056

Golder Associates

Prepared by LG

Checked by ________

GRAIN SIZE DISTRIBUTION Sand and Gravel	FIGURE 2



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

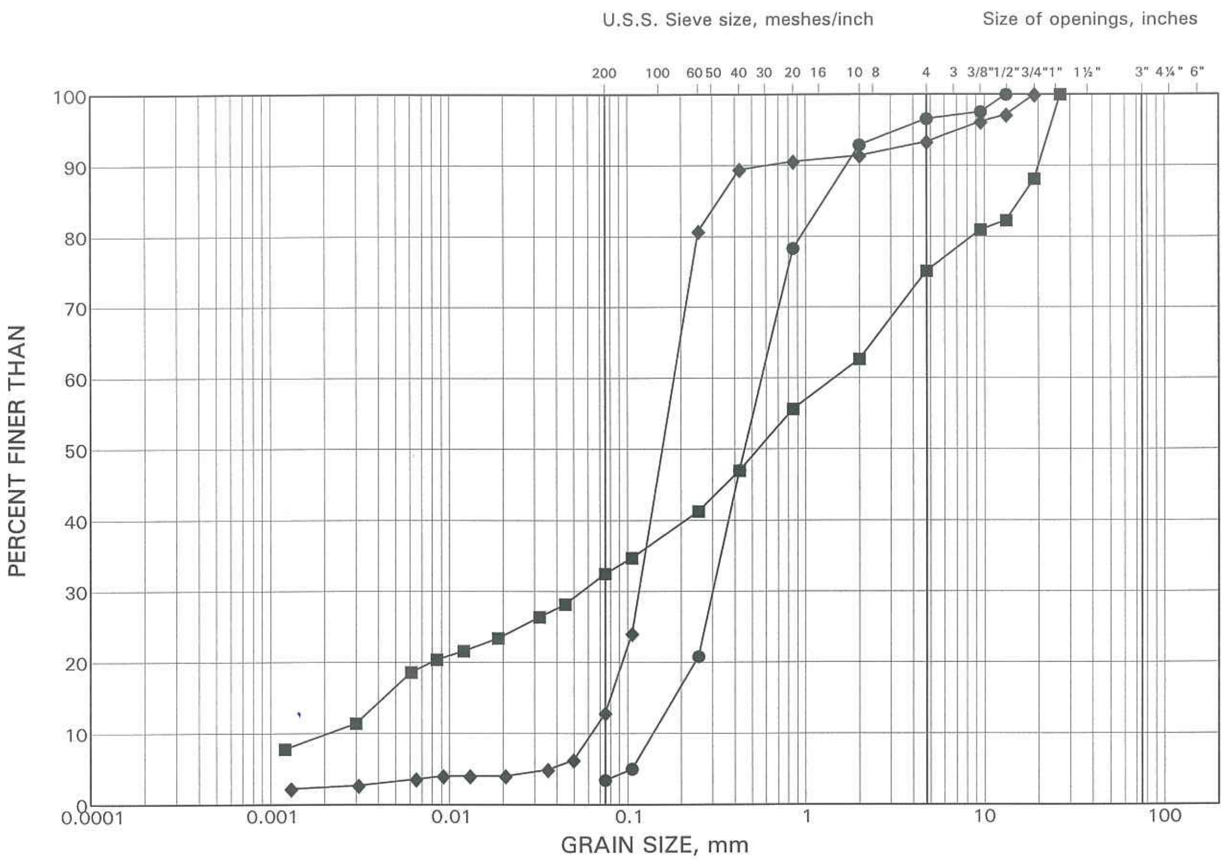
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	1	6	297.8
■	1	13	287.2
◆	2	7	294.6
○	3	4	299.2
□	4	5	297.9
◇	4	7	295.6
⬤	5	3	303.2
■ (large)	5	10	294.8
◯	5	13	277.8
☐	7	2	300.2
•	7	4	298.7
▪	10	5	4.6-5.2

Date December, 2005
Project 04-1111-056

Golder Associates

Prepared by LG
Checked by _______

GRAIN SIZE DISTRIBUTION Sand to Silty Sand (Seams)	FIGURE 3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	1	19	278.0
■	4	3	299.4
◆	6	2	302.8

Date December, 2005
Project 04-1111-056

Golder Associates

Prepared by LG
Checked by ________

View of west shoreline. Photo taken August 25, 2005.

Close up view of west shoreline. Photo taken August 25, 2005.

FIGURE 5

View of west pier location.
Photo taken July 27, 2005.

Close up view of west pier location. Note cobbles and boulders (and wood) at surface. Photo taken August 25, 2005.

APPENDIX A

NON-STANDARD SPECIAL PROVISIONS

ROCK POINTS - Item No.

Non-Standard Special Provision

Scope

As part of the work under the above tender item, the Contractor shall supply TITUS Rock Injector Pile Points on HP 310 x 132 Piles for the Montreal River Bridge Replacement. Piles will be driven through cobbles and boulders.

References

OPSS 906 – Structural Steel

Materials

The pile points shall be of the following:

Product	Manufacturer
HPP-R-12	Titus Steel Company Ltd. 6767 Invader Cr. Mississauga, ON Tel (905) 564-2446

(Or approved equivalent)

Basis of Payment

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

<u>UNWATERING FOR STRUCTURE EXCAVATION</u> - Item No.

Non-Standard Special Provision

Scope

The contractor shall be alerted that the soils at the Montreal River Bride Replacement site consist of water-bearing sand and gravel containing cobbles and boulders. Pile caps construction below the groundwater and/or river water levels must be carried out in the dry. The excavation shall be kept stable during the work.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.

PILES - Item No.
CAISSONS – Item No.
EXCAVATION – Item No.

Non-Standard Special Provision

Scope

As part of the work for the installation of piles and/or caissons as well as excavations for pile caps at the Montreal River Bridge Replacement site, the Contactor shall be alerted that the overburden soils consist of water-bearing sand and gravel containing cobbles and boulders. In addition, the soils will be susceptible to cave-in, sloughing and boiling. Boulders in excess of 750 mm were encountered at this site.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.