



November 30, 2009

REPORT



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**Foundation Investigation and Design Report
Oxtongue River Bridge Replacement and Detour
Highway 60, Site No. 40-002
Township of Algonquin Highlands, Ontario
Ministry of Transportation, Ontario
GWP 5550-04-00**

Submitted to:

URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9



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NSSP	Rock Points
OC	Obstructions and Ground Control
NSSP	Vibration Monitoring



PART A

**FOUNDATION INVESTIGATION REPORT
OXTONGUE RIVER BRIDGE REPLACEMENT AND DETOUR
HIGHWAY 60, SITE NO. 40-002
TOWNSHIP OF ALGONQUIN HIGHLANDS, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5550-04-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Oxtongue River Bridge and Detour on Highway 60 in the Township of Algonquin Highlands, east of Dwight, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P7-1191-0001, dated January 19, 2007, which forms part of the Consultant's Agreement (P.O. Number 5005-E-0077) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated October 16, 2007. The General Arrangement (GA) drawing for the main bridge structure was provided to Golder by URS in September 2008 and updated in February 2009. The GA drawing for the detour structure was provided in September 2008.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement and detour structure locations by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The boreholes for the current investigation were located in the field by Golder relative to the centerline stakes laid out at the site by URS, based on the September 2008 GA's. The location of the investigated area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Algonquin Highlands on Highway 60 crossing Oxtongue River, approximately 12 km east of the village of Dwight, Ontario. The surrounding land is mainly comprised of scattered residences. Grass cover and tree cover extend beyond the limits of the site. An MTO "picnic rest area" is located on the southwest side of the bridge. The river is on a skew from northwest to southeast and is less than 30 m wide at the existing bridge location, increasing in width in the northwest and southeast quadrants.

We understand that the existing Oxtongue River bridge was constructed in 1947 and founded on timber piles. The existing seven span bridge has an overall deck length of about 42.6 m and an overall width of 11.0 m. We understand from URS that the embankments were rehabilitated in 1969 with the installation of gabion walls placed in front of the abutments. In 2003, emergency repairs were made to correct the failure of several timber piles.

The existing highway grade is about 4 m and 5 m above the measured water level (Nov. 2008) at the existing west and east abutments, respectively. The water level in the river was measured at Elevation 364.1 m in November 2008. The highwater level is at Elevation 364.8 m, as shown on the GAs.

3.0 INVESTIGATION PROCEDURES

A total of nine (9) boreholes advanced at the site between November 3, 2008 and January 12, 2009. Four boreholes (OX-1 to OX-4) were advanced for the proposed main bridge abutments and approaches and five boreholes (OX-6 to OX-9) were advanced for the detour bridge abutments, centre pier and approaches. The locations and elevations of the boreholes are shown on Drawings 1 and 3.

Boreholes OX-1 to OX-6, OX-8 and OX-9 were drilled using a track mounted D50 drill rig that was supplied and operated by Walker Drilling Ltd. (Walker) of Barrie, Ontario. Borehole OX-7 was advanced using a D-25 barge mounted drill rig in Oxtongue River supplied and operated by Walker.

The boreholes were advanced using either 108 mm inside diameter (I.D.) continuous flight hollow stem augers and/or NW casing with wash boring. Soil samples were obtained at intervals of depth of about 0.75 m to 2.5 m, using a 50 mm outer diameter (O.D.) split spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Rock core samples were obtained using an 'NQ' size core barrel at OX-2, OX-3, and OX-6 to OX-8.



The boreholes were advanced to depths ranging from 9.8 m to 36.3 m below the existing ground, bridge deck or water surface. Boreholes OX-1, OX-4, OX-5 and OX-9, advanced for the approaches, were terminated prior to reaching refusal. A minimum of 3 m of rock core was obtained from the boreholes drilled at the foundation elements.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in two boreholes, OX-5 and OX-9, at the west and east detour approaches, respectively, to allow monitoring of the groundwater level at these locations. The piezometers consisted of a 50 mm outside diameter rigid PVC tubing with a 1.5 m long slotted screen, sealed within the silt and sand to sandy silt stratum. The remaining boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended by O. Reg. 372) upon completion of drilling. The piezometers were backfilled in a similar manner after a final water level reading was obtained on November 25, 2008. The installation details and water level readings are presented on the Record of Borehole sheets in Appendix A.

The fieldwork 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 and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further 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. In addition, uniaxial compressive strength (UCS) testing was carried out on selected specimens of the bedrock core recovered from the boreholes.

The proposed boreholes were laid out in the field by Golder relative to the proposed centreline alignment and offset stakes in the field surveyed by URS and based on the dimensions shown on the GAs supplied by URS in September 2008. The northings and eastings in MTM NAD 83 were determined by plotting the station and offset of the boreholes (relative to the stakes) on the September 2008 GAs and converting to the coordinate system. The ground surface and water surface elevations at the borehole locations were surveyed by Golder relative to the centerline stakes and are referenced to geodetic datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the McClintock Domain of the Algonquin Terrane, which is located in the Grenville Province (Geology of Ontario; OGS Special Volume 4)¹. The bedrock of this domain generally consists of metasedimentary gneiss in granulite facies.

Based on terrain mapping (Ontario Geological Survey)², the subsurface soils in the vicinity of the site consist of glaciofluvial plains comprising sandy deposits.

¹ Geology of Ontario, 1991. Ontario Geological Survey, 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.

² Southern Ontario Engineering Geology Terrain Study



4.2 Subsurface 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 and rock samples, are given on the attached Record of Borehole and Drillhole sheets in Appendix A. The results of the laboratory tests carried out on selected soil and rock samples are presented in Appendix B. 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 borehole investigation at the bridge locations are shown on Drawings 1 to 3.

In general, the subsoils at the structure site consist of sand to sand and gravel fill underlain by strata consisting of sand, silty sand, silt and sand, sandy silt and/or silt. These deposits are underlain at depth by a sand to sand and gravel deposit, containing cobbles and boulders within a few meters of the bedrock surface. The thickness of overburden was between 21.4 m and 32.9 m, being thicker on the east side of the river.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

In boreholes OX-1 to OX-4 (drilled through the existing roadway surface or the existing bridge deck), approximately 90 mm to 150 mm of asphalt was encountered. In boreholes OX-2 and OX-3, approximately 200 mm of concrete was encountered below the asphalt on the bridge deck. The road surface at these boreholes ranges from Elevation 367.8 m to 369.6 m.

All the boreholes at the site were advanced through the existing embankment fill. Boreholes OX-2 and OX-3 were advanced through the bridge deck, having an approximate 2.3 m air void between the deck and the fill. The upper 0.2 m of fill in Borehole OX-4 consisted of Granular A, B and chip and tar materials. Elsewhere the fill generally consisted of moist to wet, brown sand to gravely sand containing trace to some silt and trace clay. A small layer of grey, moist silt fill containing some clay and sand was noted within the fill in Borehole OX-6. Organics were noted in the upper 0.6 m of fill in Boreholes OX-5 and OX-6. Cobbles were noted within the fill in Boreholes OX-3, OX-4 and OX-7, on the east side of the river. The fill was noted to be layered and oxidized in Boreholes OX-5 and OX-6 on the west side of the river. The top of the fill stratum was encountered from Elevation 362.0 m to 370.4 m and the thickness ranged from 1.4 m to 5.0 m.

SPT 'N' values measured within the fill stratum typically ranged from 2 to 21 blows per 0.3 m of penetration indicating a very loose to compact relative density. In Borehole OX-5, 'N' values of 47 and 63 were encountered near the base of the fill, indicating the fill becomes very dense in this borehole.

Grain size distribution tests were carried out on several samples of the fill stratum and the results are shown on Figure B-1.

The natural water content measured on samples of the fill stratum range from about 4 percent to 25 percent.



4.2.2 Upper Sand to Silty Sand

Below the embankment fill in Boreholes OX-1, OX-2, OX-4 and OX-9, a deposit of moist, brown sand to silty sand containing trace silt and trace gravel was encountered. The top of this deposit ranged from Elevation 368.3 m to 361.5 m and the thickness ranged from 1.2 m to 5.1 m.

SPT 'N' values measured within the upper sand to silty sand deposit ranged from 4 to 28 blows per 0.3 m of penetration indicating a very loose to compact relative density.

Grain size distribution tests were carried out on several samples of this deposit and the results are shown on Figure B-2.

The natural water content measured on samples of the upper silt sand to silty sand deposit range from about 4 percent to 24 percent.

4.2.3 Silt and Sand to Sandy Silt

A deposit of moist to wet, light brown to grey, silt and sand to sandy silt containing trace clay and trace gravel was encountered below the upper sand to silty sand in boreholes OX-1, OX-2 and OX-4 and below the fill in Boreholes OX-3 and OX-5 to OX-8. The top of the deposit ranged from Elevation 360.3 m to 368.4 m and the thickness of the deposit ranged from 6.1 m to 18.9 m where it was fully penetrated. Boreholes OX-1, OX-4 and OX-5 were terminated within this stratum.

SPT 'N' values measured within the silt and sand to sandy silt deposit ranged from 2 to 35 blows per 0.3 m of penetration indicating a very loose to dense relative density.

Grain size distributions were carried out on several samples of the silt and sand to sandy silt deposit and the results are shown on three B-3 Figures.

The natural water content measured on samples of the silt and sand to sandy silt deposit range from about 10 percent to 30 percent.

A 1.5 m thick layer of sand containing trace silt was encountered within the silt and sand to sandy silt deposit in Borehole OX-6. The grain size distribution for this sample is shown on Figure B-5.

4.2.4 Silt to Sandy Silt

A 3.0 m thick deposit of wet, brown, silt containing some sand, trace clay and sand layers was encountered below the silt and sand to sandy silt deposit at Elevation 361.0 m in Borehole OX-8. In Borehole OX-9, this deposit was classified as a sandy silt and was encountered at Elevation 363.2 m; Borehole OX-9 terminated within this deposit after 2.6 m. In Boreholes OX-2 and OX-4, a 1.5 m to 2.2 m thick seam of wet, brown silt containing trace sand was encountered within the silt and sand to sandy silt deposit at Elevation 354.4 m and 362.4 m, respectively.

SPT 'N' values measured within the silt ranged from 4 to 17 blows per 0.3 m of penetration indicating a loose to compact relative density.

Grain size distribution tests were carried out on samples of the silt and the results are shown on Figure B-4.

The natural water content measured on samples of the sand and silt seam range from about 24 percent to 27 percent.



4.2.5 Sand to Sand and Gravel

Beneath the silt and sand to sandy silt stratum in Boreholes OX-2, OX-3, OX-6 and OX-7, and beneath the silt in Borehole OX-8, a deposit of wet, brown, sand to sand and gravel containing trace silt was encountered. The top of this deposit was encountered from Elevation 345.5 m to 358.0 m and the thickness of the deposit was between 8.1 m and 13.9 m. Cobbles and boulders were typically encountered throughout the deposit and with more frequency within about 3 m of the bedrock surface. Boulders were cored in Boreholes OX-3, OX-6 and OX-7 for 0.3 m to 1.1 m thickness and were encountered between Elevation 336.1 m and 342.8 m.

SPT 'N' values measured within the sand to sand and gravel deposit ranged from 7 to 69 blows per 0.3 m of penetration indicating a loose to very dense relative density. The higher 'N' values were typically measured near the base of the deposit.

Grain size distribution tests were carried out on several samples, retrieved within the 36.5 mm I.D. sampler, of the sand to sand and gravel deposit and the results are shown on Figure B-5. These tests do not reflect the full range of particle sizes, particularly the coarser gravel, cobbles and boulders.

The natural water content measured on samples of the sand to sand and gravel deposit range from about 8 percent to 27 percent.

4.2.6 Bedrock

Bedrock was encountered in Boreholes OX-2, OX-3 and OX-6 to OX-8. The depth to and elevation of the bedrock surface in these holes is summarized below.

Location	Borehole	Depth to Bedrock Surface* (m)	Bedrock Surface Elevation (m)
Main Bridge WA	OX-2	32.6	335.5
Main Bridge EA	OX-3	27.6	341.5
Detour Bridge WA	OX-6	32.9	333.3
Detour Bridge Pier	OX-7	23.6	340.7
Detour Bridge EA	OX-8	29.7	340.0

* Depth below ground surface, bridge deck or water surface

Based on a review of the bedrock core samples, the bedrock at the site generally consisted of pink/grey fine to medium grained, fresh to slightly weathered, gneiss. Healed, partially healed and open joints were noted as well as layers, banding and foliation. In Boreholes OX-6 and OX-7 the gneiss comprised feldspar, quartz, amphibole and biotite with granitic alterations.

The Rock Quality Designation (RQD) measured on the core samples typically ranged from about 80 percent to 100 percent, indicating rock mass of excellent quality. In Boreholes OX-3 and OX-8, RQD was measured between 23 and 65 percent in the upper portion of the rock core, increasing with depth. The Total Core Recovery (TCR) during bedrock coring was generally 100 percent.

Laboratory UCS testing was carried out on four samples of the gneiss bedrock from Boreholes OX-3 and OX-6 to OX-8. The UCS results range between about 57 MPa and 136 MPa, indicating strong to very strong rock, using the "Intact Rock Strength Classification" table. The depths and corresponding elevations of the tested samples and results of the UCS testing are presented in Table B-1, in Appendix B.



4.2.7 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed with screened sections sealed within the silt and sand to sandy silt deposit in Boreholes OX-5 and OX-9. Details of the piezometer installations are shown on the Record of Borehole Sheets reported in Appendix A. In general, the soil samples taken in the boreholes were noted to be moist to wet.

The water level of Oxtongue River was measured at Elevation 364.3 m in November 2008 during the field investigation by Golder. The water level was measured by others in April 2008 at Elevation 364.1 m. The water levels in the piezometers and open holes during drilling and upon completion of drilling are summarized below and they are between 1.2 m higher and 0.9 m lower than the November 2008 river level.

Borehole	Installation	Groundwater Level Depth (m)	Groundwater Level Elevation (m)	Date
OX-1	Open Borehole	4.4	363.4	January 12, 2009
OX-2	Open Borehole	3.8	364.3	November 21, 2008
OX-3	Open Borehole	3.6	365.5	November 18, 2008
OX-4	Open Borehole	5.6	364.0	November 17, 2008
OX-5	Piezometer	3.2	363.9	November 25, 2008
OX-6	Open Borehole	1.6	364.9	November 11, 2008
OX-7	Open Borehole	0.0	364.3	November 3, 2008
OX-8	Open Borehole	5.0	364.7	November 13, 2008
OX-9	Piezometer	5.6	364.8	November 25, 2008

It should be noted that the groundwater levels in the area are subject to seasonal fluctuations and after precipitation events. The high water level for the Oxtongue River is Elevation 364.8 m.

5.0 CLOSURE

The field personnel supervising the drilling program was Mr. Ed Savard and Mr. Trevor Moxam. This report was prepared by Mr. Tim Rancourt, EIT, and Mr. André Bom, P.Eng., and the technical aspects were reviewed by Mrs. Sarah Coyne, P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Fintan J. Heffernan, P.Eng., Golder's Designated MTO Contact for this project.



Report Signature Page

GOLDER ASSOCIATES LTD.

Timothy Rancourt
Geotechnical EIT



Sarah E. M. Coyne, P.Eng.
Associate, Senior Geotechnical Engineer

TR/SEMC/FJH/lb



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



PART B

**FOUNDATION DESIGN REPORT
OXTONGUE RIVER BRIDGE REPLACEMENT AND DETOUR
HIGHWAY 60, SITE NO. 40-002
TOWNSHIP OF ALGONQUIN HIGHLANDS, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5550-04-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed new Highway 60 bridge structure and associated detour structure over Oxtongue River. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site. The interpretation and recommendations presented 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.

6.1 General

The existing bridge carrying Highway 60 over Oxtongue River is a seven span structure with an overall deck length of about 42.6 m and width of about 11.0 m. The structure is supported on timber piles to an unknown depth likely terminating within the overburden, based on the depth to bedrock where encountered at the boreholes. We understand from URS that the existing structure was constructed in 1947 and the embankments were rehabilitated in 1969 with the installation of gabion walls placed at the front of the abutments. In 2003, emergency repairs were made to correct the failure of several timber piles.

The existing west and east approach embankments at the abutments are about 4 m and 5 m above the measured water level (November 2008), respectively. The river is less than 30 m wide at the existing highway crossing location as the approach embankments appear to have been extended into the river.

We understand the new bridge will be located on the existing alignment with traffic diverted onto a single lane modular (ACROW) detour bridge south of the highway. The proposed 12 m wide main bridge will have one 37 m long span with the abutments located slightly in front of and skewed to the existing bridge abutments. The proposed grade of Elevation 369.2 m and 370.0 m at the west and east abutments is about 1 m higher than the existing highway grade. The new west and east approach embankments will be up to 5.1 m and 5.9 m above the November 2008 water level of Elevation 364.1 m, respectively.

The 9 m wide temporary detour bridge will have an overall length of 79.2 m and a grade at the west and east abutments of Elevation 369.1 m and 371.1 m, respectively. This results in embankments up to 3.6 m and 1.6 m above the ground surface at the west and east detour approaches, respectively.

The subsurface conditions generally consist of embankment fill over cohesionless deposits of sand to silt and sand followed by sand and gravel. Cobbles and/or boulders were encountered within the lower deposits immediately overlying gneiss bedrock. At the investigated locations for the proposed abutments for both the main bridge and detour bridge, the total overburden thickness, including fill materials ranged between 25.0 m and 29.8 m. At the centre pier for the detour bridge, the overburden thickness below the bottom of the riverbed was 21.4 m. The bedrock surface varied between Elevation 333.3 m and 341.5 m, generally sloping from the west to east and north to south.

The recommendations on the foundation design aspects of the new structure presented in this report take into consideration the impact of the detour bridge foundations and approach embankments on the existing bridge foundations and approach embankments during construction as well as the removal of the existing and detour bridge.



Steel H-piles driven to bedrock are the preferred founding alternative for support of the main bridge because of the loose nature of the overburden. Shallow foundations are not considered feasible for this bridge. Table 1 summarizes the advantages, disadvantages, relative costs and risks/consequences of the two deep foundation alternatives for the main bridge: steel H-piles driven to bedrock or caissons socketted into the bedrock. Discussion on the alternatives is given in the sections below.

Steel H-piles driven to the bedrock surface are recommended for support of the abutments and central pier of the detour. However, shallow spread footings supported on a granular pad constructed over the native material could also be considered for support of the detour abutments. Table 2 summarizes the advantages, disadvantages, relative costs and risks/consequences of the founding alternatives for the detour bridge.

6.2 Shallow Foundations

Given the thickness of the very loose to compact relative density of the overburden soils at the main bridge site within the potential shallow foundation zone, spread footings founded at shallow depth on either the native soil deposits or perched within the existing (or new) embankment fill are not recommended to support the new main bridge structure due to the low geotechnical axial resistance and expected settlement of these strata.

At the detour bridge abutments, the native silt and sand to sandy silt is compact within the shallow foundation zone of influence and is suitable for support of a granular pad on which the abutments can be founded. Shallow foundations are not recommended at the central pier of the detour due to the filling that would be required in the river.

Consideration could be given to founding the detour bridge abutment footings on a granular pad extending to the surface of the native silt and sand to sandy silt. The surface of the native silt and sand to sand is at Elevation 364.5 m and 368.4 m at the west and east detour abutment locations, respectively. The overburden should be removed to this elevation and the granular pad constructed to the underside of footings, a minimum of 2 m above the native soil. The pad will be 2.8 m and 0.8 m thick at the west and east abutments, respectively, where the underside of the footing is proposed to be at Elevation 367.3 m and 369.2 m. Since the pad at the east abutment will be less than 2 m if only the fill is removed, additional sub-excavation of the native material is required such that the underside of the granular pad is at Elevation 367.2 m. Excavations for pad construction are expected to be at or above the water level as discussed further in Section 6.9.2.

In both cases, the thickness of the pad is greater than the frost depth of 1.8 m, as given in Section 6.2.3, although the detour is not expected to be in use for more than one construction season (spring to fall).

6.2.1 Geotechnical Axial Resistance

For spread footings placed (or perched) within the approach embankments on a compacted Granular 'A' core, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 850 kPa may be used. A corresponding Serviceability Limit States (SLS) value of 350 kPa may be used assuming a 2 m to 3 m wide footing. These values assume a minimum 2 m thick granular pad placed below the base of the footing placed directly on the native silt and sand to sandy silt material. The granular pad should extend at least 1 m beyond the plan limits of the footing and be sloped no steeper than 1 Horizontal:1 Vertical (1H:1V) in general accordance with MTO guidelines and Figure 1. The granular pad should be constructed in accordance with MTO Special Provision 105S10 (Compaction). MTO SP 902S01 (Excavation and Backfilling) should also be included in the Contract Documents.

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*.



6.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the mass concrete and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.70 between the concrete and the compacted granular pad. This value represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.2.3 Frost Protection

All footings should be provided with a minimum of 1.8 m of conventional soil cover for frost protection (OPSD 3090.100). 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, 25 mm of rigid polystyrene insulation is assumed to be equivalent to about 300 mm of soil cover.

6.3 Steel H-Pile Foundations

Based on the borehole information obtained at this site, steel H-piles driven to bedrock are recommended for support of the main bridge abutments and detour bridge abutments and central pier. We understand an integral abutment design is being considered for the main bridge at this site. Details of founding elevations are given below for each of the bridges.

Main Bridge

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design for the main bridge (through which the piles will be driven), the CSPs should be backfilled with a loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix C.

For design, the estimated tip elevations for the piles terminating on the bedrock surface are presented below. The elevations are based on the depth to bedrock encountered in the boreholes advanced for the proposed abutments and an underside of pile cap at Elevation 363.3 m and 364.1 m for the proposed west and east abutment, respectively, as shown on the General Arrangement drawing.

Foundation Unit	Borehole Numbers	Bedrock Surface Elevation (m)	Approximate Design Pile Length (m)
West Abutment	OX-2	335.5	27.8
East Abutment	OX-3	341.5	22.6

The elevations given above should be assumed to be the design pile tip elevations. However, practically, the piles could “hang up” on the cobbles and boulders deposit overlying the bedrock at the west abutment. The zone where hard driving may be encountered is within 1.7 m and 1.3 m of the bedrock surface at the west and east abutments, respectively.



Detour Bridge

For design, the estimated tip elevations for the piles terminating on the bedrock surface are presented below. The elevations are based on the depth to bedrock encountered in the boreholes advanced for the proposed abutments and central pier and the top of the pile at Elevation 367.1 m, 367.7 m and 369.1 m for the proposed west abutment, central pier and east abutment, respectively, as shown on the General Arrangement drawing.

Foundation Unit	Borehole Numbers	Bedrock Surface Elevation (m)	Approximate Design Pile Length (m)
West Abutment	OX-6	333.3	33.8
Central Pier	OX-7	340.7	27.0
East Abutment	OX-8	340.0	29.1

The elevations given above should be assumed to be the design pile tip elevations. However, practically, the piles could “hang up” on the cobbles and boulders deposit overlying the bedrock at the west abutment. The zone where hard driving may be encountered is within 2.8 m, 2.1 m and 0.6 m of the bedrock surface at the west abutment, central pier and east abutment, respectively.

At the centre pier of the detour bridge, approximately 5 m of pile will be exposed above the river bed. The structural design should consider the absence of soil around these piles. In order to prevent buckling of the piles the piles could be surrounded by a protective casing (i.e. steel tube filled with concrete).

6.3.1 Geotechnical Axial Resistance

For the support of the main bridge and detour bridge foundation elements, HP 310X110 piles driven to bedrock could be used and a factored geotechnical axial resistance at ULS of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

6.3.2 Downdrag

The subsoils at this site consist of cohesionless soils. Downdrag loads will not need to be taken into account for design of the piles supporting the foundation elements.

6.3.3 Set Criteria

All pile installation/driving should be in accordance with the latest Special Provision, SP903S01. The piles should be provided with rock points, Titus Injector or equivalent for adequate seating on the sloping bedrock surface. An NSSP should be included in the Contract Documents to address the requirements for rock points; an example is included in Appendix C.



For piles to be driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the piles.

Based on our experience, consideration should be given to the following preliminary criteria. The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven by increasing the hammer energy slowly in stages up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of the seating of the pile on the bedrock surface, which is sloping at this site. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy.

6.3.4 Pile Driving Note

The pile driving note to be added to the drawings for this project is Note 4 in Clause 2.5.11 of the Structural Manual:

“Piles to be driven to bedrock”.

6.3.5 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles only are used, such as in integral abutments, 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 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 resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equation for cohesionless soils given below.

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

where: n_h = the constant of horizontal subgrade reaction (kPa/m)
 z = the depth (m)
 B = the pile diameter or width (m).

For the very loose to very dense cohesionless soils (below the groundwater table) at this site, an average value for n_h of 4,400 kPa/m may be assumed in the structural analysis of the lateral pile deflections.

It is understood that an integral abutment foundation design is being considered for the main bridge. In this case, the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand) and the upper portion of the H-piles will be free to flex and move laterally. With this design, the passive lateral resistance over the length of the CSP liner should be neglected.



At the proposed main bridge foundation elements, the lateral resistance of the piles will be developed from the passive resistance of the soil over the portion of the piles below the CSP liners (where installed). For a single HP 310X110 pile surrounded by a 3 m long CSP liner below the pile cap and embedded into the cohesionless soils below the CSP at the abutments, the estimated maximum lateral resistance at ULS is about 110 kN and at SLS is about 40 kN (for 10 mm of deflection and assuming a steel yield strength of 300 MPa). If piles are used at the detour bridge abutments, CSP liners will not be used and the lateral resistance values specified above may be taken along the full embedment of the pile.

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 at ULS. At 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 as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments (CHBDC Commentary C6.8.7.1).

The upper zone of soil (down to a depth below the pile cap equal to about $1.5 \times B$ after Broms (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 be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2), 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

The subgrade reaction reduction factor should be interpolated for pile spacing in between those listed above.

6.3.6 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of conventional soil cover for frost protection (per OPSD 3090.101). 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, 25 mm of rigid polystyrene insulation is assumed to be equivalent to about 300 mm of soil cover.

6.4 Caissons

Consideration could be given to the use of caissons for support of the foundation elements although caissons are not considered to be practical at all at this site due to the depth to bedrock. The high axial capacity of the caissons would result in fewer units being required to support the abutments than that required for the H-pile design and the possible elimination of a pile cap. It should be noted, however, that there will be difficulty in socketting the large diameter caissons within the strong to very strong gneiss bedrock and achieving an



adequate seal. Temporary liners and tremie concrete will likely be required to install caissons at this site. As well, the presence of cobbles and boulders above the bedrock may also provide some difficulties in caisson installation.

Based on our experience with caissons socketted into bedrock at similar sites, it is possible that the caissons could be advanced with drilling equipment consisting of down-hole hammering system consisting of an external liner advanced as the bit is rotated and driven into the bedrock.

6.4.1 Geotechnical Axial Resistance

If caissons are considered as a founding alternative, the caissons at this site will derive their axial resistance mainly from the shaft resistance of the rock socket. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets. The factored geotechnical axial resistance at ULS for two different caisson diameters socketted a minimum of 2 m into the gneiss bedrock are given below.

Caisson Diameter(m)	Gneiss Bedrock (minimum 2 m socket)	
	ULS (kN)	SLS for 25 mm
1.0	5,000	n/a
1.5	8,000	n/a

The resistance required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

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

6.4.2 Downdrag

As discussed in Section 6.3.2, downdrag loads would not need to be considered for caissons at this site.

6.4.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.4.1 and Table 2, using the horizontal subgrade reaction formulas. The recommended maximum lateral resistances for the caissons are as follows.

Caisson Diameter (m)	Factored Lateral Resistance at ULS (kN)	Lateral Resistance at SLS (kN)
1.0	950	350
1.5	2,200	800



6.4.4 Frost Protection

The caisson caps for the caissons at the abutments should be provided with a minimum of 1.8 m of conventional soil cover for frost protection or sufficient insulation as described in Section 6.3.6.

6.5 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site, in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.5, consistent with Soil Profile Type III.

6.6 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.

6.6.1 Static

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) 1010 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 Ontario Provincial Standard Drawings (OPSD) 3101.150 and 3121.150.
- 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 fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200.
- 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 6.6. Compaction equipment should be used in accordance with OPSS 501.06 or SP 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (as outlined on Figure C6.20(a), Case I, 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 base of the footing/pile cap (as outlined in Figure C6.20(b), Case II, of the Commentary to the *CHBDC*).



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- 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	0.36

- 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 the following (in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the *CHDBC*):

- rotation (i.e. ratio of wall movement to wall height) 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 a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.6.2 Dynamic

The potential for seismic (earthquake) loading must also be considered (if applicable) for the design of abutment stems/retaining walls 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 may 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 Table A3.1.7 of the *CHBDC*, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the Huntsville area is 0.05. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$), resulting in an increase in the ground surface acceleration from 0.05 g to 0.075 g (PHA).



We understand that this highway route/bridge is not designated as a lifeline bridge. As such, based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned to Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, structures located in Seismic Performance Zone 1 need not be analysed for seismic loads.

6.7 Approach Embankment Design and Construction

The new bridge will be replaced along the same alignment as the existing bridge with a final grade of Elevation 369.2 m and 370.0 m, at the west and east approaches, respectively, resulting in a grade raise of about 1 m. The new embankment will be approximately 5 m and 6 m above the river water surface at the west and east abutments, respectively. Further, the new bridge will be about 1 m wider than the existing bridge which will result in minor widening of the existing approach embankments. The existing embankment side slopes on the northwest and southeast embankment slopes are about 1.6H:1V and 1.3H:1V, respectively. This is considered to be very steep for granular fill. The existing west and east front slopes are at about 3H:1V and 1.6H:1V, respectively. We understand gabions were added to the top of the slope at the east abutment in 1969.

The west and east approach embankments of the detour structure will be at a final grade of Elevation 369.1 m and 371.1 m, respectively, which is up to 3.6 m and 1.6 m above the existing ground, respectively. The front face of the proposed west and east abutment centrelines will be about 12 m and 40 m back from the river shoreline, respectively.

The soils encountered below the proposed approach embankments consisted primarily of cohesionless fill overlying cohesionless soils consisting of sand, silty sand, sandy silt, silt and sand and sand and gravel.

For the purpose of analysis, both granular fill and rock fill have been considered for the construction of the approach embankments using side slopes at 2H:1V and 1.25H:1V, respectively. Although granular fill is locally available, we understand from URS that rock fill is being considered for this site to minimize fill encroachment into the river.

The piezometric conditions were assessed based on the water levels observed in Oxtongue River in April 2008 (Elevation 364.1 m) and the groundwater levels noted during drilling of the boreholes in and immediately adjacent to this area. For design purposes in our analysis, the groundwater level has been assumed to be consistent with the minimum river level, at about Elevation 364.1 m. The analyses were checked against the high water level of 364.8 m.

The following sections present the stability and settlement analysis that was carried out at the site. The proximity of the new, existing and detour bridges and approach embankments has also been taken into consideration.

6.7.1 Stability

Analyses were performed on the critical sections of the proposed approach embankments to assess the stability and liquefaction potential for the proposed embankment height and geometry and soil stratigraphy. The critical embankment sections at this site are the front slopes (into the river) and the northwest and southeast side slopes as shown on Figure 2. For the detour structure, only minimal filling is required above the existing ground and, therefore, the west abutment front slope is considered the most critical. The geometry of the proposed approach embankments, existing ground surface and existing riverbed included in the analyses is based on the information from the General Arrangement drawing and survey information provided by URS.



6.7.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design. Block and wedge surfaces were also analyzed in some cases.

6.7.1.2 Parameter Selection

For the cohesionless and cohesive layers, effective stress parameters were employed in the analysis assuming drained conditions. The effective stress parameters (effective friction angle and cohesion) for these soils were estimated from empirical correlations using the results of in situ SPTs, in conjunction with engineering judgement considering experience in similar soil conditions.

Summarized below are the simplified stratigraphy and the associated unit weights and friction angles employed for the different soil types in the proposed approach/abutment areas.

Main Bridge and Detour Bridge Approaches

Soil Type	Unit Weight (kN/m³)	Angle of Internal Friction
New Granular Fill	21	35°
New Rock Fill	19	40°
Existing Granular Embankment Fill	20	30°
Upper Sand	20	32°
Silt and Sand to Sandy Silt	20	30°
Sand and Gravel	21	35°
Cobbles and Boulders	19	40°

Note: Oxtongue River water level assumed to be at Elevation 364.1 m (April 2008).

6.7.1.3 Embankment Fill Types

The different embankment fill alternatives (i.e. granular fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils/bedrock), construction cost and time, and ease of construction/availability.



Granular Fill

The main advantage of using granular fill is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than for rock fill slopes. At this site, the use of granular fill could result in encroachment into the river. For this project, acceptable granular fill is considered to be well graded, locally available and/or imported, granular material such as Granular 'B' Type I or II.

Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with a limited filling restrictions, which we understand a concern at this site. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year after construction and at this site, it may not be locally available.

6.7.1.4 Results of Analysis

The results of the stability analyses for the case of the embankment constructed with granular fill are summarized below for the critical sections. The minimum FoS is based on a deep-seated, global trial failure surface that would impact the operation of the roadway. In all cases, the FoS equal to or greater than the target FoS of 1.3 and, therefore, mitigation measures are not required.

Bridge	Approach	Slope	Embankment Height* (m)	Minimum Factor of Safety	Reference Figure #
Main	West	Front	5.1	1.30	3
		North		1.35	4
	East	Front	5.9	1.30	5
		Southeast		1.33	6
Detour	West	Front	5.0	1.74	7

* Embankment height at centre of abutment above the April 2008 water level.

The results of the analyses typically also give shallow surficial slip surfaces with FoS less than 1.3; however, these surfaces are not realistic or representative of failure conditions. Further, as described above, only failure surfaces that would impact the operation of the roadway have been considered for establishing the minimum FoS.

In addition to the analysis summarized above, limit equilibrium analysis was carried out for rock fill embankments with 1.25H:1V. Each of the critical sections list above resulted in FoS > 1.3 except at the north side of the west approach. At this location, in order to achieve FoS > 1.3, either flattening of the rock fill slope to 1.5 H:1V or a 2 m bench would be required. If rock fill is used, it would slightly decrease the requirement for fill encroachment into the river.



6.7.2 Liquefaction Potential and Seismic Analysis

As noted in Section 6.6.2, this site is located in Seismic Zone 1 with a $PHA < 0.08$. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the *CHBDC*, the site is assigned a Seismic Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the *CHBDC*, liquefaction analysis is not required.

6.7.3 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the embankment fill itself.

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed main bridge and detour approach embankments using hand and spreadsheet calculations.

6.7.3.1 Parameter Selection

The immediate compression of the subsoils were assessed by estimating an elastic modulus of deformation based on the SPT 'N' values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments. The geometry used in the analysis is shown on Figures 4 and 6 for the west and east approaches, respectively. The analysis assumes that the existing fill will be removed to the underside of the pile cap.

Soil	Maximum Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
New Granular Fill	6.0 (West and East Approach – main)	21	-
	4.6 (West Approach – detour)		
New Rock Fill	3.9 (East Approach - detour)	19	-
Existing Granular Embankment Fill	1.7 (West Approach – main)	20	E' = 10 MPa
	2.5 (East Approach – main)		
Silt and Sand to Sandy Silt, Silt to Sandy Silt	16.0 (West Approach – main)	19	E' = 15 MPa
	11.9 (East Approach – main)		
	18.9 (West Approach – detour)		
	9.2 (East Approach – detour)		
Sand to Sand and Gravel	9.9 (West Approach – main)	20	E' = 30 MPa
	8.1 (East Approach – main)		
	12.2 (West Approach – detour)		
	18.0 (East Approach – detour)		

The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fill) is presented below and a discussion on the rate of settlement is included.



6.7.3.2 Results of Analysis

The settlement of new Granular 'B' fill at this site is anticipated to be less than 25 mm and will occur during construction. If rock fill is used for the construction of the new embankments for the main bridge, settlement of the rock fill itself will occur, which will be less than 25 mm if placed in accordance with SP206S03.

At the west and east approaches to the main bridge, settlements of the existing fill left in place below the new fill is anticipated to be about 10 mm. Settlement of the foundation soils is anticipated to be about 25 mm. Therefore, the total estimated settlement of the fill and native soils will be less than about 50 mm.

Settlement of the foundation soils is anticipated to be less than 25 mm for the west approach and less than 10 mm for the east approach.

The above settlements are expected to occur during construction such that post-construction settlements are not anticipated at this site.

6.7.3.3 Embankment Widening

In accordance with the requirements of MTO NRE 98-200, the minimum required embankment widening at this site to account for the potential for future pavement overlays is 1.0 m per embankment side.

6.8 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all topsoil/vegetation/organic soils must be removed below the footprint of the proposed embankments. In general, the existing fill may be left in place below the pile cap level. All softened/loosened fill or native material should be stripped from below the new approach embankments, prior to placement of new fill.

Granular fill material specifications should meet those for Granular 'B' Type I or II. Compaction and placement should be carried out in accordance with the requirements as outlined in Special Provision SP206S03. Granular fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Side slopes for granular fill embankments should be no steeper than 2H:1V. The final lift of fill prior to placement of the roadway granular subbase and base courses should be compacted to 100 percent 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 embankment slopes and the newly placed embankment fill, the new fill should be keyed into the existing embankment side slope per the requirements of OPSD 208.010.

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with SP511S01. 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 potential for scour below the pile caps should 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 after construction where granular fill is used. 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.



6.9 Design and Construction Considerations

6.9.1 Excavations

Excavations for construction of the pile caps and the abutments of the main bridge will extend to Elevation 363.3 m and 364.1 m at the west and east abutments, respectively. The base of the excavation will be 5.0 m below the existing ground surface and 1.0 m below the April 2008 water level at the west abutment. At the east abutment, the base of the excavation will be 5.0 m below the ground surface and about 0.2 m below the April 2008 water level.

For the detour pier and abutments, the pile caps will be built above the surface of the water and above the existing ground surface and, therefore, excavation is not required.

In general, the excavations can be carried out in open cut except where temporary roadway protection will be required between the existing abutment and detour abutment on the west side of the river. Open cut slopes within the fill materials should be maintained at no steeper than 1.5H:1V above the water level through the material and 2H:1V below the water level. Depending on the proximity to the river and the Contractor's construction techniques, temporary shoring may be required.

Conventional excavation equipment should be suitable for excavation through the on-site soils; however, the Contractor shall be made aware of the potential for obstructions as discussed in Section 6.9.4.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The existing fill materials and the native soils should be classified as Type 3 soil, according to the OHSA.

6.9.2 Temporary Shoring

Temporary roadway protection is required between the detour west abutment and the main bridge west approach embankment to allow for construction of the main bridge west abutment.

Given the depth of excavation required to construct the main bridge abutments and the proximity to the river, a temporary cut-off wall (cofferdam) may be required. Although the east abutment is at about the water level, a cofferdam may still be required, depending on the time of year and the dewatering requirements.

Temporary excavation support systems should be designed and constructed in accordance with Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2.

6.9.3 Groundwater and Surface Water Control

The main bridge abutments are located adjacent to Oxtongue River and groundwater inflow should be expected during excavation for the pile caps and controlled dewatering within the temporary shoring will be required. The shoring should be advanced to an appropriate depth to control groundwater inflow. It should be anticipated that the excavations will have to be advanced using shoring, in conjunction with controlled dewatering or with fluid support, in order to minimize ground loss during excavation, backfilling and concrete placement. The Contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction.

The Contractor should be alerted that excavations will be advanced through cohesionless soils. The cohesionless soils are expected to be unstable below the groundwater level at this site. This Contractor should be alerted to this in an Operational Constraint (OC), an example of which is included in Appendix C.

Surface water should be directed away from the excavation at all times.



6.9.4 Obstructions

As part of the design and construction of the new foundations, careful consideration should be given to the location of the existing and/or older abandoned bridge foundations (for example, timber piles or cribs) relative to the new construction. Specifically, the designer should check that new piles (batter and orientation) and temporary shoring do not interfere with the existing and/or older abandoned piles. This should be checked to the full extent of the pile/shoring length.

Although the existing bridge, including pile caps, will be removed, the existing piles will likely be left in place. As is current practice, the existing timber piles should be cut off at the river bed level after installation of the new piles to enhance future use of waterway. It is not necessary to fully extract the timber piles.

A layer of cobbles and boulders was encountered within the sand to sand and gravel overlying the bedrock at the foundation elements. During the field investigation, rip-rap and larger rock fill fragments were also observed on the side slopes of the existing embankment and will require removal during construction of the bridge. The Contractor should be alerted to the presence of cobbles/boulders above the bedrock surface as they may interfere with pile driving as well as the presence of rip-rap and larger rock fragments on the slope during surface excavations. This can be accomplished in an OC, an example of which is included in Appendix C.

6.9.5 Existing Structure Monitoring

Given the age of the existing structure and the close proximity of the existing structure to the detour west abutment and central pier and the requirement for the existing structure to remain in operation during construction of the detour, it is recommended that the east and west abutments of the existing structure be monitored for excessive vibrations during pile driving (during seating of the pile on bedrock) for the detour bridge and/or other construction techniques that may cause vibration. An example NSSP is included in Appendix C for reference.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., a geotechnical engineer with Golder and the technical aspects were reviewed by Mrs. Sarah Coyne, P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Contact, conducted a quality control review of the report.



Signature Page

GOLDER ASSOCIATES LTD.

André Bom, P.Eng.
Geotechnical Engineer

Sarah E. M. Coyne, P.Eng.
Associate, Senior Geotechnical Engineer



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



AB/SEMC/FJH/lb

n:\active\2007\1190 sudbury\1191\07-1191-0001 urs oxtongue river\7000 reporting\oxtongue river\final oxtongue report\07-1191-0001-ox rpt 09nov30 oxtongue river and detour.docx



REFERENCES

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- Broms, B.B., 1964a. Lateral Resistance of Piles in Cohesive Soils; Journal for Soil Mechanics and Foundation Engineering., ASCE, Vol. 90, SM2, pp. 27-64.
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FOUNDATION REPORT - OXTONGUE RIVER BRIDGE REPLACEMENT AND DETOUR - HIGHWAY 60, GWP 5550-04-00

Table 1: Evaluation of Foundation Alternatives – Main Bridge

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock	1	<ul style="list-style-type: none"> ■ Straightforward construction. 	<ul style="list-style-type: none"> ■ Possibility of piles “hanging up” on cobbles and boulders. ■ Cofferdam construction required for pile cap construction depending on water levels at time of construction. 	<ul style="list-style-type: none"> ■ Lower relative costs compared with caisson option. 	<ul style="list-style-type: none"> ■ Dewatering within a cofferdam may be required in order to construct the pile cap.
Caissons Socketted into Bedrock	2	<ul style="list-style-type: none"> ■ Reduced number of deep elements compared to steel H-piles. ■ Possible elimination of pile cap. 	<ul style="list-style-type: none"> ■ Temporary liners would be required for groundwater control and support through overburden. ■ Concrete for caissons would have to be placed by tremie methods below the water level. ■ Difficulty socketting caissons into gneiss bedrock. ■ Cofferdam construction required for caisson cap construction adjacent to river. 	<ul style="list-style-type: none"> ■ Cost many times higher than for piles. 	<ul style="list-style-type: none"> ■ Risk of difficulties in penetrating the cobble and boulder layer and achieving seal and drilling large diameter bedrock socket.



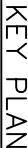
FOUNDATION REPORT - OXTONGUE RIVER BRIDGE REPLACEMENT AND DETOUR - HIGHWAY 60, GWP 5550-04-00

Table 2: Evaluation of Foundation Alternatives – Detour Bridge

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock	1 (abutments and central pier)	<ul style="list-style-type: none"> ■ Straightforward construction. 	<ul style="list-style-type: none"> ■ Possibility of piles “hanging up” on cobbles and boulders. 	<ul style="list-style-type: none"> ■ Lower relative costs compared with caisson option. 	
Spread Footings Founded on Compacted Granular Pad over Native materials	2 (abutments only)	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Construction and excavation above the water level. 	<ul style="list-style-type: none"> ■ Low axial resistance. ■ Immediate settlement of cohesionless soils will occur; maintenance may be required during detour lifespan. 	<ul style="list-style-type: none"> ■ Lower compared to deep foundation alternatives. 	<ul style="list-style-type: none"> ■ Risk of settlement during construction, though detour bridge is not sensitive to expected settlements. ■ Risk of needing shoring/cofferdam near the river.
Caissons Socketted into Bedrock	3	<ul style="list-style-type: none"> ■ Reduced number of deep elements compared to steel H-piles. ■ Possible elimination of pile cap. 	<ul style="list-style-type: none"> ■ Temporary liners and cofferdams would be required for groundwater control and support through overburden. ■ Concrete for caissons would have to be placed by tremie methods below the water level. Difficult installation from water at pier. ■ Difficulty socketting caissons into bedrock. 	<ul style="list-style-type: none"> ■ Cost many times higher than for piles. 	<ul style="list-style-type: none"> ■ Risk of difficulties in penetrating the cobble and boulder layer and achieving seal and drilling large diameter bedrock socket.



SHEET



Borehole

- | | |
|------|---|
| N | Standard Penetration Test Value |
| 4 | Blows/0.3 m unless otherwise stated
(Std. Pen. Test, 475/j/blow) |
| 100% | Rock Quality Designation (RQD) |
| ∇ | WL upon completion of drilling |

CO-ORDINATES

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.

The complete foundation investigation and design report for this project, including all test logs, test results, 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 OC 2.01 of OPS General Conditions.

Base plan provided in digital format by URS Canada Inc., drawing file no. 01_Oxt_River_GA.dwg (dated Feb., 2009) and key plan, drawing file Keyplan GMP 5550-04-00.dwg (received Nov., 2009).






WEST ABUT.

PROFILE A-A'

The figure shows two scales used for the model. The horizontal scale is marked from 0 to 10 m, with major ticks every 5 m. The vertical scale is marked from 0 to 5 m, with major ticks every 2.5 m. Both scales are represented by a black bar with white rectangular segments.

METRIC

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

CONT No.
WP No.5550-04-00

HIGHWAY 60
OXTONGUE RIVER BRIDGE
SOIL STRATA



Goldier Associates Ltd.
SUBSBURY, ONTARIO, CANADA

SHEET

LEGEND

- Borehole
- Standard Penetration Test Value
- Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- Rock Quality Designation (RQD)
- WL upon completion of drilling

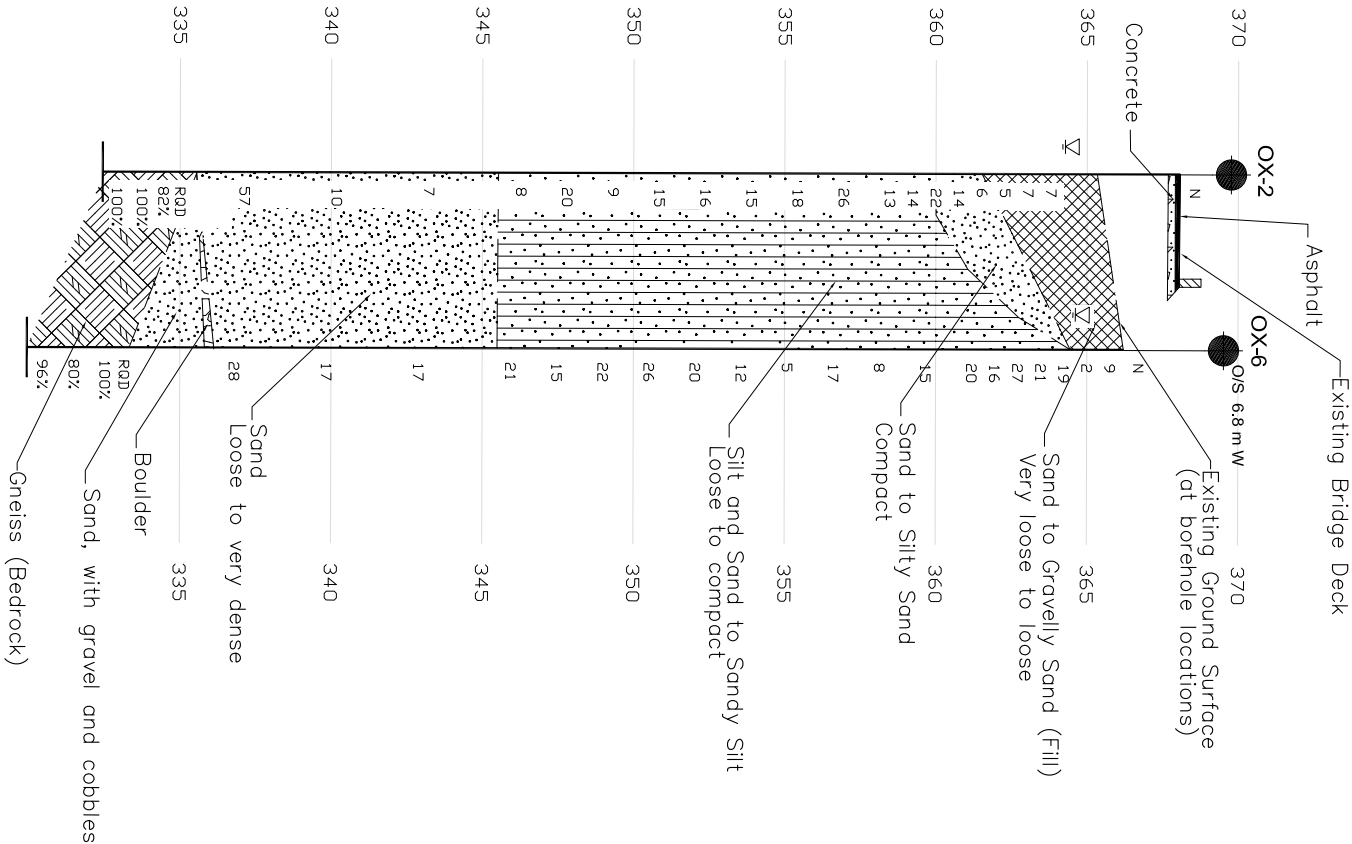
No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
OX-1	367.8	5027330.2	351126.5
OX-2	368.1	5027338.7	351133.2
OX-3	369.1	5027355.8	351163.7
OX-4	369.6	5027366.9	351183.9
OX-5	367.1	5027316.4	351120.1
OX-6	366.2	5027326.1	351137.5
OX-7	364.3	5027345.3	351172.6
OX-8	369.7	5027366.3	351205.5
OX-9	370.4	5027377.0	351223.8

NOTES

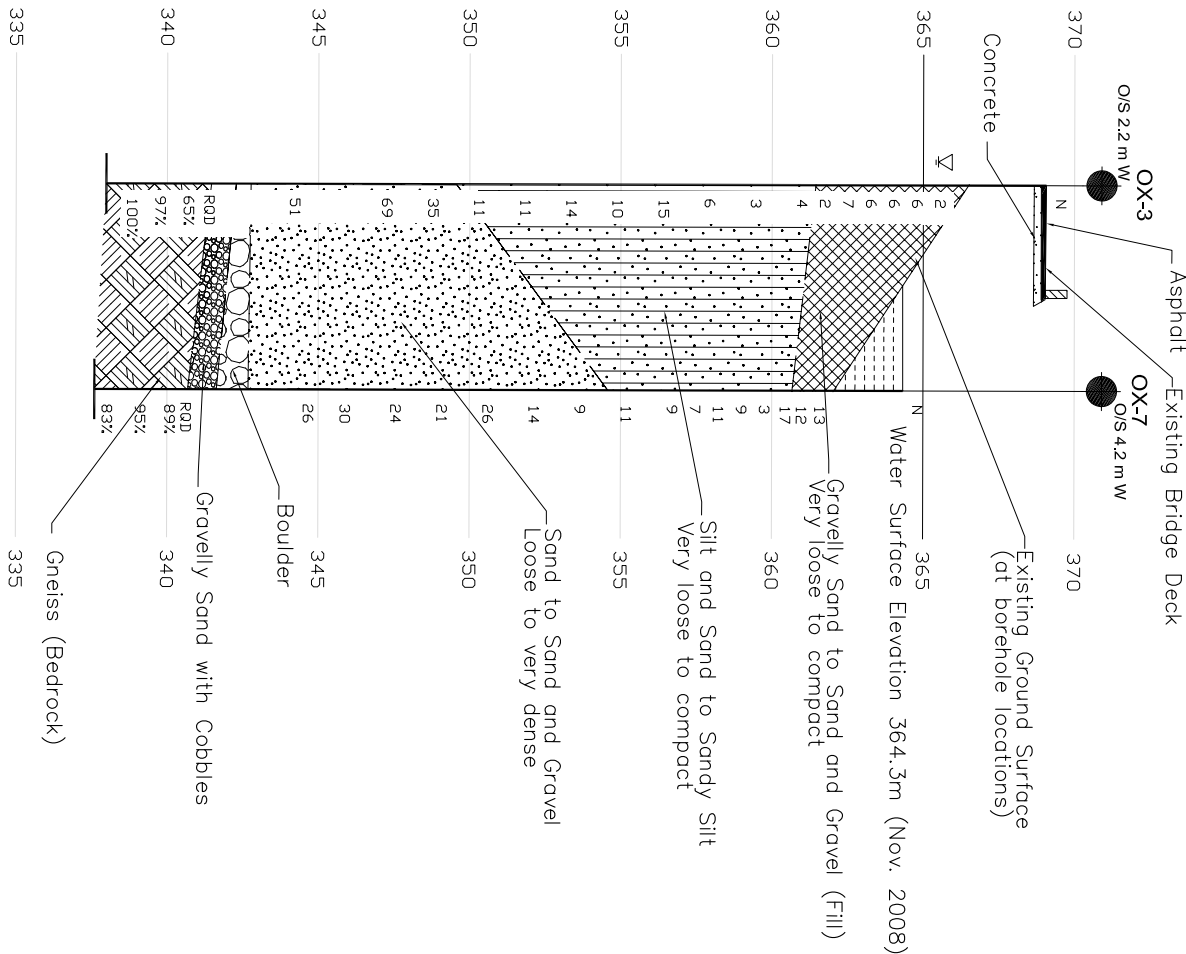
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

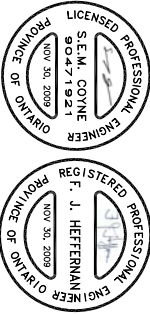
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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



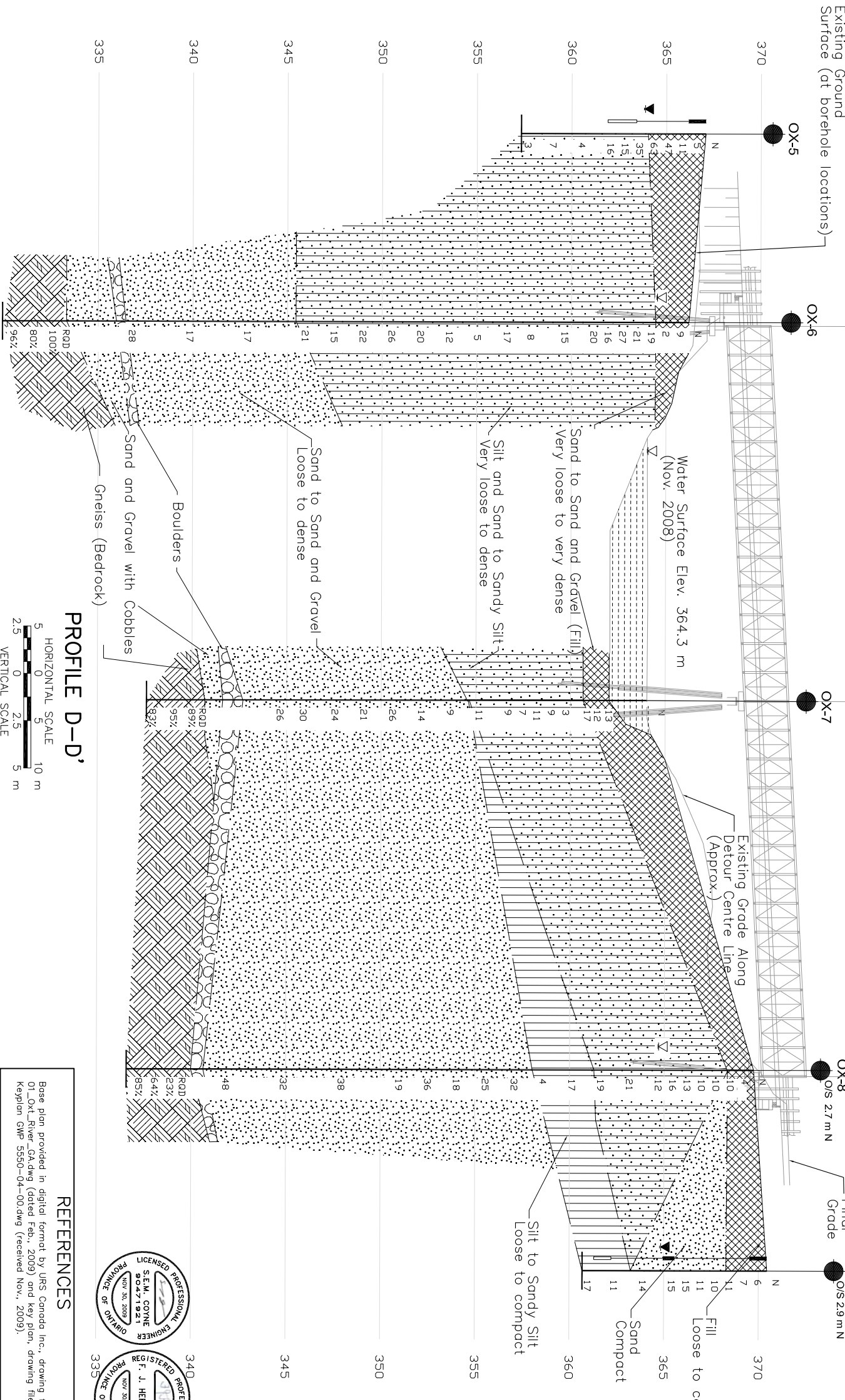
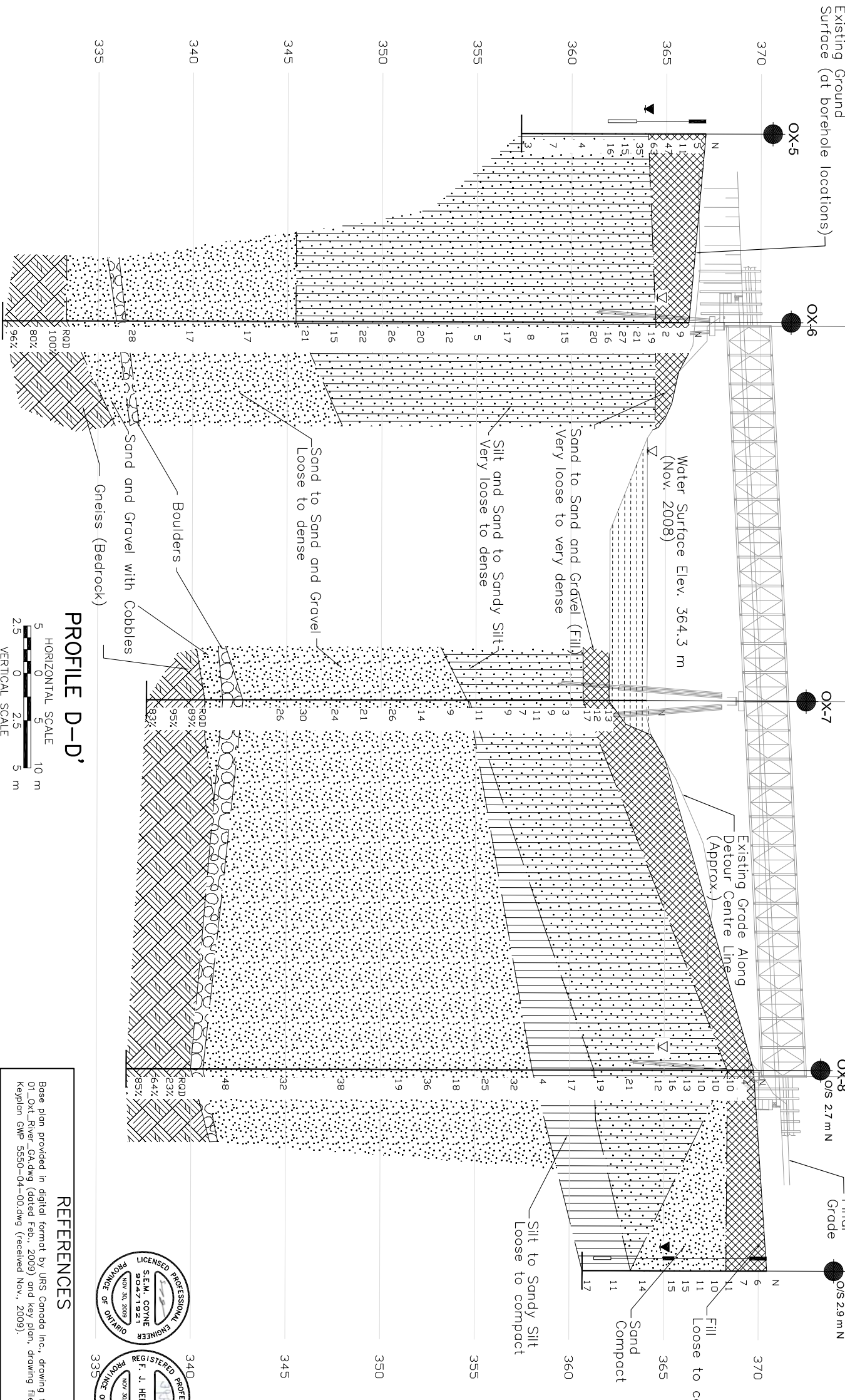
WEST ABUTMENT SECTION B-B'



EAST ABUTMENT SECTION C-C'

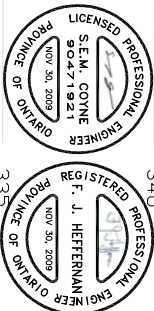







NO.	DATE	BY	REVISION
HWY. 60			
SUBMTD. TRR	CHRD. AB	DATE: Nov 2009	SITE: 40-002
DRAWN: MM	CHRD. SEMC	APPD: FJH	DWG. 2



The figure shows two scales. The horizontal scale is at the top, with markings at 0, 2.5, 5, and 10 m. The vertical scale is on the left, with markings at 0, 2.5, and 5 m. Both scales have alternating black and white segments.

Base plan provided in digital format by URS Canada Inc., drawing file no. 01_Oxt_River_GA.dwg (dated Feb., 2009) and key plan, drawing file Keyplan GWP 5550-04-00.dwg (received Nov., 2009).



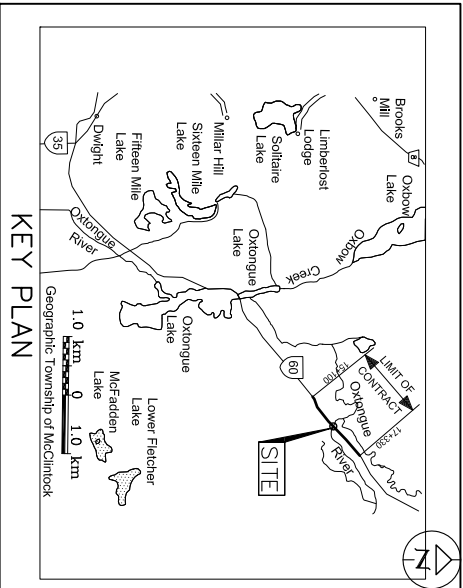
LEGEND			
	Borehole		
N	Standard Penetration Test Value		
4	Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475/blow)		
	Seal		
	Piezometer		
100%	Rock Quality Designation (RQD)		
	WL in piezometer, measured on April 28, 2009		
	WL upon completion of drilling		
No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
OX-1	367.8	5027338.2	351126.5
OX-2	368.1	5027330.7	351133.2
OX-3	369.1	5027355.8	351163.7
OX-4	369.6	5027366.9	351183.9
OX-5	367.1	5027316.4	351120.1
OX-6	366.2	5027326.1	351137.5
OX-7	364.3	5027345.3	351172.6
OX-8	369.7	5027366.3	351205.5
OX-9	370.4	5027377.0	351223.8

NOTES

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

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 OHS General Conditions.

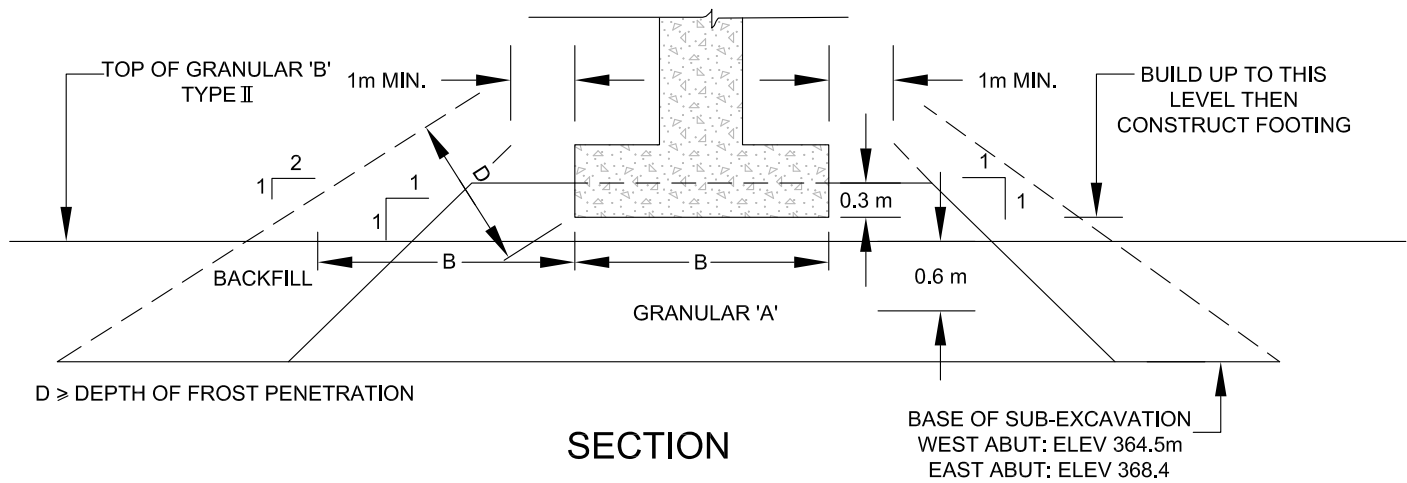
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Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

CONT No.
WP No. 5550-04-00

HWY 60 OXTONGUE RIVER BRIDGE
DETOUR STRUCTURE
BOREHOLE LOCATION AND
SOIL STRATA





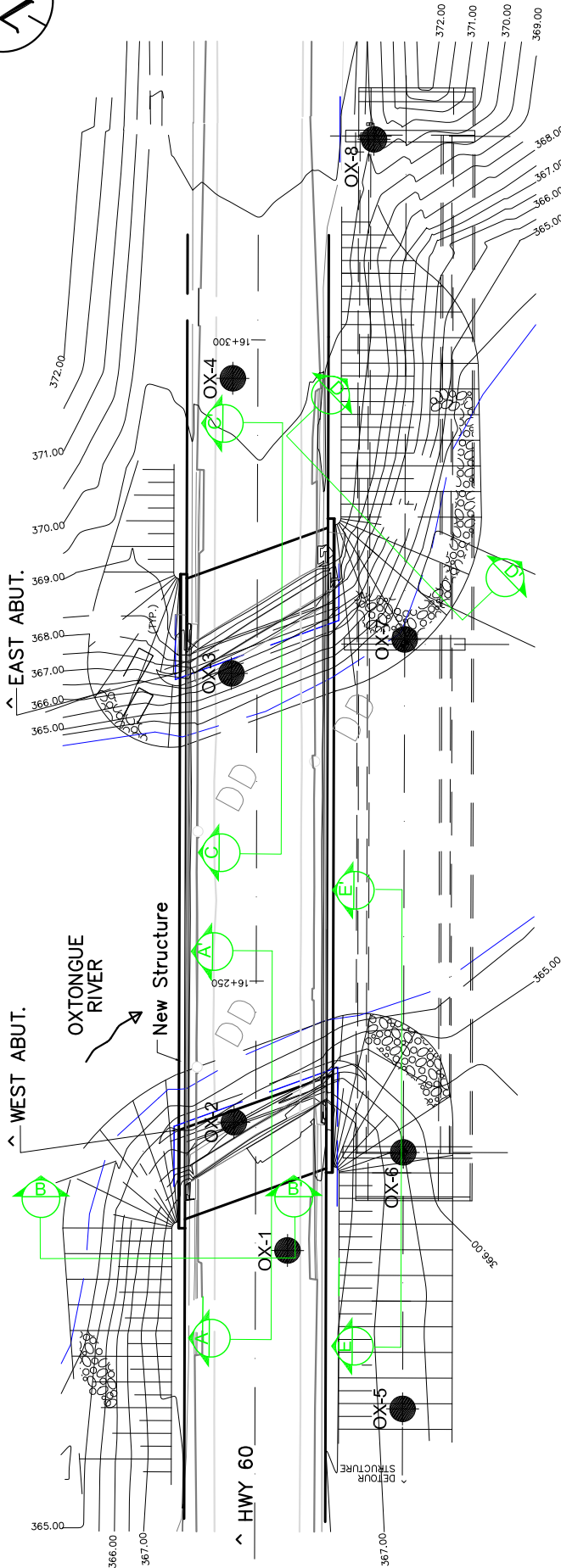
CONSTRUCTION SEQUENCE:

1. REFER TO ACCOMPANYING FOUNDATION DESIGN REPORT, SECTION 6.2.
2. REMOVE SUBSOILS UNDER FOOTPRINT OF COMPACTED GRANULAR CORE TO ELEVATION SPECIFIED.
3. PLACE AND COMPACT GRANULAR 'A' IN ACCORDANCE WITH SP105 510 TO UNDER SIDE OF FOOTING LEVEL.
4. CONSTRUCT CONCRETE FOOTING.
5. PLACE REMAINDER OF GRANULAR 'A' AND BACKFILL AS REQUIRED.
6. SOURCE M.T.C 1982.

NOT TO SCALE

PROJECT		GWP 5550-04-00 OXTONGUE RIVER DETOUR BRIDGE	
TITLE		TYPICAL DETOUR ABUTMENT ON COMPACTED FILL CORE	
PROJECT No. 07-1191-0001		FILE No. 07-1191-0001FIG1.dwg	
DESIGN		SCALE	NTS REV.
CAD	PL	NOV 2009	FIGURE No. 1
CHECK	SEMC	NOV 2009	
REVIEW	FJH	NOV 2009	





- NOTES**
1. Sketch shown for illustration purposes. Section lines are approximate only.
 2. Refer to Section 6.7. of accompanying Foundation Design Report.

REFERENCES

Base plan provided in digital format by URS Canada Inc., drawing file no. 01_Oxt_River_GA.dwg (dated Feb., 2009) and key plan, drawing file oxtongue key plan.pdf (received Nov., 2008).

LEGEND



TITLE

PROJECT No.07-1191-0001		
FILE No.0711910001FIG1.DWG		
REV. 0	SCALE AS SHOWN	
DESIGN		
CADD	PL	NOV 2009
CHECK	SEMC	NOV 2009
REVIEW	FJH	NOV 2009

FIGURE 2

CRITICAL STABILITY SECTIONS

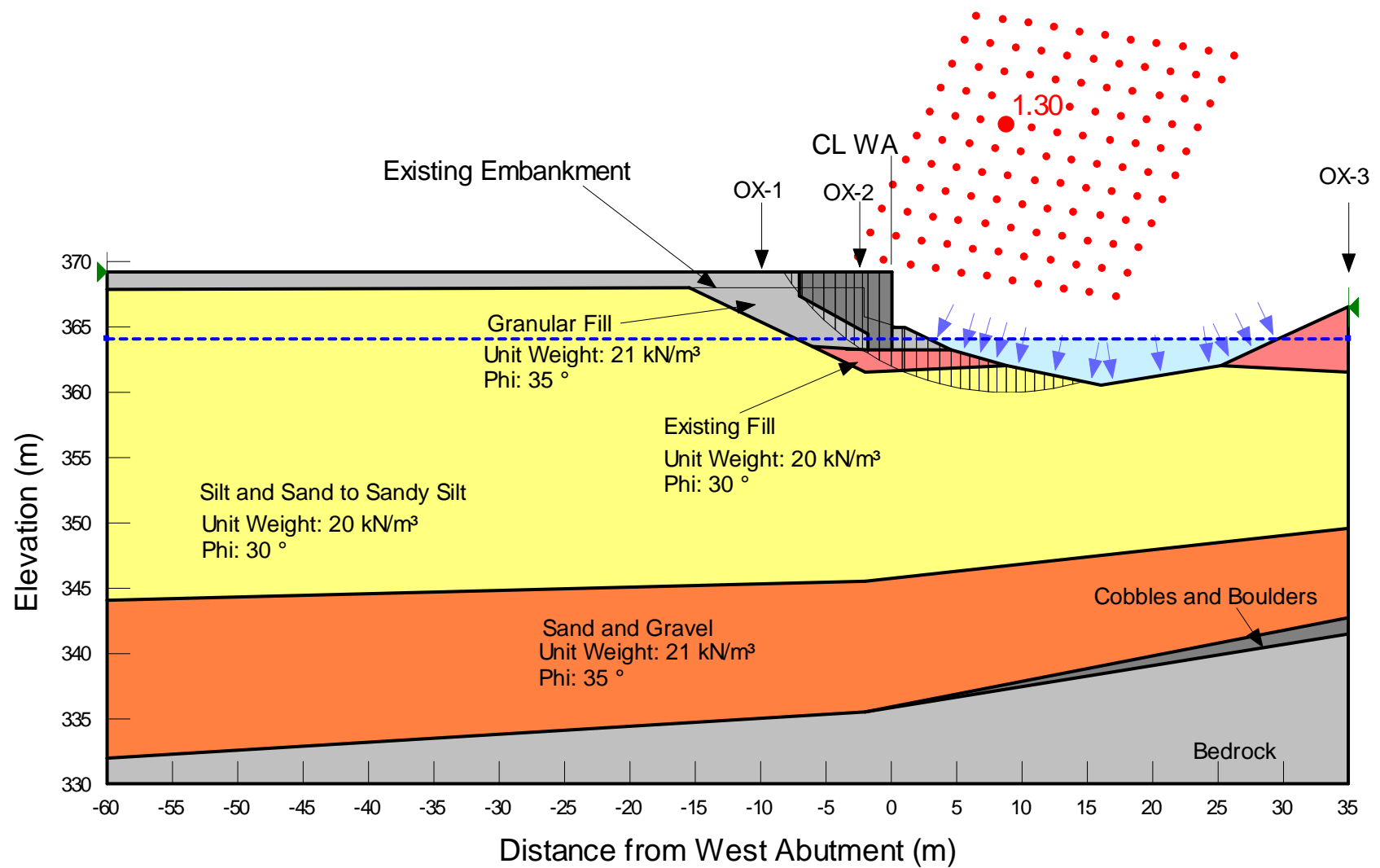
PROJECT

OXTONGUE RIVER BRIDGE
GWP 5550-04-00



STABILITY ANALYSIS West Approach Front Slope

FIGURE 3



DATE: November 2009

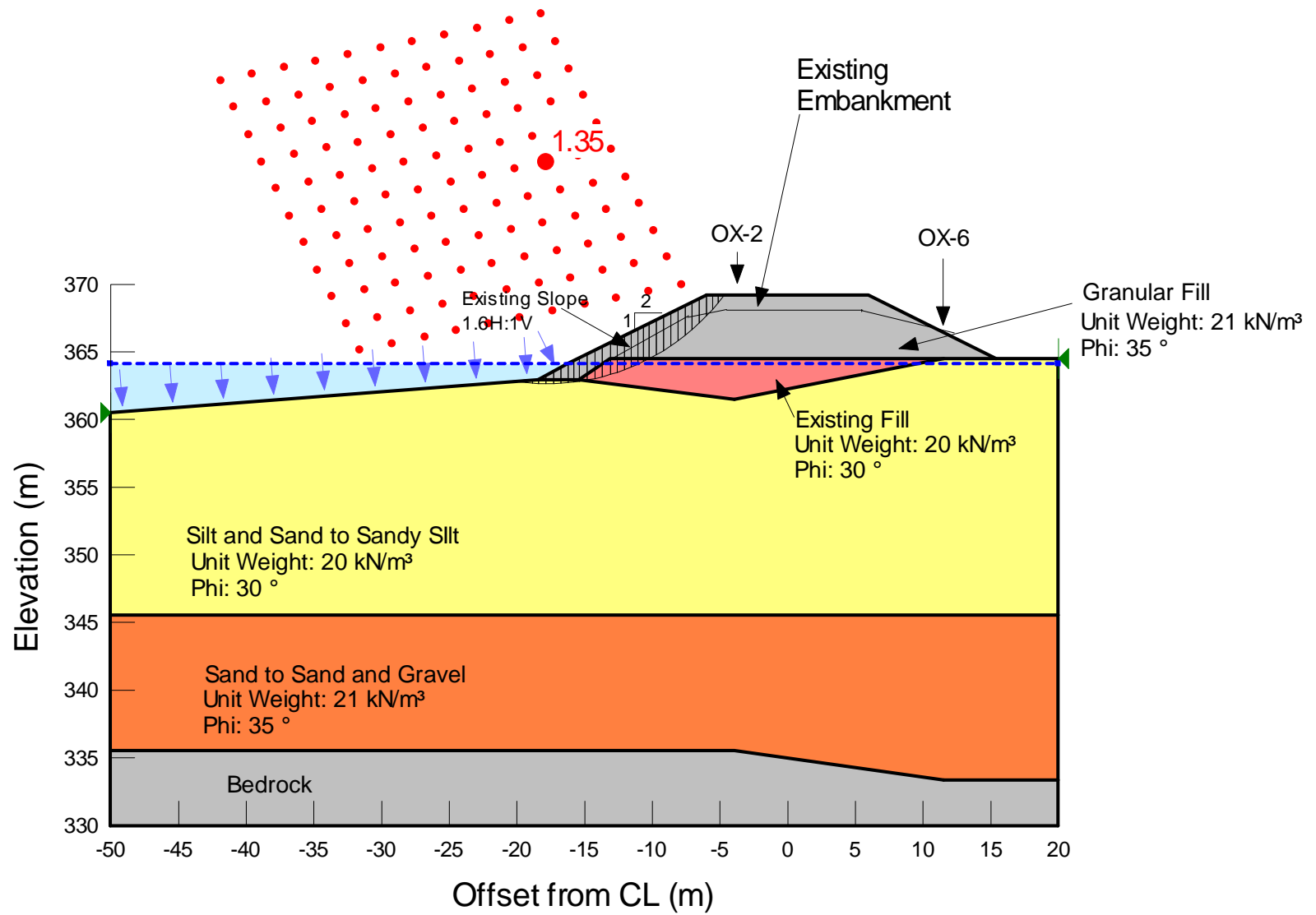
PROJECT: 07-1191-0001-OX



Drawn by: AB Checked by: SEMC

STABILITY ANALYSIS West Approach Northwest Slope

FIGURE 4



DATE: November 2009

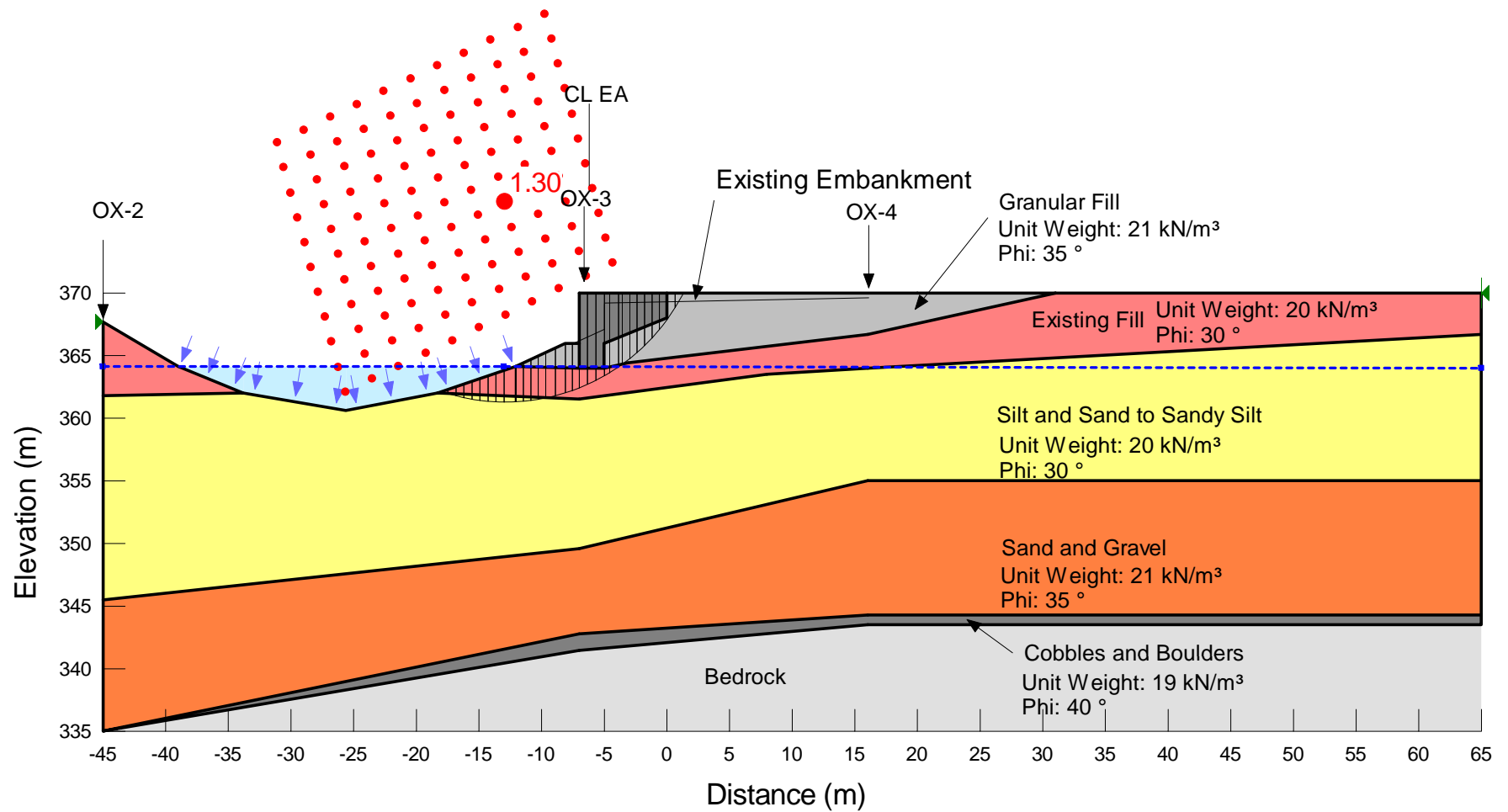
PROJECT: 07-1191-0001-OX



Drawn by: AB Checked by: SEMC

STABILITY ANALYSIS East Approach Front Slope

FIGURE 5



DATE: November 2009

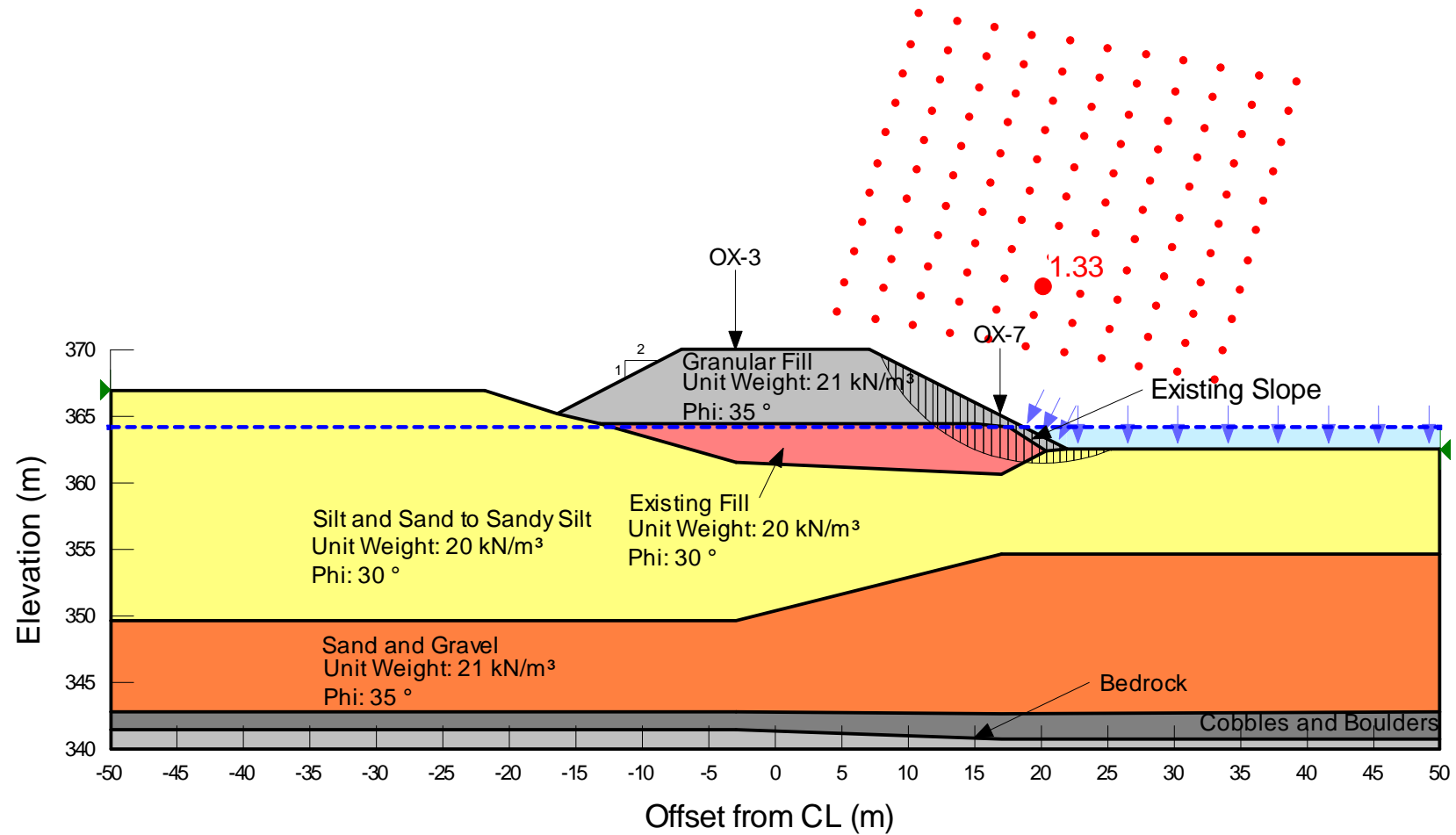
PROJECT: 07-1191-0001-OX



Drawn by: AB Checked by: SEMC

STABILITY ANALYSIS
East Approach Southeast Slope

FIGURE 6



DATE: November 2009

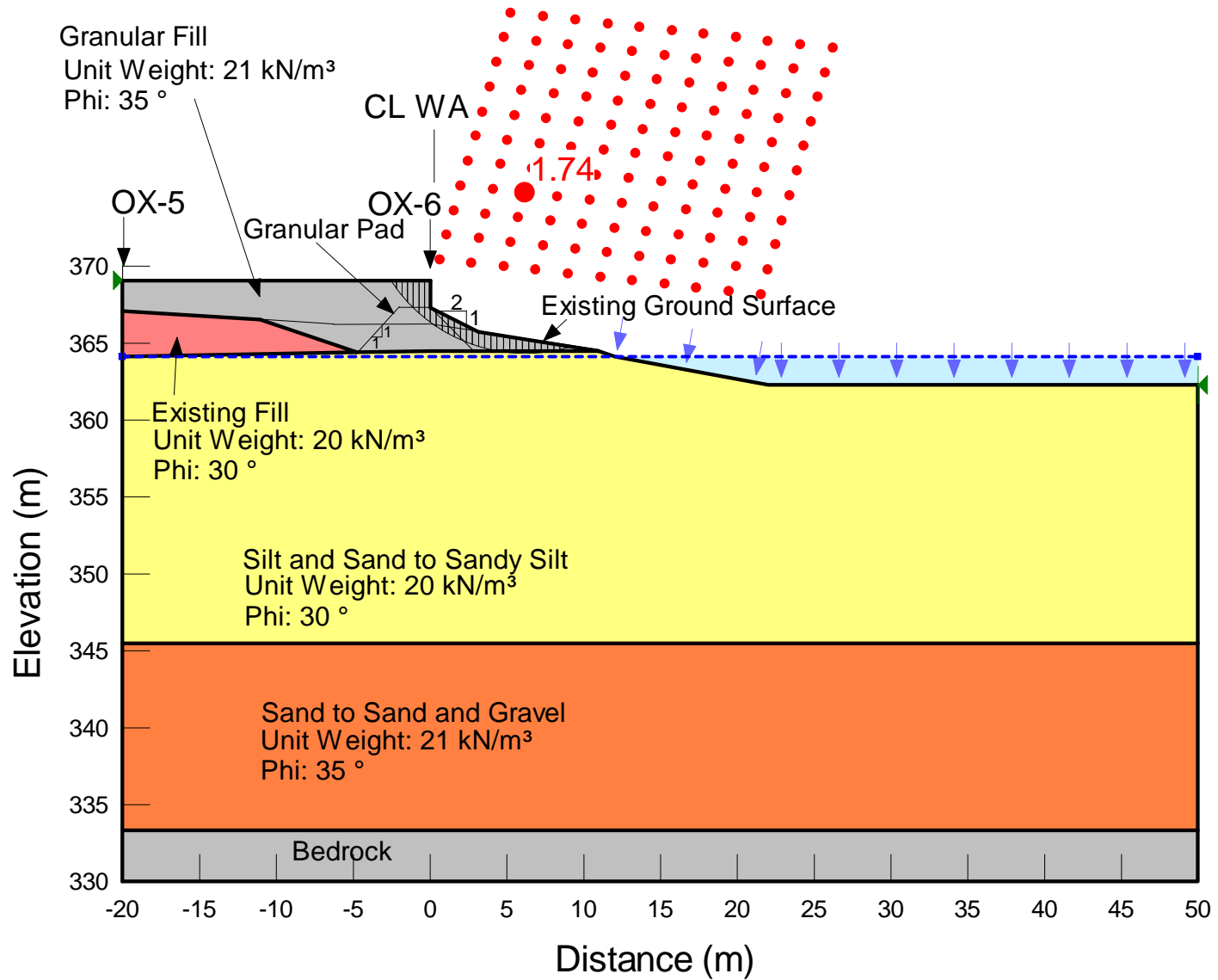
PROJECT: 07-1191-0001-OX



Drawn by: AB Checked by: SEMC

STABILITY ANALYSIS Detour West Approach Front Slope

FIGURE 7



DATE: November 2009

PROJECT: 07-1191-0001-OX



Drawn by: AB Checked by: SEMC



APPENDIX A

RECORD OF BOREHOLES AND DRILLHOLES

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. 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 $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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

Piezcone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

w	water content
w _p	plastic 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.

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of Major discontinuities

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

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

PROJECT <u>07-1191-0001-OX</u>			RECORD OF BOREHOLE No OX-1			1 OF 1 METRIC											
W.P. <u>5550-04-00</u>			LOCATION <u>N 5027330.2 ; E 351126.5</u>			ORIGINATED BY <u>TDM</u>											
DIST <u> </u> HWY <u>60</u>			BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>			DATE <u>January 12, 2009</u>			CHECKED BY <u>SEMC</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
367.8	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.2	Gravelly sand, some silt (FILL) Loose Brown Moist		1	AS	-												23 62 (15)
			2	AS	-												
			3	SS	9												
365.5																	
2.3	Silty SAND, some silt, trace gravel Compact Light brown Moist		4	SS	23												
			5	SS	28												1 76 (23)
			6	SS	20												
			7	SS	19												
361.7																	
6.1	SILT and SAND Compact Grey Wet		8	SS	15												0 37 (63)
			9	SS	20												
358.1																	
9.7	End of Borehole		10	SS	27												
	Note: 1. Water level in open borehole at a depth of 4.4 m below ground surface (Elev. 363.4 m) upon completion of drilling.																

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MIS-MTO 001 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

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


2 OF 3 METRIC

CHECKED BY SEMC

Continued Next Page

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

MIS-MTO 001 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

PROJECT <u>07-1191-0001-OX</u>			RECORD OF BOREHOLE No OX-2				3 OF 3 METRIC				
W.P. <u>5550-04-00</u>		LOCATION <u>N 5027338.7 ;E 351133.2</u>				ORIGINATED BY <u>EHS</u>					
DIST <u> </u> HWY <u>60</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>					
DATUM <u>Geodetic</u>		DATE <u>November 20 and 21, December 22 and 23, 2008</u>				CHECKED BY <u>SEMC</u>					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND, trace gravel, trace silt, with gravel layers Loose to very dense Brown Wet		19	SS	57		338				
							337				
							336				
335.6	GNEISS (BEDROCK)										
32.5	Bedrock cored from 32.5 m to 35.6 m depth. For coring details, refer to Record of Drillhole OX-2.		1	RC	REC 92%		335				RQD = 79%
			2	RC	REC 100%		334				RQD = 100%
			3	RC	REC 95%		333				RQD = 95%
332.5	End of Borehole										
35.6	Notes: 1. Water level in open borehole at a depth of 3.8 m below ground surface (Elev. 364.3 m) upon completion of drilling. 2. No recovery in samples 2, 3 and 18 on the first attempt. 3. Borehole stopped on Nov. 21, 2008 due to weather at 32.5 m depth. Casing left in the hole. Borehole (rock coring) resumed on Dec. 22, 2008.										

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

CHECKED: SEMC

MIS-RCK 004 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

RECORD OF BOREHOLE No OX-3

1 OF 3 **METRIC**

PROJECT 07-1191-0001-OX

W.P. 5550-04-00

LOCATION N 5027355.8 ; E 351163.7

ORIGINATED BY EHS

DIST HWY 60

BOREHOLE TYPE NW Casing, Wash Boring

COMPILED BY MM

DATUM Geodetic


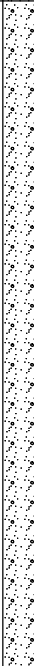



DATE November 18 and 19, 2008

CHECKED BY SEMC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
369.1	GROUND SURFACE													
0.0	ASPHALT (90mm)													
0.3	CONCRETE (200mm)													
	VOID (Beneath Bridge)													
366.5														
2.6	Gravelly sand, trace silt and occasional cobbles (FILL) Very loose to loose Brown Moist to wet Concrete and asphalt rubble above 3.7 m depth.		1	SS	2		366							
			2	SS	6		365							
			3	SS	6		364							
			4	SS	6		363							34 62 (4)
			5	SS	7		362							
			6	SS	2		361							0 34 64 2
361.5			7	SS	4		360							
7.6	SILT and SAND to Sandy SILT, trace clay, layered Very loose to compact Light brown Wet		8	SS	3		359							
			9	SS	6		358							0 56 44 0
			10	SS	15		357							
			11	SS	10		356							
							355							0 44 (56)

Continued Next Page

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

PROJECT <u>07-1191-0001-OX</u>				RECORD OF BOREHOLE No OX-3				2 OF 3 METRIC												
W.P. <u>5550-04-00</u>				LOCATION <u>N 5027355.8 ;E 351163.7</u>				ORIGINATED BY <u>EHS</u>												
DIST <u> </u> HWY <u>60</u>				BOREHOLE TYPE <u>NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>												
DATUM <u>Geodetic</u>				DATE <u>November 18 and 19, 2008</u>				CHECKED BY <u>SEMC</u>												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										20 40 60 80 100 10 20 30		
--- CONTINUED FROM PREVIOUS PAGE ---																				
349.6 19.5	SILT and SAND to Sandy SILT, trace clay, layered Very loose to compact Light brown Wet		12	SS	14		354										0 25 75 0			
								353												
								352												
								351												
								350												
								349												
								348												
								347												
								346												
								345												
342.8 26.3 342.3 26.8 341.5 27.6	SAND and GRAVEL, trace silt, occasional cobbles, layered Dense to very dense Light brown Wet Difficult to advance casing below 19.8 m depth Oxidized layer 12 mm thick at 20.1 m depth.		15	SS	35		349										57 34 (7)			
								348												
								347												
								346												
								345												
								344												
								343												
								342												
								341												
								340												
342.8 26.3	BOULDERS																			
342.3 26.8	COBBLES in a sand and gravel matrix																			
341.5 27.6	GNEISS (BEDROCK) Bedrock cored from 27.6 m to 31.1 m depth. For coring details, refer to Record of Drillhole OX-3.		1	RC	REC 100%		341									RQD = 65%				
			2	RC	REC 100%		340									RQD = 97%				

Continued Next Page

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PROJECT <u>07-1191-0001-OX</u>				RECORD OF BOREHOLE No OX-3				3 OF 3 METRIC									
W.P. <u>5550-04-00</u>		LOCATION <u>N 5027355.8 ;E 351163.7</u>				ORIGINATED BY <u>EHS</u>											
DIST <u> </u> HWY <u>60</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>		DATE <u>November 18 and 19, 2008</u>				CHECKED BY <u>SEMC</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
								20	40	60	80	100	10	20	30		
338.0			3	RC	REC 100%		339										
31.1	End of Borehole Note: 1. Water level in open borehole at a depth of 3.6 m below ground surface (Elev. 365.5 m) upon completion of drilling. 2. No recovery in samples 2 to 4 and 6 on first attempt.						338										

PROJECT: 07-1191-0001-OX

RECORD OF DRILLHOLE: OX-3

SHEET 1 OF 1

LOCATION: N 5027355.8 ;E 351163.7

DRILLING DATE: November 18 and 19, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50 Turbo Bomb

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE min/m	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
				DEPTH											RECOVERY	R.Q.D.	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
				(m)														TOTAL CORE %	SOLID CORE %	%	B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr			Ja	Jn	K, cm/sec	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
																		80 60 40 20	80 60 40 20	80 60 40 20														0 30 60 90 120 150 180 210 240 270 300 330 360	0 30 60 90 120 150 180 210 240 270 300 330 360	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								

PROJECT <u>07-1191-0001-OX</u>			RECORD OF BOREHOLE No OX-4			1 OF 1 METRIC											
W.P. <u>5550-04-00</u>			LOCATION <u>N 5027366.9 ; E 351183.9</u>			ORIGINATED BY <u>EHS</u>											
DIST <u> </u> HWY <u>60</u>			BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>			DATE <u>November 16 and 17, 2008</u>			CHECKED BY <u>SEMC</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
369.6	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (120 mm)																
369.2	Granular A (140 mm), Granular B (30 mm) and Chip and Tar (30 mm) (FILL)																
0.4	Gravelly sand, trace silt, occasional cobble, layered, oxidized (FILL) Loose to compact Brown Moist		1	SS	21												
			2	SS	7												
			3	SS	9												
366.7																	
2.9	SAND, trace to some gravel, trace silt, layered, oxidized Very loose to loose Brown Moist		4	SS	5												
			5	SS	5												
			6	SS	4												
364.0																	
5.6	SILT and SAND to Sandy SILT, trace clay, trace organics, occasional sand seam, layered Compact Light brown Wet		7	SS	14												
	A 1.5 m thick, light brown, wet, silt, trace sand layered at 7.2 m depth.		8	SS	17												
359.9			9	SS	2												
9.7	End of Borehole																
	Note: 1. Water level in open borehole at a depth of 5.6 m below ground surface (Elev. 364.0 m) upon completion of drilling.																

MIS-MTO 001 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

[illegible]

RECORD OF BOREHOLE No OX-6

1 OF 3 **METRIC**

PROJECT 07-1191-0001-OX

W.P. 5550-04-00

LOCATION N 5027326.1 :E 351137.5

ORIGINATED BY EHS

DIST HWY 60

BOREHOLE TYPE 108 mm Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY MM

DATUM Geodetic

DATE November 10 to 12, 2008

CHECKED BY SEMC

[illegible]

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

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

MIS-MTO 001 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

PROJECT <u>07-1191-0001-OX</u>				RECORD OF BOREHOLE No OX-6				2 OF 3 METRIC										
W.P. <u>5550-04-00</u>		LOCATION <u>N 5027326.1 ; E 351137.5</u>				ORIGINATED BY <u>EHS</u>												
DIST <u> </u> HWY <u>60</u>		BOREHOLE TYPE <u>108 mm Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>												
DATUM <u>Geodetic</u>		DATE <u>November 10 to 12, 2008</u>				CHECKED BY <u>SEMC</u>												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								20	40	60	80	100						
--- CONTINUED FROM PREVIOUS PAGE ---							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)						
							20	40	60	80	100	10	20	30				
	SILT and SAND to Sandy SILT, trace to some gravel, trace silt, trace clay, oxidized, layered Loose to compact Light brown to grey Wet		14	SS	26		351										0 92 8 0	
							350											
			15	SS	22		349											
							348											
			16	SS	15		347										0 68 32 0	
							346											
345.5			17	SS	21		345											
20.7	SAND, some gravel, trace silt, layered, oxidized Compact Light brown Wet						344											
			18	SS	17		343										14 78 (8)	
							342											
							341											
			19	SS	17		340											
							339											
							338											
			20	SS	28		337											

Continued Next Page

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

PROJECT <u>07-1191-0001-OX</u>		RECORD OF BOREHOLE No OX-6				3 OF 3 METRIC										
W.P. <u>5550-04-00</u>		LOCATION <u>N 5027326.1 ; E 351137.5</u>				ORIGINATED BY <u>EHS</u>										
DIST <u> </u> HWY <u>60</u>		BOREHOLE TYPE <u>108 mm Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>										
DATUM <u>Geodetic</u>		DATE <u>November 10 to 12, 2008</u>				CHECKED BY <u>SEMC</u>										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			W _p	W	W _L	γ	GR SA SI CL	
336.1 335.8	BOULDER						336									
30.4	SAND, with gravel and cobbles, layered Light brown Wet						335									
333.3							334									
32.9	GNEISS (BEDROCK)						333									RQD = 100%
	Bedrock cored from 32.9 m to 36.3 m depth. For coring details, refer to Record of Drillhole OX-6.		1	RC	REC 100%		332									RQD = 80%
			2	RC	REC 100%		331									RQD = 94%
329.9			3	RC	REC 100%		330									
36.3	End of Borehole Note: 1. Water level in open borehole at a depth of 1.6 m below ground surface (Elev. 364.6 m) upon completion of drilling.															

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

[illegible]

LOGGED: EHS
CHECKED: SEMC

1 : 50

PROJECT 07-1191-0001-OX		RECORD OF BOREHOLE No OX-7		1 OF 2 METRIC	
W.P. 5550-04-00		LOCATION N 5027345.3 ; E 351172.6		ORIGINATED BY EHS	
DIST HWY 60		BOREHOLE TYPE NW Casing, Wash Boring		COMPILED BY MM	
DATUM Geodetic		DATE November 3 to 5, 2008		CHECKED BY SEMC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								20	40	60	80	100			W _p	W	W _L
								○ UNCONFINED			+	FIELD VANE					
						● QUICK TRIAXIAL			×	REMOULDED							
364.3	GROUND SURFACE																
0.0	WATER																
362.0																	
2.3	Sand and gravel, with silt, trace organics, occasional cobbles (FILL) Compact Brown Wet		1	SS	13							○					
			2	SS	12												
360.7																	
3.6	SILT and SAND, trace gravel, layered, oxidized Very loose to compact Brown to light brown Wet		3	SS	17							○			39 36 23 2		
			4	SS	3												
			5	SS	9								○		3 64 33 0		
			6	SS	11												
			7	SS	7												
			8	SS	9							○			6 55 39 0		
			9	SS	11												
354.6																	
9.7	SAND to SAND and GRAVEL, trace silt, oxidized, layered Loose to compact Brown Wet		10	SS	9												
			11	SS	14							○			3 91 6 0		
	Grinding of casing on dense layers throughout.		12	SS	26												

Continued Next Page

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

MIS-MTO 001 OXTONGUE LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

PROJECT <u>07-1191-0001-OX</u>		RECORD OF BOREHOLE No OX-7		2 OF 2 METRIC	
W.P. <u>5550-04-00</u>		LOCATION <u>N 5027345.3 ; E 351172.6</u>		ORIGINATED BY <u>EHS</u>	
DIST <u> </u> HWY <u>60</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 3 to 5, 2008</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					
								20	40	60	80	100	W _p	W	W _L			
--- CONTINUED FROM PREVIOUS PAGE ---																		
342.7 21.6 341.6 22.7 340.7 23.6	SAND to SAND and GRAVEL, trace silt, oxidized, layered Loose to compact Brown Wet		13	SS	21		349										36 61 (3)	
							348											
							347											
							346											
							345											
							344											
	BOULDER			RC			342											53 43 (4)
	Boulder cored from 21.6 m depth to 22.7 m depth.																	
	Gravelly SAND, with cobbles						341											
340.7 23.6	GNEISS (BEDROCK)		1	RC	REC 100%		340											RQD = 85%
			2	RC	REC 100%		339										RQD = 95%	
			3	RC	REC 100%		338										RQD = 85%	
337.6 26.7	End of Borehole																	
	Notes: 1. No recovery in samples 2, 3 and 14 on the first attempt. 2. Difficult casing advance below 21.6 m depth.																	

MIS-MTO-001 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

PROJECT: 07-1191-0001-OX

RECORD OF DRILLHOLE: OX-7

SHEET 1 OF 1

LOCATION: N 5027345.3 ;E 351172.6

DRILLING DATE: November 3 to 5, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D25 Barge

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE min/m	FLUSH % RETURN	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
				DEPTH (m)										RECOVERY			R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY			Diameter Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
														TOTAL CORE %	SOLID CORE %	%			B Angle	DIP W.R.T. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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DEPTH SCALE

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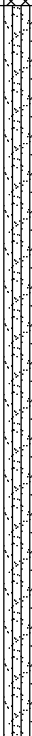

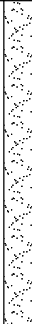


LOGGED: EHS

CHECKED: SEMC

MIS-RCK 004 OXTONGUE LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

PROJECT 07-1191-0001-OX		RECORD OF BOREHOLE No OX-8		1 OF 3 METRIC	
W.P. 5550-04-00		LOCATION N 5027366.3 ; E 351205.5		ORIGINATED BY EHS	
DIST HWY 60		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring		COMPILED BY MM	
DATUM Geodetic		DATE November 13, 2008		CHECKED BY SEMC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE					w _p w w _L							
								● QUICK TRIAXIAL × REMOULDED												
369.7	GROUND SURFACE						20	40	60	80	100									
0.0	Fine to medium sand, trace gravel, trace silt (FILL) Loose to compact Light brown Moist		1	SS	4									○						
			2	SS	10															
368.3																				
1.4	SILT and SAND to Silty SAND, trace gravel, layered Compact to dense Light brown Moist to wet		3	SS	10									○			2	70 (28)		
			4	SS	10															
			5	SS	13															
			6	SS	16									○			0	64 (36)		
			7	SS	12									○			1	72 27 0		
			8	SS	21															
			9	SS	19															
361.0																				
8.7	SILT, some sand, trace clay, containing sand layers Loose to compact Light brown Wet		10	SS	17									○			0	17 80 3		
			11	SS	4															
358.0																				
11.7	SAND to SILT and SAND Compact to dense Light brown Wet		12	SS	32									○			0	79 21 0		
			13	SS	25															

Continued Next Page

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

MIS-MTO 001 OXTONGUE LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

RECORD OF BOREHOLE No OX-8

2 OF 3 **METRIC**

PROJECT 07-1191-0001-OX

W.P. 5550-04-00

LOCATION N 5027366.3 ; E 351205.5

ORIGINATED BY EHS

DIST HWY 60

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY MM

DATUM Geodetic

DATE November 13, 2008


CHECKED BY SEMC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	10	20	30		
	--- CONTINUED FROM PREVIOUS PAGE ---																
	SAND to SILT and SAND Compact to dense Light brown Wet		14	SS	18		354										
							353										
			15	SS	36											0 85 (15)	
							352										
			16	SS	19		351										
							350										
							349										
			17	SS	38		348									0 61 (39)	
							347										
							346										
			18	SS	32		345										
							344										
							343										
			19	SS	48		342										
							341										
340.6 29.1	COBBLES and BOULDERS in a sand and gravel matrix						340										

Continued Next Page

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

MIS-MTO-001 OXTONGUE_LOGS_1DECIMAL.GPJ GAL-MISS.GDT 24/11/09

PROJECT <u>07-1191-0001-OX</u>				RECORD OF BOREHOLE No OX-8				3 OF 3 METRIC										
W.P. <u>5550-04-00</u>				LOCATION <u>N 5027366.3 ; E 351205.5</u>				ORIGINATED BY <u>EHS</u>										
DIST <u> </u> HWY <u>60</u>				BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>										
DATUM <u>Geodetic</u>				DATE <u>November 13, 2008</u>				CHECKED BY <u>SEMC</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)					
--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
	GNEISS (BEDROCK)		1	RC	REC 97%													RQD = 23%
	Bedrock cored from 29.7 m to 33.1 m depth. For coring details, refer to Record of Drillhole OX-8.		2	RC	REC 100%													RQD = 64%
			3	RC	REC 100%													RQD = 88%
336.6																		
33.1	End of Borehole Note: 1. Water level in open borehole at a depth of 5.0 m below ground surface (Elev. 364.7 m) upon completion of drilling.																	

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

CHECKED: SEMC

PROJECT <u>07-1191-0001-OX</u>				RECORD OF BOREHOLE No OX-9				1 OF 1 METRIC										
W.P. <u>5550-04-00</u>				LOCATION <u>N 5027377.0 ; E 351223.8</u>				ORIGINATED BY <u>EHS</u>										
DIST <u> </u> HWY <u>60</u>				BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>				COMPILED BY <u>MM</u>										
DATUM <u>Geodetic</u>				DATE <u>November 12, 2008</u>				CHECKED BY <u>SEMC</u>										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					
370.4	GROUND SURFACE																	
0.0	Fine to medium sand, trace gravel, trace silt (FILL) Loose to compact Light brown Moist	X	1	SS	6													
			2	SS	7													4 93 (3)
			3	SS	11													
368.3																		
2.1	SAND, trace to some silt, trace gravel, layered with black metallic seams Compact Light brown Moist to wet	X	4	SS	10													
			5	SS	11													2 94 (4)
			6	SS	15													
			7	SS	15													
			8	SS	14													0 82 17 1
363.2																		
7.2	Sandy SILT, trace clay, layered Compact Light brown with dark brown specs Wet	X	9	SS	11													
			10	SS	17													0 22 77 1
360.7																		
9.7	End of Borehole																	
	Notes: 1. Water level measured in piezometer at 5.8 m and 5.7 m (Elev. 364.6 m and 364.7 m) on November 13 and 14, 2008 respectively. 2. Water level measured in piezometer at a depth of 5.6 m below ground surface level (364.8 m) on November 25, 2008.																	

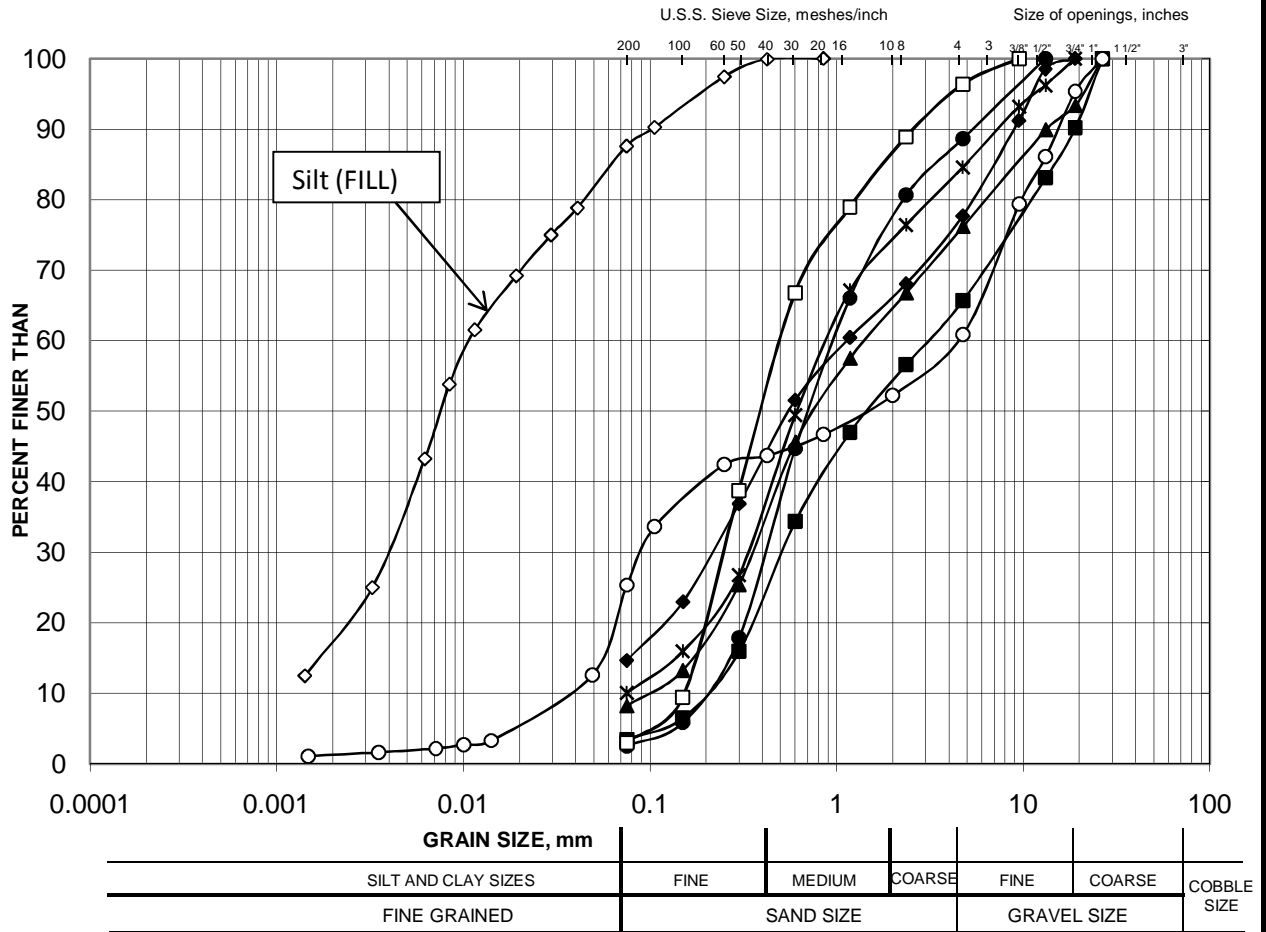


APPENDIX B

LABORATORY TEST RESULTS

GRAIN SIZE DISTRIBUTION Sand to Sand and Gravel (FILL)

**FIGURE
B-1**



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	OX-1	2	366.7
●	OX-2	3	362.4
■	OX-3	4	363.4
▲	OX-4	3	367.1
✱	OX-5	2	366.0
◇	OX-6	2	365.0
○	OX-7	3	360.7
□	OX-9	2	369.4

Project Number: 07-1191-0001-OX

Checked By: SEMC

Golder Associates

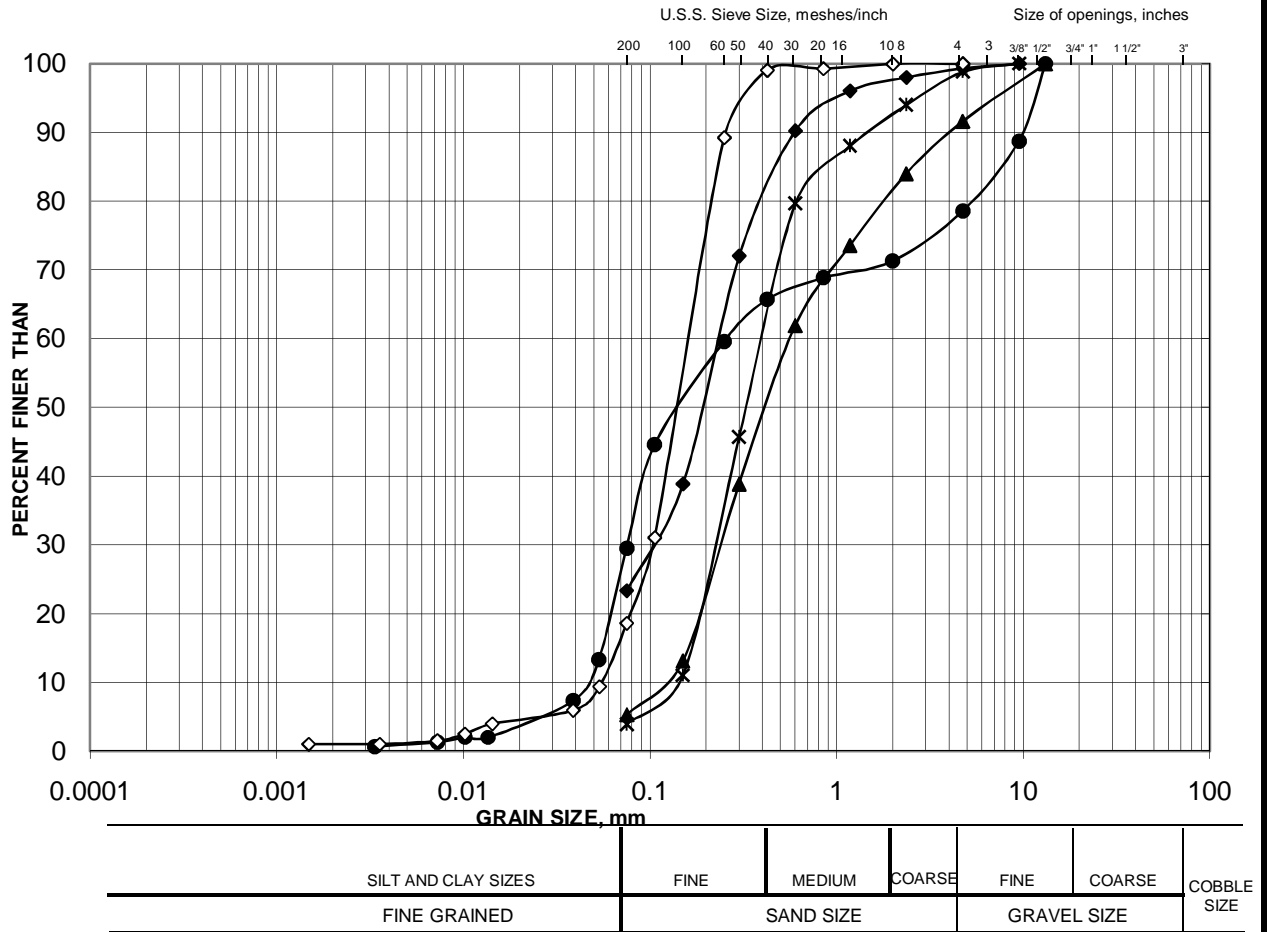
Date:

November 2009

GRAIN SIZE DISTRIBUTION

Upper Sand to Silty Sand

**FIGURE
B-2**



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	OX-1	5	364.5
●	OX-2	4b	361.5
▲	OX-2	6a	360.4
✱	OX-9	5	367.1
◇	OX-9	8	364.1

Project Number: 07-1191-0001-OX

Checked By: SEMC

Golder Associates

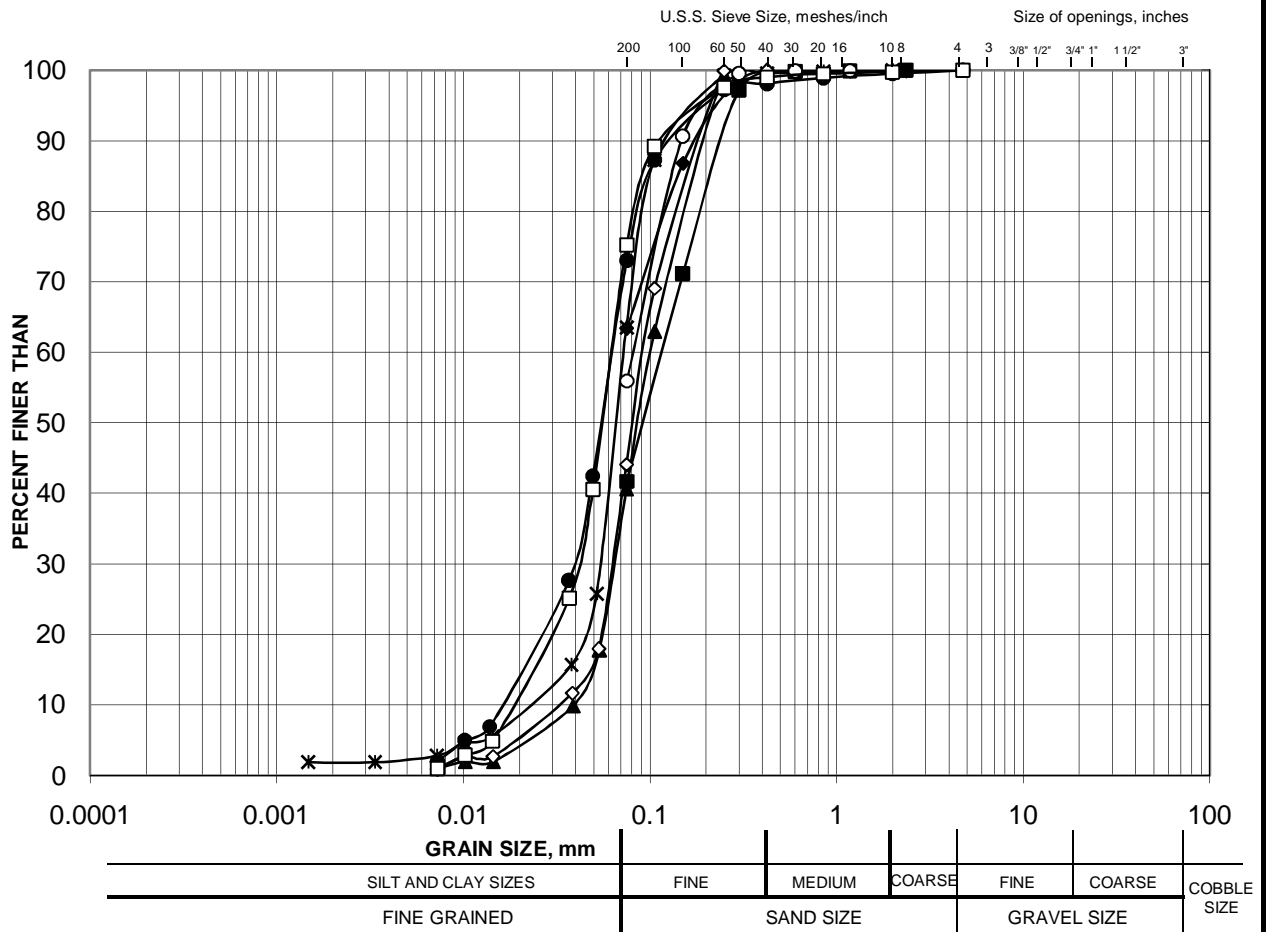
Date:

November 2009

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sandy Silt

FIGURE
B-3
page 1 of 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	OX-1	8	361.4
—●—	OX-2	6b	360.1
—■—	OX-2	8	358.6
—▲—	OX-2	15	347.9
—*—	OX-3	7	361.2
—◇—	OX-3	9	358.1
—○—	OX-3	11	355.0
—□—	OX-3	14	350.6

Project Number: 07-1191-0001-OX

Checked By: SEMC

Golder Associates

Date:

November 2009

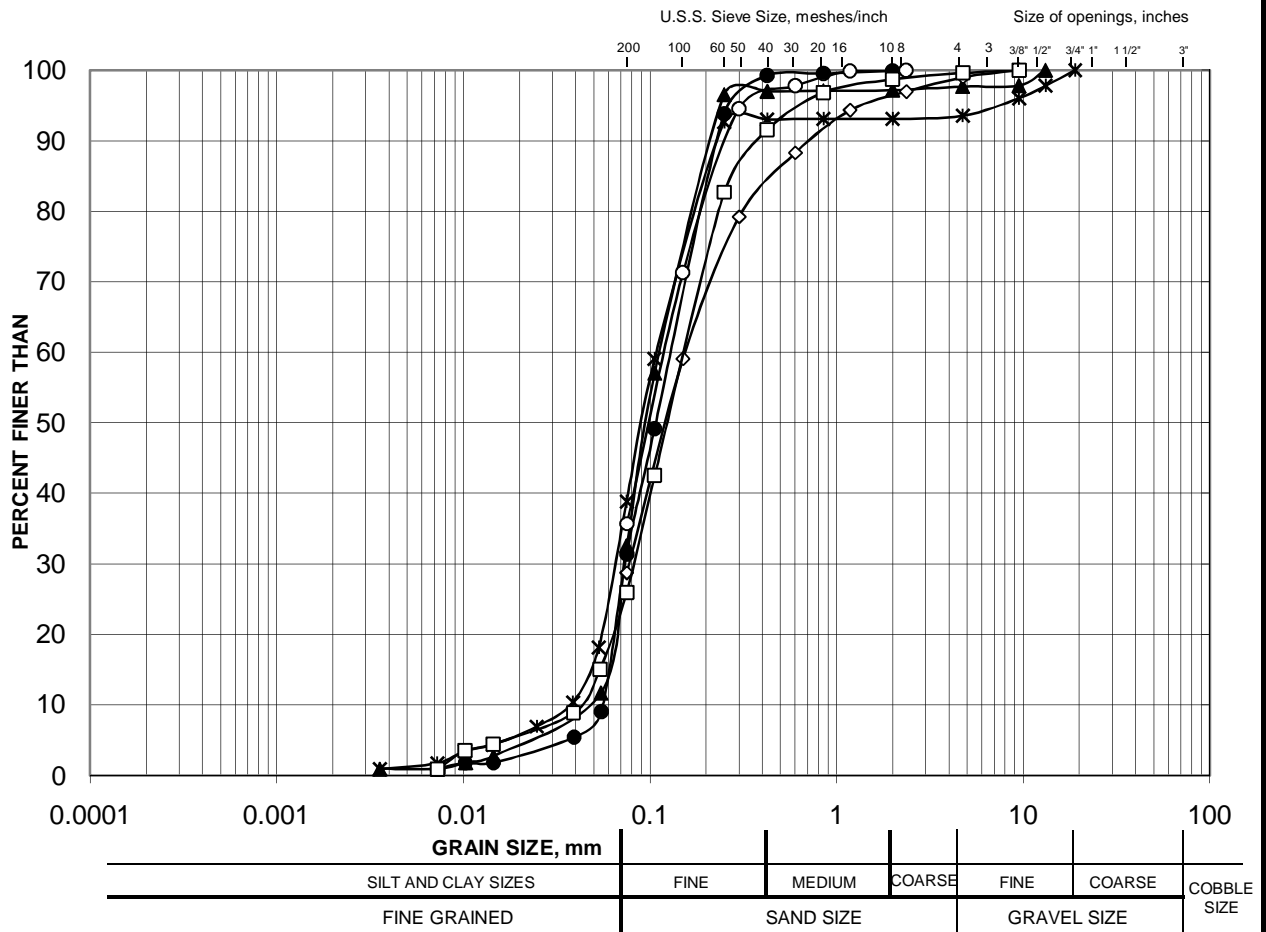
FIGURE
B-3
page 2 of 3

Date: November 2009

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sandy Silt

FIGURE
B-3
page 3 of 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	OX-6	16	347.7
▲	OX-7	5	359.2
✱	OX-7	8	356.8
◇	OX-8	3	367.9
○	OX-8	6	365.7
□	OX-8	7	364.8

Project Number: 07-1191-0001-OX

Checked By: SEMC

Golder Associates

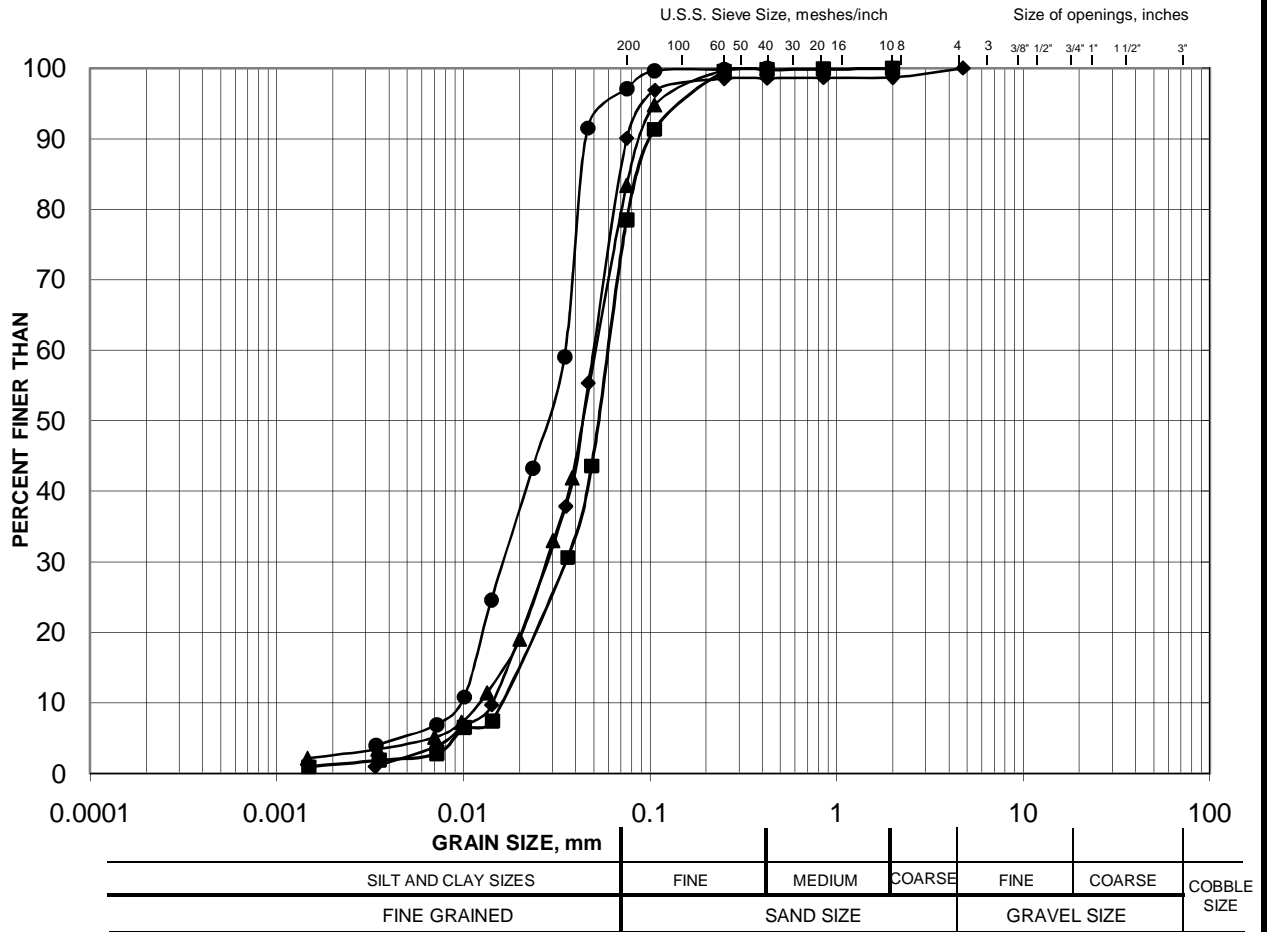
Date:

November 2009

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt

**FIGURE
B-4**



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	OX-2	11	354.1
—●—	OX-4	8	361.8
—▲—	OX-8	10	360.3
—■—	OX-9	10	361.0

Project Number: 07-1191-0001-OX
Checked By: SEMC

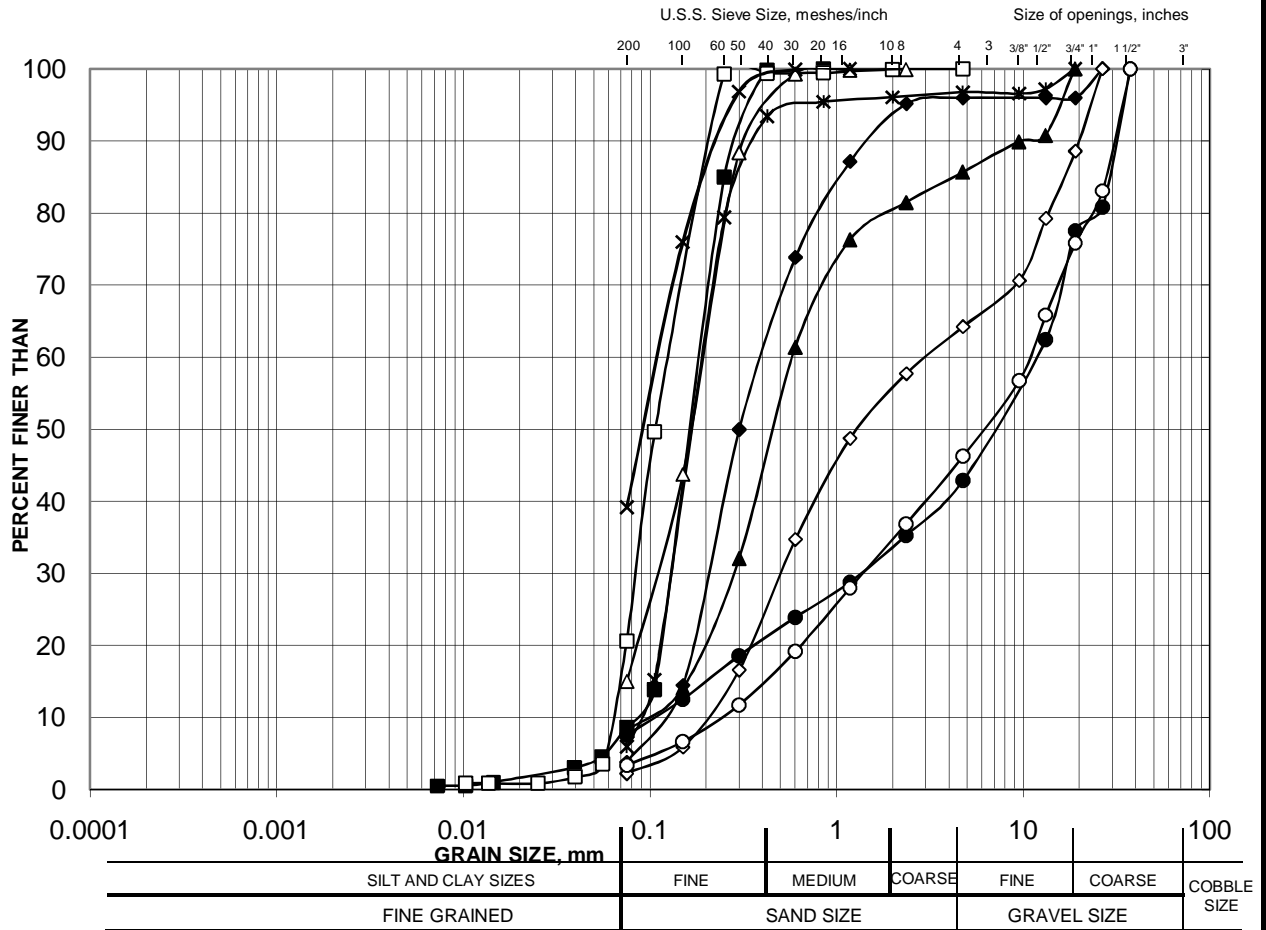
Golder Associates

Date: November 2009

GRAIN SIZE DISTRIBUTION

Sand to Sand and Gravel

**FIGURE
B-5**



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	OX-2	18	340.4
●	OX-3	16	347.5
■	OX-6	14	351.0
▲	OX-6	18	343.1
✱	OX-7	11	352.3
◇	OX-7	14	347.8
○	OX-7	16	344.7
□	OX-8	12	357.2
△	OX-8	15	352.7
✕	OX-8	17	348.1

Project Number: 07-1191-0001-OX

Checked By: SEMC

Golder Associates

Date:

November 2009

TABLE B-1
UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
HIGHWAY 60 OXTONGUE RIVER BRIDGE AND DETOUR
G.W.P 5550-04-00, SITE NO. 44-002

Borehole Number	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
OX-3	29.7	339.4	Gneiss	47.5	108.5
OX-6	35.7	330.5	Gneiss	47.5	136.4
OX-7	24.1	340.2	Gneiss	47.5	56.8
OX-8	30.9	338.8	Gneiss	47.5	87.1

Compiled by: TR
Checked by: SEMC
Reviewed By: FJH



APPENDIX C

NON-STANDARD SPECIAL PROVISIONS AND OPERATIONAL CONSTRAINTS

Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Special Provision

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to bedrock.
4. Place loose sand into 600 diameter CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

The CSP at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

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, or equivalent, on HP 310 x 110 Piles.

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.

Operational Constraint – Obstructions/Ground Control

Pile cap construction below the groundwater and/or river water levels must be carried out in-the-dry. The excavations shall be kept stable during the work.

The Contactor shall be alerted that the cohesionless soils at the site are water-bearing and susceptible to soil cave-in, sloughing and boiling. The contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction.

The Contractor shall be alerted that the fill and slope materials may contain rip-rap, cobbles and/or boulders and other obstructions.

The Contactor shall be alerted that timber cribbing and/or old shoring elements may be present at the site.

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

1.0 GENERAL

1.1 Scope

This special provision describes requirements for vibration monitoring of the existing structure during pile driving and shoring installation at the site.

2.0 DEFINITIONS

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

3.0 SUBMISSION REQUIREMENTS

The Contractor shall submit three (3) copies of the vibration monitoring plan to the Contract Administrator at least 3 weeks prior to the piling operations. The vibration monitoring shall satisfy the specifications and at a minimum contain the following specific information:

Name of Firm/QVE responsible for monitoring including qualifications of vibrations monitoring specialist;
Proposed instrumentation;
Proposed location of instruments on existing structure;
Proposed frequency of readings; and
Proposed methods for adjusting piling methods if readings show excess vibrations.

4.0 PROCEDURES

4.1 Locations of Vibration Monitoring Equipment

The vibration monitoring equipment shall be placed directly on the concrete foundations of the existing bridge abutments as close as possible to the pile driving and shoring installation operations.

5.0 MONITORING

5.1 Vibration Limits

The vibrations on the existing footing shall not exceed 100 mm/s (peak particle velocity).

5.2 Frequency of Readings

- 5.2.1 For pile driving operations; the Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing abutment. As a minimum, the readings should be taken and recorded during the first 3 m of driving and continuously during driving through the bouldery deposits and during seating of the pile onto the bedrock.
- 5.2.2 For shoring installation operations; the Contractor shall take readings continuously during installation of the main shoring elements, starting with the elements furthest away from the existing abutment.

5.3 Submission of Results

- 5.3.1 The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results. Additional submissions may be required at the discretion of the Contract Administrator. The results shall be immediately reviewed by the QVE and submitted to the Contract Administrator prior to the Contractor continuing with the remaining piles.
- 5.3.2 If the results are acceptable, the Contractor may continue installing the remaining piles/shoring elements with vibration monitoring readings being taken during driving of each pile during bedrock seating. The results of subsequent piles should be submitted to the Contract Administrator at the end of each day.
- 5.3.3 If the readings are not within the limits stated above, the Contractor must alter his driving/installation procedures until the vibrations on the existing structure are within acceptable levels. The above process must be repeated for each pile/shoring element.

6.0 CERTIFICATE OF CONFORMANCE (COC)

Upon completion of the work in each area of pile driving, the Contractor shall submit to the Contract Administrator a CoC sealed and signed by the QVE. The certificate shall state that the vibrations on the existing structure were below the limits stated above, and where the levels were exceeded, what procedures were used to reduce the vibrations to below the limits stated above.

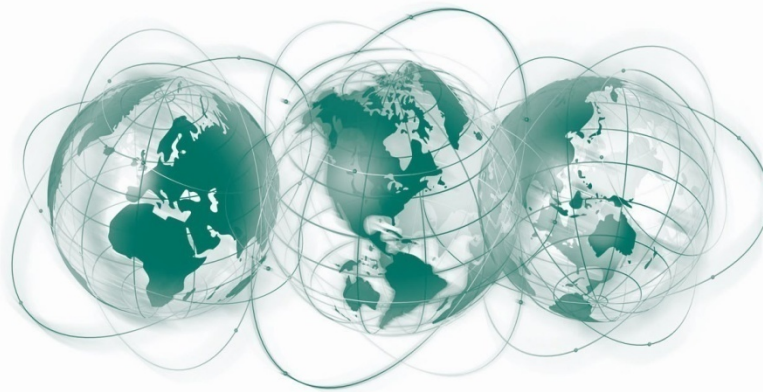
7.0 BASIS OF PAYMENT

Payment at the contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com



Golder Associates Ltd.
1010 Lorne Street
Sudbury, Ontario, P3C 4R9
Canada
T: +1 (705) 524 6861

