May 6, 2013

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF GROUNDHOG RIVER BRIDGE, SITE 39W-093
HIGHWAY 11 AT FAUQUIER
COCHRANE DISTRICT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5049-07-00

Submitted to:
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GEOCRES NO.: 42G-40

Report Number: 11-1191-0025-2

Distribution:

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REPORT

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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 preliminary foundation engineering services for the replacement of the Groundhog River Bridge (Site 39W-093), located on Highway 11 in Fauquier, Cochrane District, Ontario. This report presents the results of the preliminary foundation investigation carried out for the proposed replacement of the Groundhog River Bridge.

The Terms of Reference and the Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal dated June 2011. The work has been carried out in accordance with Golder's Supplementary Specialty Quality Control Plan for foundations engineering services for this project, dated December 2011. The Base Plan (General Arrangement Drawing) showing the alignment of Groundhog River Bridge was provided to Golder by URS in July 2011.

2.0 SITE DESCRIPTION

The Groundhog River Bridge carrying Highway 11 is situated immediately to the west of Fauquier, Cochrane District, Ontario. The surrounding land is generally flat, but slopes down towards the river, with a boat launch located on the east shore and residential development to the east of the boat launch (beyond 100 m east of the bridge). On the west side of the river, the topography is also generally flat-lying, with moderate tree cover and a bedrock outcrop west of the tree cover (beyond 100 m west of the bridge). The Ontario Northland Railway (ONR) bridge is located to the south and parallels the Groundhog River Bridge. The Groundhog River flows in a northerly direction and is approximately 150 m wide and about 4 m deep at the bridge location based on the drawing provided by URS. The river water level was surveyed at Elevation 213.1 m on April 15, 2012, by Callon Dietz Inc. (Callon Dietz) under subcontract to URS.

The existing Highway 11 Bridge was constructed in 1939 and consists of a 10 m wide, 180 m long, four-span structure comprised of two 76 m long steel trusses and two 14 m long concrete approach slabs.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this subsurface investigation was carried out on July 25 and 26 and December 13, 2012, at which time Boreholes GHR-1 and GHR-1a were advanced at the proposed east abutment and GHR-2 was advanced at the proposed west abutment, approximately at the locations shown on Drawing 1.

The boreholes were advanced using a CME 850 track-mounted drill rig supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario. The boreholes were advanced using 108 mm inner diameter continuous flight hollow stem augers to the bedrock surface and soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Field vane shear tests were carried out in the cohesive deposit for determination of undrained shear strengths (ASTM D2573, Field Vane Strength Shear Test) using an MTO Standard 'N' size vane. One sample of the cohesive deposit was obtained using a 76 mm O.D. thin-walled 'Shelby' tube (ASTM D1587, Thin-Walled Tube Sampling) for a relatively undisturbed sample in Borehole GHR-1. The boulders were cored in Borehole GHR-1a and the bedrock was cored in Borehole GHR-2 using NW casing and a NQ size core barrel. The boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The groundwater conditions were observed in the open boreholes during, and immediately following, the drilling operations and a standpipe piezometer was installed in Borehole GHR-2 to permit monitoring of the groundwater level. The piezometer consists of a 50 mm diameter polyvinyl chloride pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite pellets to create a seal, then backfilled to near ground surface with cuttings from the borehole mixed with bentonite. A seal of bentonite was then placed over the backfill to ground surface. The piezometer installation details and water level readings are indicated on the Record of Borehole sheets in Appendix A.

The fieldwork was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling and in situ testing operations and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's Sudbury Laboratory for further examination and laboratory testing. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. One Uniaxial Compression Strength (UCS) test was carried out on a select sample of the bedrock core. The geotechnical laboratory testing was completed according to applicable MTO LS standards.

The borehole locations and elevations were measured in the field by Golder personnel relative to the stakes installed by Callon Dietz. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths are shown on Drawing 1 and presented on the Record of Borehole sheets in Appendix A and are summarized below.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
GHR-1	5 464 438.5	229 164.2	216.8	10.4
GHR-1a	5 464 439.7	229 164.2	216.8	11.4
GHR-2	5 464 505.7	228 986.5	216.1	5.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on NOEGTS[1] mapping, the subsoils in the vicinity of the Groundhog River Bridge site are characterized as an alluvial plain deposit consisting of silty soils.

Based on geological mapping by the Ministry of Natural Resources[2], the site is underlain by bedrock of the Early Precambrian era consisting of granitic, metasedimentary or minor metavolcanic migmatite.

4.2 Subsurface Conditions

The preliminary foundation investigation consisted of drilling two boreholes advanced in the vicinity of the existing Groundhog River Bridge. The borehole locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawing 1. The detailed subsurface soil and groundwater

[1] Northern Ontario Engineering Geology Terrain Study, Digital Maps, Ontario Geological Society Map Reference Number 42GSE.

[2] Ministry of Natural Resources, Geological Highway Map, Ontario Geological Survey, Map 2440.

conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of the in situ tests (i.e. SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole Sheets and in Section 4 are uncorrected. The results of geotechnical laboratory testing are presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile on Drawing 1 are inferred from non-continuous sampling and, therefore, represents transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site generally consist of organic silty topsoil, fill material, and deposits of clayey silt, silty clay, sand and silt, silty sand and sand, underlain by boulders in Borehole GHR-1a and bedrock in Borehole GHR-2.

4.2.1 Fill

At Borehole GHR-1, an approximately 1.7 m thick deposit of fill comprised of an upper 0.2 m thick layer of silty topsoil, underlain by 1.5 m of silt was encountered from ground surface (Elevation 216.8 m).

Two Standard Penetration Test (SPT) "N"-values measured in the fill deposit are 8 blows and 11 blows per 0.3 m of penetration, indicating a loose to compact relative density.

4.2.2 Silty Topsoil

In Borehole GHR-2, a 1.4 m thick layer of silty topsoil was encountered from ground surface (Elevation 216.1 m).

Two SPT "N"-values measured in the silty topsoil range are 8 blows and 10 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural moisture content of a sample of the silty topsoil is about 19 per cent.

4.2.3 Clayey Silt

A deposit of clayey silt was encountered below the fill in Borehole GHR-1 at a depth of 1.7 m below ground surface (Elevation 215.1 m). In Borehole GHR-2, a clayey silt deposit was encountered at a depth of 2.2 m below ground surface (Elevation 213.9 m), underlying the sand and silt deposit discussed in Section 4.2.4. The thickness of the deposit is 0.5 m and 1.2 m in Boreholes GHR-1 and GHR-2, respectively.

The SPT "N"-values measured in the clayey silt deposit are from 0 blows (weight of hammer) and 8 blows per 0.3 m of penetration, indicating a very soft to firm consistency.

Atterberg limits testing was carried out on two selected samples of the clayey silt deposit and measured liquid limits of 20 per cent and 31 per cent, plastic limits of 15 per cent and plasticity indices of 5 per cent and 16 per cent. These results, which are plotted on a plasticity chart on Figure B1 in Appendix B, indicate that the tested sample consist of clayey silt of low plasticity.

The natural moisture content measured on two samples of the clayey silt deposit are 18 per cent and 22 per cent. The organic content measured on a sample of the clayey silt deposit in Borehole GHR-1 is 0.5 per cent (slightly organic).

4.2.4 Sand and Silt

A 3.4 m and 0.8 m thick deposit of sand and silt was encountered below the clayey silt in Borehole GHR-1 at a depth of 2.2 m below ground surface (Elevation 214.6 m) and below the silty topsoil in Borehole GHR-2 at a depth of 1.4 m below ground surface (Elevation 214.7 m).

The measured SPT "N"-values in the sand and silt deposit range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

Grain size distribution tests carried out on two samples of the sand and silt deposit are shown in Figure B2 in Appendix B.

The natural moisture content measured on two samples of the sand and silt deposit are 16 per cent and 25 per cent.

4.2.5 Silty Clay

A 3.9 m thick deposit of silty clay was encountered below the sand and silt deposit in Borehole GHR-1 at a depth of 5.6 m below ground surface (Elevation 211.2 m).

The SPT "N"-values measured in the silty clay range are 2 blows and 5 blows per 0.3 m of penetration. In situ field vane testing carried out in the silty clay measured two undrained shear strengths of 78 kPa and 56 kPa, with sensitivities of 4 and 6, respectively. The in situ vane test result, together with the SPT 'N'-values, suggest that the silty clay deposit generally has a firm to stiff consistency.

Atterberg limits testing was carried out on two selected samples of this deposit and measured liquid limits of 45 per cent and 46 per cent, plastic limits of 19 per cent and 20 per cent and plasticity indices of 26 per cent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, indicate that the tested samples are classified as silty clay of intermediate plasticity.

The natural moisture content measured on two samples of the silty clay deposit is 38 per cent.

4.2.6 Silty Sand to Sand

A 0.9 m and 0.4 m thick deposit of silty sand to sand was encountered below the silty clay deposit in Borehole GHR-1 at a depth of 9.5 m (Elevation 207.3 m) and below the clayey silt deposit in Borehole GHR-2 at a depth of 3.4 m (Elevation 212.7 m), respectively.

4.2.7 Boulders

Borehole GHR-1a was advanced about 1.2 m north of Borehole GHR-1 without sampling to a depth of 10 m (Elevation 206.8 m), which is approximately 0.4 m above the bottom of Borehole GHR-1, and encountered a 1.4 m thick deposit of boulders and gravel seams. Borehole GHR-1a was then advanced below a depth of 10 m by NQ coring to the borehole termination depth at Elevation 205.4 m.

4.2.8 Bedrock

In Borehole GHR-2, bedrock was encountered at a depth of 3.8 m below ground surface, corresponding to Elevation 212.3 m and was cored for a depth of 1.6 m. The retrieved bedrock core is described as very coarse grained, grey, gneiss. A photograph of the retrieved bedrock core is shown on Figure B4 in Appendix B.

The Total Core Recovery (TCR) from Borehole GHR-2 is 100 per cent. The Rock Quality Designation (RQD) measured on the core run is 100 per cent, indicating a rock mass of excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)[3].

Laboratory UCS testing was carried out on one sample of the bedrock core and yielded a compressive strength of 97 MPa. The UCS value is presented on the Record of Drillhole sheet in Appendix A and indicates that the bedrock is strong (R4, 50<USC< 100 mPa) as per Table 3.5 of CFEM (2006)[3].

4.2.9 Groundwater Conditions

The water level in Borehole ONR-1 upon completion of drilling was measured at a depth of 8.6 m below ground surface, corresponding to Elevation 208.2 m. The groundwater level measured in the piezometer installed in Borehole GHR-2, sealed within the sand deposit/bedrock, was 3.1 m below ground surface corresponding to Elevation 213.0 m on August 1, 2012, approximately one week after installation.

Groundwater levels are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

5.0 CLOSURE

The field drilling program was supervised by Mr. Ed Savard and Mr. Indy Dumpis. This Preliminary Foundation Investigation Report was prepared by Mr. Adam Core, E.I.T. and reviewed by Mr. Andre Bom, P.Eng. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, carried out an independent quality control review of this report.

[3] Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition, BiTech Publications.

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http://capws.golder.com/sites/capws2/p111910025groundhogriverandonrbridges/reports/ghr/final report/11-1191-0025-2 final rpt 13may6 prelim fidr ghr.docx

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT OF GROUNDHOG RIVER BRIDGE, SITE 39W-093
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Groundhog River Bridge (Site No. 39W-093) located on Highway 11 west of Fauquier, Cochrane District, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project. Those requiring information on construction aspects should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

The existing four-span Groundhog River Bridge structure was constructed in 1939 and the abutments and piers are likely supported on shallow foundations as shown on the General Arrangement drawing dated May 1987, for the rehabilitation of the existing bridge. Due to the age and poor condition of the existing bridge, replacement will be required. We understand that the new structure will be skewed to the north of the existing bridge, with the center of the new east and west abutments located 16 m and 30 m from the center of existing east and west abutments, respectively.

The proposed finished grade for the new Highway 11 alignment as provided by URS will be Elevation 224.9 m at the east abutment and Elevation 224.0 m at the west abutment. The new east and west approach embankments will be up to approximately 8 m and 9 m high, respectively, relative to the existing natural ground surface at the abutments, and up to about 12 m above the Groundhog River water level (approximately Elevation 213.1 m) surveyed in April 2012 by Callon Dietz. The current General Arrangement drawing provided by URS indicates that the new 190 m long four-span structure will consist of two middle 55 m long spans and two outer 40 m long spans with the three piers located in the Groundhog River.

Based on the proposed bridge geometry and the subsurface conditions at this site, both shallow and deep foundations founded on bedrock have been considered for support of the west abutment of the replacement structure. Due to the presence of thicker soil deposits including a firm to stiff silty clay deposit encountered at the east abutment, deep foundations have been considered for support of the east abutment of the replacement structure. At Detail Design, additional boreholes should be advanced at the location of the proposed abutments for the preferred alignment, as well as at the pier locations. A summary of the advantages and disadvantages associated with each abutment foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Driven and socketed steel H-piles:** Driven steel H-piles founded on bedrock are feasible for support of the west abutment and the steel H-piles may require socketing into the strong bedrock depending on the required minimum pile length at this location. At the east abutment, driven steel H-piles founded on the boulders are feasible for support of the abutment. Depending on the thickness of the boulder deposit and depth to bedrock, consideration should be given at the Detail Design stage to drilling through the boulder deposit and founding the piles on bedrock.

- **Driven steel pipe (tube) piles:** Driven steel pipe (tube) piles founded on bedrock at the both abutments and socketed into bedrock at the west abutment, if required, could also be considered as a deep foundation option for support of the abutments.

- **Strip or spread footings founded on bedrock at the west abutment and deep foundations at the east abutment:** Strip or spread footings founded on bedrock are feasible for support of the new west abutment but the subsurface conditions are not suitable for the support of a shallow foundation at the east abutment. Therefore, different types of foundations would need to be used in the design. Based on the river water level measured in April 2012 at Elevation 213.1 m and the piezometer at Borehole GHR-2 in July 2012, excavations below the groundwater/river water level and unwatering within a temporary shored excavation will likely be required to expose the bedrock surface and for construction of the footings in the dry.

- **Caissons:** Caissons socketed into the bedrock are also considered to be feasible for a deep foundation option at this site. However, caissons are not generally constructed in Northern Ontario due to constructability issues associated with socketing the large diameter caissons within the strong bedrock. Tremie concrete construction methods would likely be required, at least for the east abutment.

The following sections provide recommendations for deep foundation options for both east and west abutments and the shallow foundation option at the west abutment to support the proposed replacement structure. From a foundations perspective, driven steel H-piles founded on the bedrock at both abutments are recommended at this site.

6.3 Driven/Socketed Steel H-Pile or Steel Pipe (Tube) Foundations

6.3.1 Founding Elevations

The abutments may be supported on steel H-piles or steel pipe (tube) piles driven into the boulder deposit at the east abutment and driven to or socketed into bedrock at the west abutment depending on the elevation of the underside of the west abutment. The following pile tip elevations may be used for preliminary design purposes:

Foundation Element (Borehole Number)	Approximate Estimated Design Elevation of Pile Tip (m)	Assumed Elevation of Underside of Pile Cap[1] (m)	Approximate Estimated Pile Length (m)
East abutment (GHR-1 and GHR-1a)	206	217.7	12
West abutment (GHR-2)	212	216.3	4[2]

1. From General Arrangement drawing (approximate).
2. Socketing into strong bedrock may be required depending on the elevation of the underside of the pile cap and the minimum pile length required for structural design.

The pile caps should be constructed at a minimum depth of 2.6 m below ground surface or provided with a similar thickness of soil cover for frost protection purposes, per OPSD 3090.100 (*Foundation Frost Penetration Depths for Northern Ontario)*.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the presence of boulders at the east abutment. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are more likely to "hang up" or deflect away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, as should be installed in accordance with OPSS 903 *(Deep Foundations).* In the boulder deposit, driving shoes such as Titus Standard "H" Bearing Pile Points are preferred over flange plates as specified in OPSD 3000.100 (*Steel H-Pile Driving Shoe*). If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).* Alternately, a heavier pile section could be used.

6.3.2 Geotechnical Axial Resistance

At the west abutment, for HP 310X110 piles driven to bedrock or placed in bedrock sockets, if required, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for the design. This value represents a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored geotechnical axial resistances at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

At the east abutment, for HP 310X110 piles driven into the boulder deposit, a factored geotechnical axial resistance at ULS of 1,600 kN and a geotechnical reaction at SLS (for 25 mm of settlement) of 1,100 kN may be used for preliminary design. If a greater geotechnical axial resistance is required, consideration should be given to drilling through the boulder deposit and founding the H-Piles on bedrock.

Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm ($^3/_8$ in). At Detail Design, pile downdrag loads at both abutments will need to be considered unless the settlement of the cohesive soils is mitigated. Settlement mitigation is discussed in Section 6.5.4.

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the 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 allow seating on the bedrock surface, if applicable, and to also avoid overdriving and possibly damaging the piles.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during Detail Design in consideration of the additional subsurface investigation at the foundation elements.

6.4 Shallow Foundations

6.4.1 Founding Elevations

For support of the new west abutment, as an alternative to deep foundations, consideration could be given to supporting the abutment on strip or spread footings founded on the bedrock surface. The following founding elevation is recommended for preliminary design of the west abutment footing. For the footings founded directly on the bedrock, frost protection is not required.

Foundation Element (Borehole)	Approximate Founding Elevation (m)	Approximate Excavation Depth Below Existing Ground surface (m)	Approximate Depth of Excavation Below Groundhog River Level[1] (m)
West Abutment (GHR-2)	212	4	1.1

1. Assumes a river water level at Elevation 213.1 m (measured April 15, 2012 by Callon Dietz)

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with OPSS 902 *(Excavating and Backfilling - Structures)* to check that the founding elevation is reached and that all unsuitable material, including loose soil materials and fractured rock, have been removed.

6.4.2 Geotechnical Resistance

Strip or spread footings for the west abutment placed on the properly prepared bedrock surface at the elevation given in the preceding section should be designed based on the factored geotechnical axial resistances at ULS of 10,000 kPa. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be unyielding materials; as such, ULS conditions will govern for this foundation type.

The Preliminary Design geotechnical axial resistance values provided above will have to be re-evaluated and modified as necessary during Detail Design, based on future additional subsurface investigation at the proposed west abutment.

6.5 Approach Embankments

6.5.1 Removal of Existing Fill and Topsoil

From ground surface, Boreholes GHR-1 at the east abutment encountered an approximately 1.7 m thick deposit of fill and Borehole GHR-2 encountered a 1.4 m thick deposit of silty topsoil. The fill and organic soil should be removed from below the final embankment footprint of the proposed approach embankments. The excavations should be backfilled with appropriate granular material as discussed in Section 6.5.2. Excavations for this purpose should be in accordance with OPSS 902 *(Excavating and Backfilling - Structures).*

6.5.2 Subgrade Preparation and Embankment Construction

The embankment fill for the realigned Highway 11 should be placed and compacted in accordance with OPSS 501 *(Compacting)* and MTO's SP 206S03 *(Earth Excavation and Grading)*. Where the new embankment will encroach onto the existing embankment, benching of the existing highway embankment should be carried out to "key in" the new fill materials for the realignment, in accordance with OPSD 208.010 *(Benching of Earth Slopes).*

The fill for construction of the new approach embankments may consist of either rock fill or granular fill (sand and gravel). The assessment of embankment stability and settlement discussed in Section 6.5.3 and 6.5.4, respectively, has considered the use of either rock fill or granular fill for embankment construction.

The abutments front slope and side slopes adjacent to the river should be provided with erosion protection in accordance with OPSS 511 (*Rip Rap, Rock Protection and Granular Sheeting*). 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 R-10 Rip Rap (300 mm diameter as per OPSS 1004, Aggregates), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding as per OPSS 802 (*Topsoil*) and OPSS 804 (*Seed and Cover*) should be carried out as soon as possible after construction of the embankments (unless rock 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 as per OPSS 511 (*Rip Rap, Rock Protection and Granular Sheeting*) to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the Spring prior to topsoil dressing and seeding.

6.5.3 Approach Embankment Stability

Slope stability analyses have been carried out for the proposed embankment using the commercially available program GeoStudio 2007 (Version 7.17), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed approach embankments at this site considering the design requirements and the available field and laboratory testing data.

The stability analysis carried out for the Preliminary Design is focused on the east side of Groundhog River as this borehole advanced in this area encountered the greatest thickness of the soft/loose foundation soils (Borehole GHR-1) compared with the west side of the river (Borehole GHR-2). Further, the analysis is focused on the east embankment front slope of the proposed bridge; stability for the side slopes should be addressed during Detail Design when additional subsurface information along the approach embankment is obtained. The stability analyses were completed for the 9 m high embankment based on the subsurface conditions as encountered in Borehole GHR-1 and the cross-section geometry provided by URS. The following parameters have been used in the analyses, based on field and laboratory test data, as well as empirical correlations found in literature [Kulhawy and Mayne (1990)]:

Soil Deposit at East Approach (Borehole GHR-1)	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
New Granular Fill (i.e. Sand and Gravel)	21	35°	-
New Rock Fill as an alternative to Granular Fill	19	40°	-
Very Soft Clayey Silt	17	-	20
Very Loose Sand and Silt	18	27°	-
Firm to Stiff Silty Clay	17	-	50
Silty Sand	19	30°	-

For the east approach, the analysis indicate that the new 9 m high embankment constructed with new granular fill (i.e. sand and gravel) with front slopes of 2H:1V or new rock fill with front slopes of 1.25H:1V will have a Factor of Safety (FoS) less than 1.3 against global instability due to the presence of very soft clayey silt and very loose sand and silt deposits. In order to mitigate the east approach embankment instability, to achieve a FoS greater than 1.3, sub-excavation of the very soft/very loose subsoils to Elevation 211.2 m (5.6 m below existing ground surface) and replacement with new granular fill is recommended, as shown on Figure 1.

For the east approach embankment, as an alternative to full sub-excavation adjacent to the existing embankment, consideration could be given to sub-excavating the existing fill deposit to Elevation 215.1 m (1.7 m below existing ground surface), replacing with granular fill and constructing a 7 m long wide berm on the front slope to achieve a FoS greater than 1.3, as shown on Figure 2. For the use of rock fill for new embankment construction, the rock fill toe berm at the front slope should be 10 m wide. The stability analysis indicates that the proposed east approach side slope has a FoS greater than 1.3 without the need for toe berms.

For the new west approach embankment, which will be up to about 8 m high, sub-excavation of the silty topsoil and loose sand and silt deposits is recommended to Elevation 213.9 m (2.2 m below existing ground surface) and replacement with granular fill or rock fill.

Depending on the final extent of local excavation and exposed height of the east and west approach embankments, mid-height berms may be required and should be incorporated along the extent of the embankment side slopes such that the uninterrupted slope is not greater than 8 m high for a granular fill embankment and 10 m high for a rock fill embankment, consistent with OPSD 202.010 (*Slope Flattening*).

This Preliminary Design assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during Detail Design.

6.5.4 Approach Embankment Settlement

Settlement of the approach embankments can be expected as a result of the loading on the compressible foundation soils from the new embankments. Settlements may also occur due to compression of the new embankment fill itself.

Settlement analyses were carried out using hand calculations for the following cases:

- Sub-excavation of the very soft/very loose subsoils to Elevation 211.2 m and 213.9 m at the east and west approach embankments, respectively, as recommended in Section 6.5.3; and
- As an alternative to sub-excavation to Elevation 211.2 m at the east approach, sub-excavation to Elevation 215.1 m could be considered, as discussed in Section 6.5.3.

The cross-section model for the east approach embankment is considered the more critical condition due to the greater thickness of the cohesive deposit below the east approach embankment relative to the west approach embankment. Due to the relatively thin deposits of cohesionless native soils below the silty clay deposit encountered in Boreholes GHR-1 and GHR-2, settlements from the proposed embankment geometry are expected to be negligible in these lower deposits and will occur during and immediately following embankment construction. The model geometry and stratigraphy used in the settlement analysis for the east approach are shown on Figure 1 for the case of sub-excavation to Elevation 211.2 m and Figure 2 for the case of sub-excavation to Elevation 215.1 m.

The consolidation settlement of the firm to stiff silty clay deposit below the east approach embankment was assessed using the results of the in situ field vane tests, the laboratory tests (i.e. water contents and Atterberg limits) and the estimated deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Koppula (1986) and NAVFAC (1986).

The following correlation proposed by Mesri (1975) was employed to estimate the preconsolidation pressure using the in situ field vane tests:

$$\sigma'_p = s_u/0.22$$

where:

s_u = average mobilized undrained shear strength (kPa)

σ'_p = preconsolidation pressure (kPa)

Bjerrum's (1973) correction factor was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \quad \text{(after Bjerrum, 1973)}$$

where:

$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)

$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)

μ = Bjerrum's correction factor based on Plasticity Index

The following consolidation parameters were estimated for the silty clay deposit at the east approach based on empirical correlations with the laboratory tests results performed on samples of the silty clay obtained from Borehole GHR-1:

Location	Approximate Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_c	C_r
East Abutment (GHR-1)	211.2 to 207.3	95	230	2.4	0.98	0.55	0.055

Case 1, Sub-excavation to Elevation 211.2 m below the East Approach

The primary consolidation of the cohesive soils below the east approach between Elevation 211.2 m and 207.3 m is estimated to be approximately 170 mm. Based on an estimated coefficient of consolidation (c_v) of 5 x 10^{-3} cm^2/s and assuming two-way drainage of the approximately 3.9 m thick cohesive deposit, it is estimated that 90 per cent of the primary consolidation settlement will be completed in about 3 months. At this Preliminary Design stage, to mitigate the settlement of the cohesive deposit, either full sub-excavation of the deposit or preloading the new approach embankments for a minimum of 3 months are both applicable alternatives. The sub-excavation alternative is considered feasible and the recommended alternative at the west approach, where the excavation would be up to about 3.0 m deep; whereas the preloading alternative is the recommended alternative at the east approach embankment as the sub-excavation would have to be up to 9.5 m deep.

In addition to primary consolidation within the silty clay deposit, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationship has been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at the east approach:

$$S_c = HC_{\alpha\varepsilon}log(\frac{t}{t_{EoP}})$$

where :

S_c = secondary compression (creep) settlement (mm)

$C_{\alpha\epsilon}$ = modified secondary compression index as correlated with water content or from the results of laboratory tests

H = initial thickness of compressible clay deposit (mm)

t = post-construction period of interest (10 years)

t_{EoP} = time to reach end of primary consolidation (years)

The magnitude of creep settlement for the cohesive deposit is estimated to be approximately 30 mm per log-cycle of time for this area, corresponding to about 30 mm over a 10-year period following completion of construction.

Case 2, Sub-excavation to Elevation 215.1 m below the East Approach

In addition to the primary and secondary settlement described above, if the subsoils between Elevations 215.1 m and 211.2 m are left in place, settlement of these soils are estimated to be approximately 300 mm. This settlement is anticipated to be completed immediately or soon after completion of embankment construction.

If granular fill is used for embankment construction, the above estimates do not include compression of the granular fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 per cent to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction. Non-granular earth fill materials are not recommended for embankment construction as they may exhibit some additional settlement over time depending on their gradation, plasticity and field compaction effort.

Should rock fill be considered for embankment construction, post-construction settlement of the rock fill will need to be considered. Based on MTO's "*Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates*" (2010), the settlement of the rock fill is estimated to be up to about 75 mm short-term and 10 mm long term at the east and west approach embankments for the case where sub-excavation extends to Elevation 215.1 m and Elevation 213.9 m, respectively. For sub-excavation to Elevation 211.2 m at the east approach, the rock fill settlement will be about 105 mm short-term and 15 mm long-term. Therefore, if rock fill is used for embankment construction at this site, preloading of the rock fill embankment for a minimum of six months will be required to mitigate the settlement to meet the MTO's "*Embankment Settlement Criteria for Design*" Final Draft dated March 2, 2010, which specifies that total post-construction settlement shall not be greater than 25 mm within 20 m of the bridge abutments.

This Preliminary Design assessment of the settlement of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during Detail Design.

6.6 Construction Considerations

The following subsections identify construction issues that should be considered at this stage of the design as they may impact the planning for Detail Design of the project. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during Detail Design for incorporation into the Contract Documents.

6.6.1 Excavation and Temporary Roadway Protection

As discussed in Section 6.5.3, sub-excavation of the fill, topsoil, clayey silt and sand and silt deposit for embankment construction should extend up to Elevation 211.2 m at the east approach and Elevation 213.9 m at the west approach. The sub-excavation of the loose/soft subsoils should be carried out using construction procedures in accordance with OPSS 209 (*Embankments Over Swamps and Compressible Soils*). Further, where sub-excavation will occur immediately adjacent to the existing toe of the north embankment, the excavation limits should incorporate the guidelines of OPSD 203.020 (*Embankments Over Swamp, Existing Slope Excavated to 1H:1V*), modified to remove the restrictions on the height of the embankment and the depth of excavation (i.e. Note A). This Standard provides guidance for the temporary excavation of existing slopes at a 1H:1V profile to allow for the potential removal of a larger extent of sub-excavation at the toe.

Open-cut excavations into the subsoils at this site should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill, topsoil and native soils at this site would be classified as Type 4 soil. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 3H:1V.

6.6.2 Groundwater and Surface Water Control

Excavation within the plan limits of the proposed works to remove organic and/or soft/loose deposits prior to embankment fill placement may extend below the water table. Groundwater flow into the excavations will occur due to the presence of relatively permeable deposits and is dependent on the groundwater/river water levels at the time of construction. Unwatering is not likely required for the excavation and backfilling for embankment construction, however, surface water should be directed away from the excavations at all times.

Excavations to construct spread footings at the west abutment (if considered) will likely extend below the groundwater/river water level. While the use of a sheet pile shoring system should be considered to control groundwater in the excavation to bedrock at this location, given the potential variable bedrock surface and the strong classification of the bedrock, and the presence of a cohesionless layer immediately above the bedrock surface, it will not be possible to toe the sheet piling into the rock to achieve complete cut-off of inflows. Additional boreholes should be advanced at the west abutment during Detail Design to determine the bedrock surface elevation if spread footings are being considered and develop recommendations for temporary excavation support and unwatering system(s).

Temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2, as specified in OPSS 539.

6.7 Recommendations for Further Work during Detail Design

Additional boreholes will be required within each of the foundation elements and within the approach embankment areas during the Detail Design stage of investigation, to further assess and/or confirm the subsurface conditions and the Preliminary Design recommendations provided herein, as follows:

- Abutments:
  - assessment of the depth and extent of the fill and/or topsoil deposit below the abutment footings/pile caps;
  - assessment of the elevation of the bedrock at the west abutment for shallow and deep foundation considerations, and to confirm refusal conditions at the east abutment for deep foundations;
  - confirmation of the tip elevation for driven steel H-piles including assessment of "refusal" condition for end bearing piles;
  - confirmation of the stabilized groundwater elevation; and
  - observation of the presence of cobbles and/or boulders within the native cohesionless deposits to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of deep foundations.
- Approach embankments:
  - assessment of the depth and extent of the fill and/or topsoil within the footprint of the new approach embankments;

- further assessment of the stability of the embankment front slopes and side slopes; and
- further assessment of the estimated magnitude of settlement under the new approach embankments, including consolidation testing of sample(s) of the cohesive deposit encountered below the proposed east approach.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

Report Signature Page

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http://capws.golder.com/sites/capws2/p111910025groundhogriverandonrbridges/reports/ghr/final report/11-1191-0025-2 final rpt 13may6 prelim fidr ghr.docx

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Bjerrum, L., 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State of the Art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.

Koppula, S.D., 1986. Discussion: Statistical Estimation of Compression Index, Geotechnical Testing Journal, ASTM, Vol. 4, No. 2, pp. 68-73.

Kulhawy, F.H. and Mayne, P.W., 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Mesri, G., 1975. Discussion on New Design Procedure for Stability of Soft Clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.

Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

GeoStudio (Version 7.17) by Geo-Slope International Ltd.

Ministry of Transportation Ontario Special Provisions

SP 110S13	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Earth Excavation, Grading

Ministry of Transportation Ontario. Guidelines for Rock Fill Settlement and Rock Fill Quantity Estimates, September 14, 2010.

Ministry of Transportation Ontario, Embankment Settlement Criteria for Design, Final Draft, March 2, 2010.

Ontario Provincial Standard Drawings

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 203.020	Embankments Over Swamp, Existing Slope Excavated to 1H:1V
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario

Ontario Provincial Standard Specifications

OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1004	Material Specification for Aggregates – Miscellaneous

Ontario Water Resources Act

Ontario Regulation 903/90	Wells; O.Reg 468/10 Amendment to Ontario Regulation 903

Table 1: Comparison of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel H-Piles (Both Abutments)	1	■ Straightforward construction. ■ Pile cap construction above river water level. ■ Same type of foundations at both abutments.	■ Depending on the elevation of the underside of pile cap at the west abutment, socketing of piles into strong bedrock will be required to achieve the minimum pile length for structural design. ■ About 2.5 times greater pile lengths at east abutment than at west abutment.	■ Relative costs lower than for caissons. ■ Cost for socketing piles into strong bedrock at west abutment.	■ Need to achieve minimum required pile length by socketing into strong bedrock.
Driven Steel Tube Piles	2	■ Straightforward construction. ■ Pile cap construction above river water level.	■ Depending on the elevation of the underside of pile cap at the west abutment, socketing of piles into strong bedrock will be required to achieve 5 m pile length. ■ About 2.5 times greater fill lengths at east abutment than at west abutment	■ Relative costs lower than caissons. ■ Cost for socketing of piles into strong bedrock at west abutment.	■ Need to achieve minimum required pile length by socketing into strong bedrock.
Strip or Spread Footing Founded on Bedrock (west abutment) and Deep Foundation (east abutment)	3	■ Straightforward construction.	■ Sub-excavation of overburden at least 3.8 m below existing ground surface and 0.7 m below the groundwater/river water level (July 2012) required at west abutment. ■ Unwatering, cleaning bedrock surface and concrete placement required within dry, shored excavation. ■ Soil conditions at east abutment not suitable for shallow foundation – different foundation types between the two abutments may not be compatible/suitable.	■ Additional costs required for installation of temporary shoring and unwatering.	■ If bedrock is higher along the west foundation than anticipated, bedrock removal may be required for leveling. ■ Variability in bedrock surface will impact mass concrete quantities and/or excavation depths and installation of temporary shoring. ■ Fractures in bedrock may cause seepage into excavation from adjacent river. ■ Shored excavation could negatively impact existing structure.

Table 1: Comparison of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons	4	■ Higher axial resistance compared to steel H-piles or tube piles. ■ Possible elimination of pile cap and associated excavation.	■ Requires rock drilling/large socket for seating caissons into bedrock. ■ Potential for difficulty associated with seating a larger diameter caisson into strong. ■ Require temporary or permanent liners to advance caissons at east abutment.	■ Relative costs much higher than for steel H-piles, although fewer foundation units are required.	■ Likely able to reach the required termination depth into bedrock. ■ Potential for construction problems associated with artesian groundwater inflow into caisson during installation due to proximity to the river – may have to use tremie concrete construction methods. ■ May also need liner to advance caissons at west abutment.

Prepared by: AB
Reviewed bv: JMAC

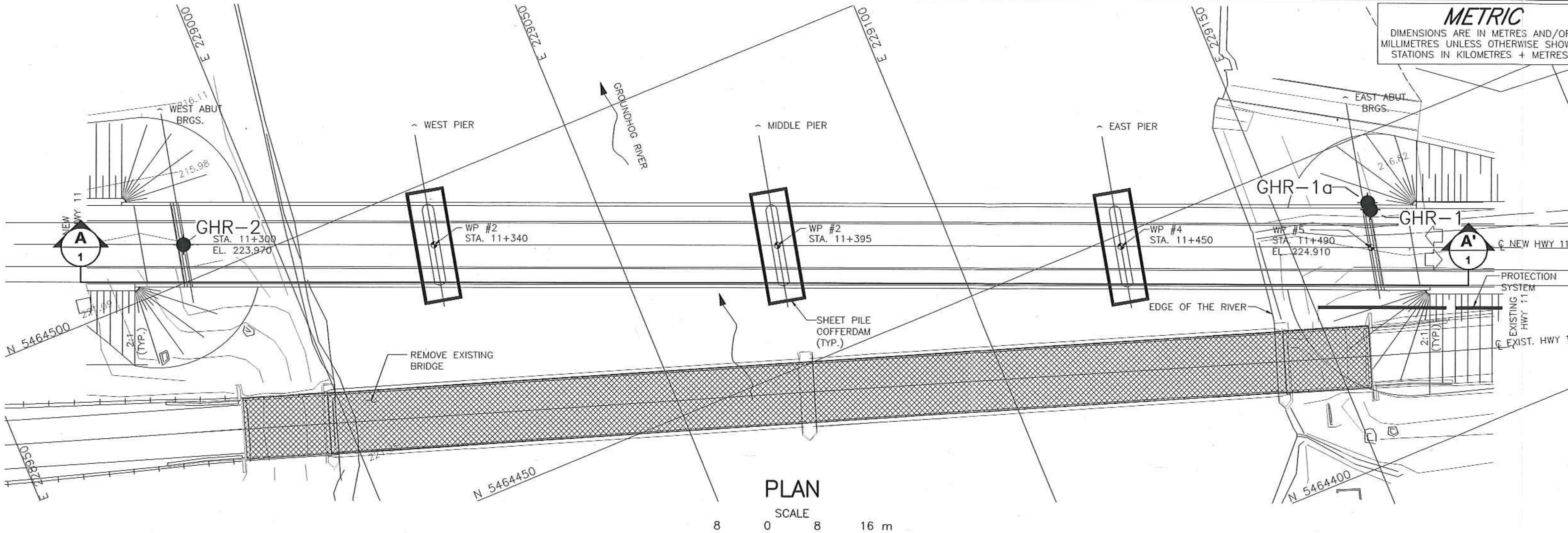
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5049-07-00

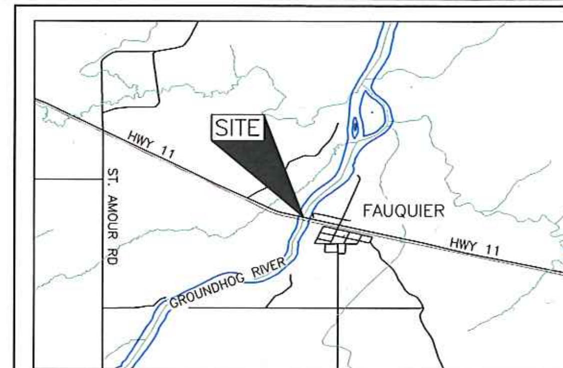
SHEET

HIGHWAY 11
Groundhog River Bridge
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



PLAN
SCALE
8 0 8 16 m



KEY PLAN
SCALE
1 0 1 km

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- R Refusal
- WL in piezometer, measured on AUG 1, 2012
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
GHR-1	216.8	5464438.5	229164.2
GHR-1a	216.8	5464439.7	229164.2
GHR-2	216.1	5464505.7	228986.5

NOTES

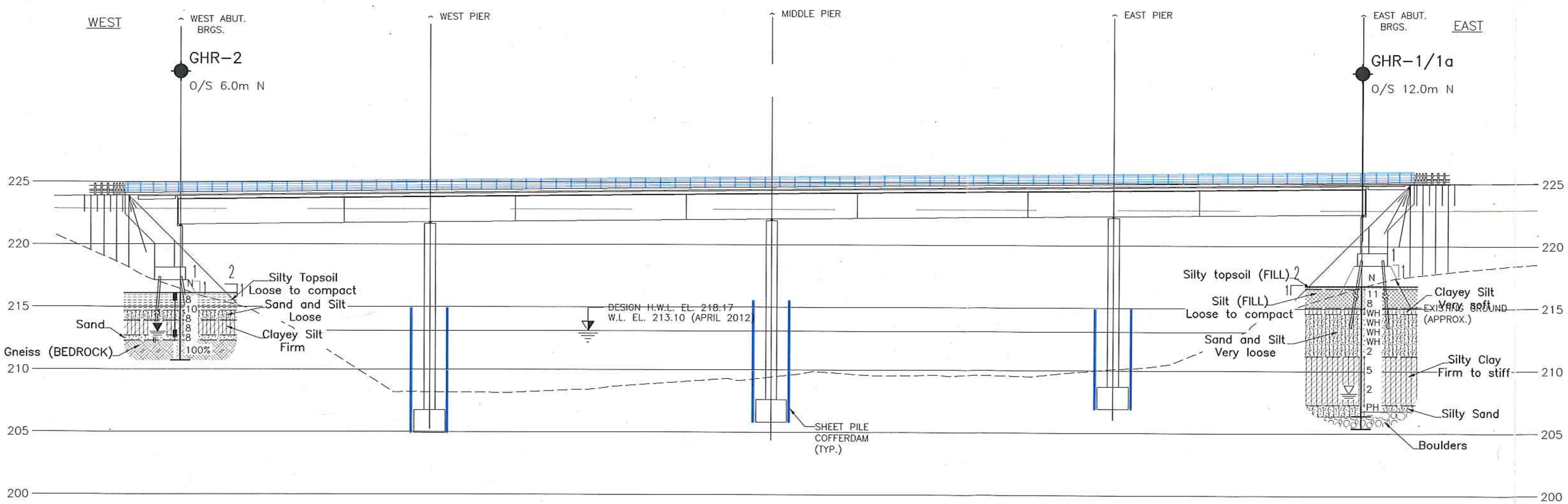
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. GA_Groundhog River_3B.dwg received APRIL 12, 2013.



A-A' 1 CENTRELINE PROFILE
HIGHWAY 11
HORIZONTAL SCALE
8 0 8 16 m
4 0 4 8 m
VERTICAL SCALE

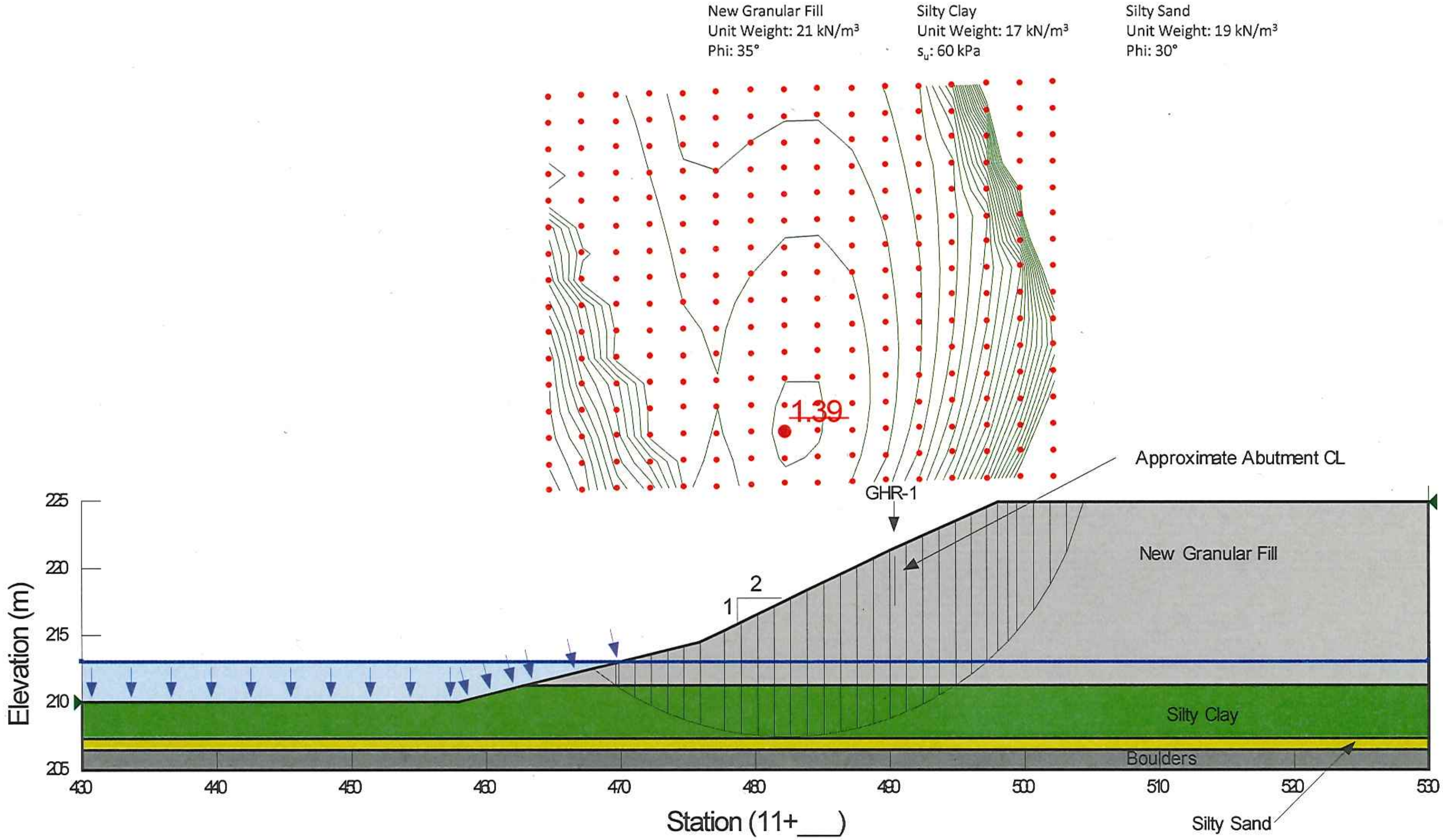
A.J.K. BOM 100075715 MAY 06, 2013 LICENSED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

J.M.A. COSTA MAY 06, 2013 LICENSED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

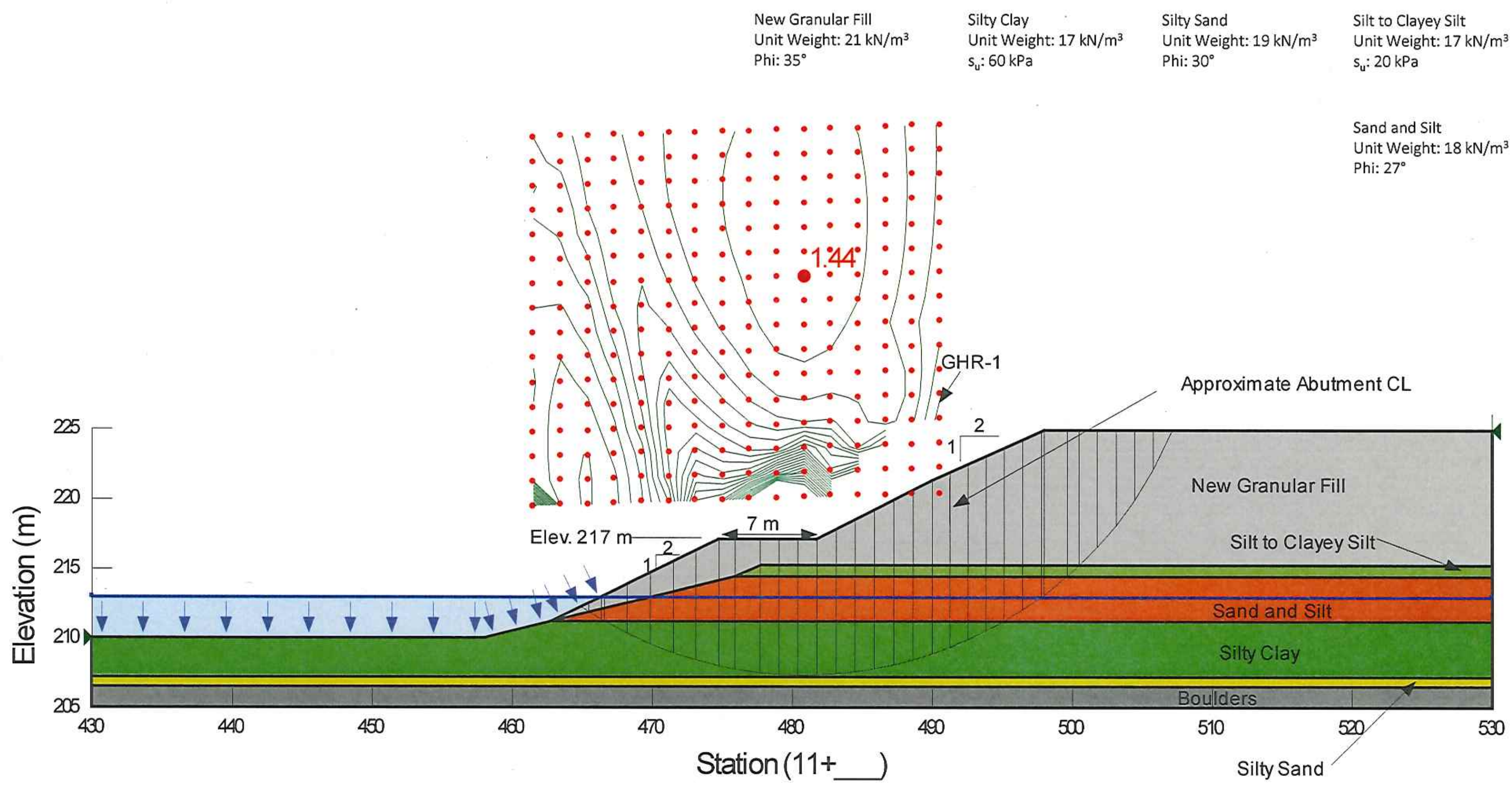
NO.	DATE	BY	REVISION

Geocres No. 42-G40

HWY. 11	PROJECT NO. 11-1191-0025		DIST.
SUBM'D. AC	CHKD. AB	DATE: MAY 2013	SITE: 39W-093
DRAWN: JJL	CHKD.	APPD. JMAC	DWG. 1



PROJECT	Highway 11 Groundhog River Bridge			
TITLE	**Stability Analysis – Front Slope of East Approach Embankment, Full Sub-Excavation**			
Golder Associates	PROJECT No. 11-1191-0025			FILE No. ----
	DESIGN	AC	May 2013	SCALE AS SHOWN REV.
	CADD	--		**Figure 1**
	CHECK	AB	May 2013	
	REVIEW	JMAC	May 2013	



PROJECT

Highway 11 Groundhog River Bridge

TITLE

Stability Analysis – Front Slope of East Approach Embankment, Toe Berm

Golder Associates

PROJECT No. 11-1191-0025			FILE No. ----
DESIGN	AC	May 2013	SCALE AS SHOWN / REV.
CADD	--		
CHECK	AB	May 2013	**Figure 2**
REVIEW	JMAC	May 2013	

APPENDIX A

Record of Boreholes and Drillholes

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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 multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	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_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS 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 and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 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	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 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 varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

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

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

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Golder Associates

RECORD OF BOREHOLE No GHR-1

 METRIC

PROJECT 11-1191-0025
W.P. 5049-07-00
LOCATION N 5464438.5; E 229164.2
ORIGINATED BY EHS
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. HOLLOW STEM AUGERS
COMPILED BY AC
DATUM GEODETIC
DATE JULY 25, 2012
CHECKED BY AB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ FIELD VANE, sensitivity)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.8	GROUND SURFACE									
0.0	Silty topsoil (FILL)									
0.2	Silt, some sand, trace gravel, trace organics (FILL) Loose to compact Brown Moist to wet	1	SS	11						
		2	SS	8		216				
215.1 / 1.7	CLAYEY SILT, some sand, trace organics Very soft Brown Wet	3	SS	WH		215		H○	OC=0.5%	
214.6 / 2.2	SAND and SILT, some clay, trace organics, clay seams / layers Very loose Brown to grey Wet	4	SS	WH		214				
		5	SS	WH		213				
		6	SS	WH				○		0 53 34 13
		7	SS	2		212				
211.2 / 5.6	SILTY CLAY, trace sand Firm to stiff Brown to grey Wet					211				
		8	SS	5		210		├─○┤		
							+ 4			
		9	SS	2		209		├─○┤		
					∇	208	+ 6			
207.3 / 9.5	Silty SAND, some gravel, trace clay Grey Wet	10	TO	PH						
		11	AS	-		207				
206.4 / 10.4	END OF BOREHOLE AUGER REFUSAL									

Note:

1. Water level at a depth of 8.6 m below ground surface (Elev. 208.2 m) upon completion of drilling.
2. On December 13, 2012, Borehole GHR-1a advanced 1.2 m north of Borehole GHR-1.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60

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

Golder Associates

RECORD OF BOREHOLE No GHR-1a

 METRIC

PROJECT 11-1191-0025
W.P. 5049-07-00
LOCATION N 5464439.7; E 229164.2
ORIGINATED BY ID
DIST HWY 11
BOREHOLE TYPE HOLLOW STEM AUGERS, NW CASING, NQ CORING
COMPILED BY MT
DATUM GEODETIC
DATE December 13, 2012
CHECKED BY AB

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.8	GROUND SURFACE										
0.0	For stratigraphy details refer to Record of Borehole GHR-1.						216, 215, 214, 213, 212, 211, 210, 209, 208, 207				
206.8 / 10.0	BOULDERS, gravel seams		1	RC	REC 100%		206				
			2	RC	REC 100%						
205.4 / 11.4	END OF BOREHOLE										

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No GHR-2

 METRIC

PROJECT 11-1191-0025			
W.P. 5049-07-00	LOCATION N 5464505.7; E 228986.5		ORIGINATED BY EHS
DIST	HWY 11	BOREHOLE TYPE 108 mm I.D. HOLLOW STEM AUGERS	COMPILED BY AC
DATUM GEODETIC	DATE JULY 25 and 26, 2012		CHECKED BY AB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.1	GROUND SURFACE						
0.0	Silty TOPSOIL, trace sand, roots / rootlets Loose to compact Brown Moist	1	SS	8	216		
		2	SS	10	215		
214.7 / 1.4	SAND and SILT, some clay Loose Brown Moist	3	SS	8			1 35 47 17
213.9 / 2.2	CLAYEY SILT, trace sand, trace gravel Firm Brown Moist to wet	4	SS	8	214		
212.7	SAND, trace silt Brown Wet	5	SS	8	213		
212.3 / 3.8	GNEISS (BEDROCK) Bedrock cored from 3.8 m depth to 5.4 m depth. For coring details see Record of Drillhole GHR-2.	1	RC	REC 100%	212 211		RQD = 100%
210.7 / 5.4	END OF BOREHOLE						

Note:

1. Water level at a depth of 2.4 m below ground surface (Elev. 213.7 m) upon completion of drilling.

2. Water level in piezometer at a depth of 3.1 m (Elev. 213.0 m) on August 1, 2012.

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

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

PROJECT: 11-1191-0025

RECORD OF DRILLHOLE: GHR-2

SHEET 1 OF 1

LOCATION: N 5464505.7 ;E 228986.5

DRILLING DATE: JULY 25 and 26, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME-850

DRILLING CONTRACTOR: Landcore Drilling

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN (FLUSH)	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA: B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		212.3																
4–5	NW; NQ Coring, JULY 26, 2012	GNEISS Very coarse grained Fresh Strong Grey		3.8	1	GREY 100%							JIR							UCS=97 MPa
				210.7 5.4																
		END OF DRILLHOLE																		

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

SUD-RCK 11-1191-0025.GPJ GAL-MISS.GDT 03/05/13 DATA INPUT:

DEPTH SCALE

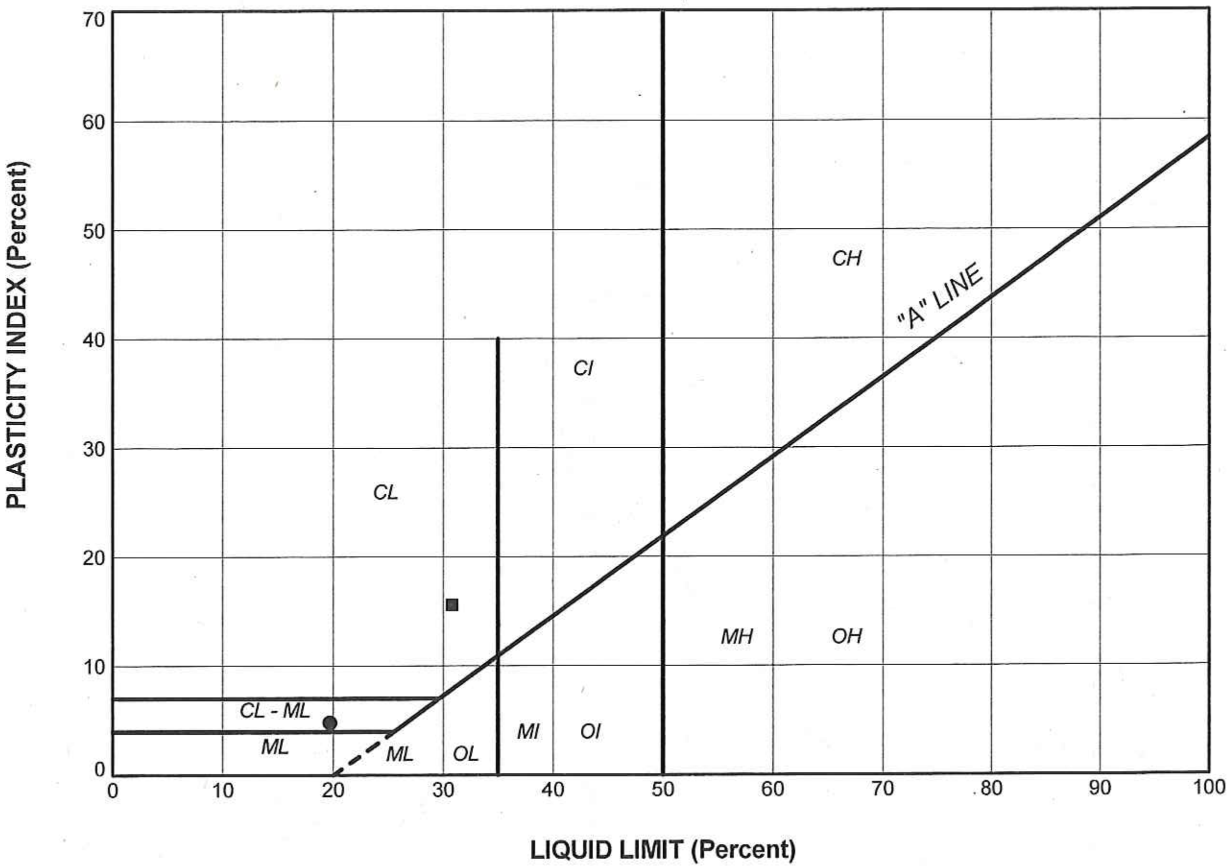
1 : 50

Golder Associates

LOGGED: EHS

CHECKED: AB

APPENDIX B
Laboratory Test Results



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	GHR-1	3	20	15	5
■	GHR-2	4	31	15	16

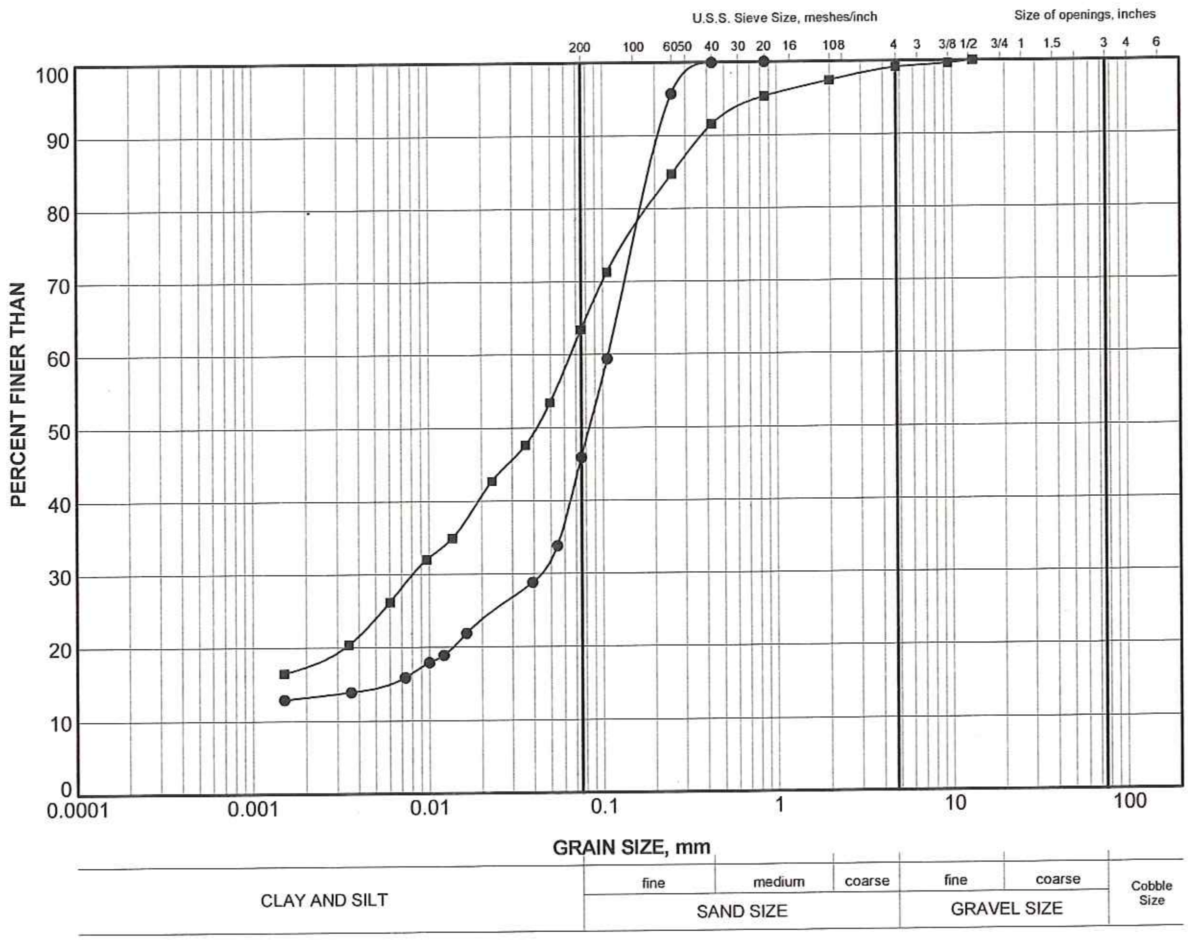
PROJECT
HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE
PLASTICITY CHART
CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2012	SCALE N/A	REV.
CHECK	AB	Nov 2012	FIGURE B1	
APPR	JMAC	Nov 2012		

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GHR-1	6	212.7
■	GHR-2	3	214.3

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

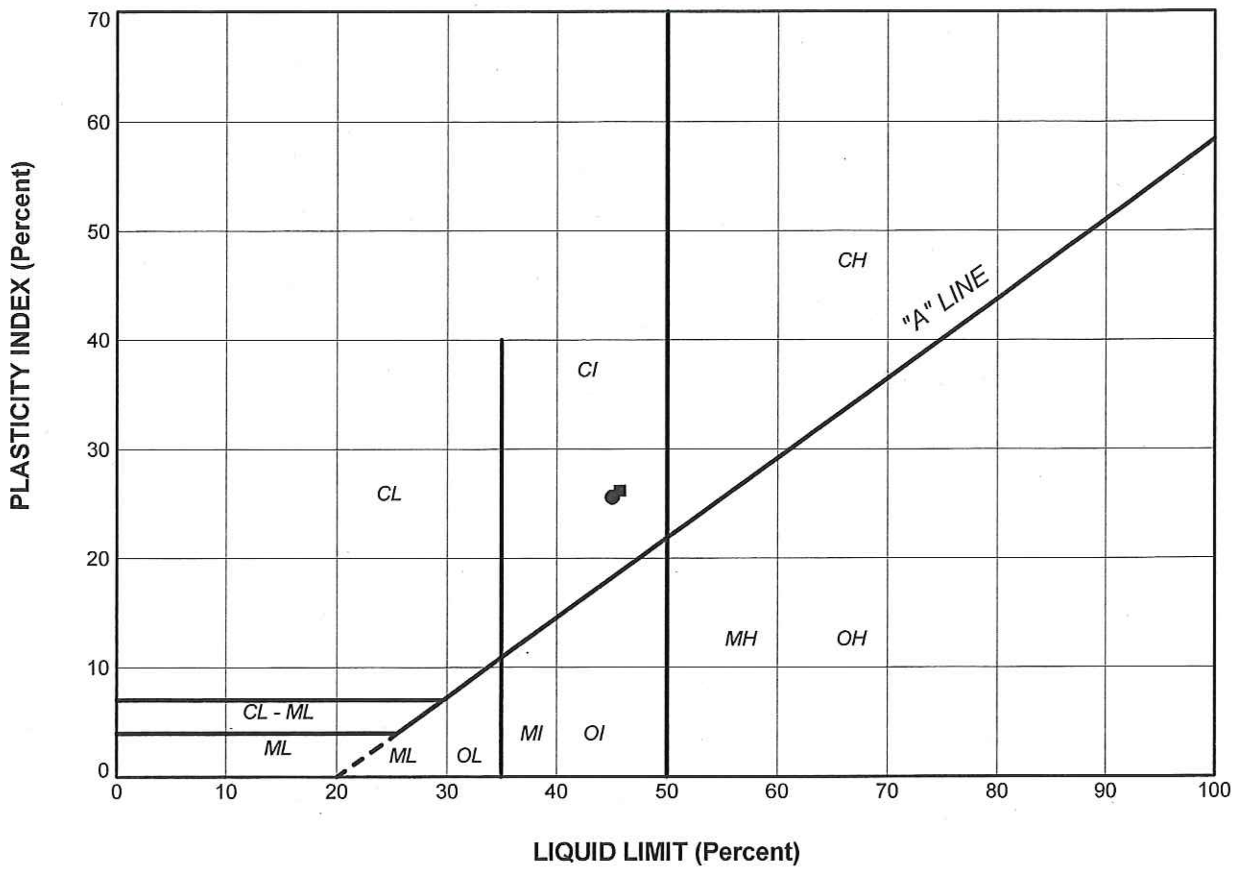
GRAIN SIZE DISTRIBUTION

SAND AND SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		11-1191-0025	FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2012	SCALE	N/A REV.
CHECK	AB	Nov 2012		
APPR	JMAC	Nov 2012	FIGURE B2	

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	GHR-1	8	45	19	26
■	GHR-1	9	46	20	26

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

PLASTICITY CHART

SILTY CLAY

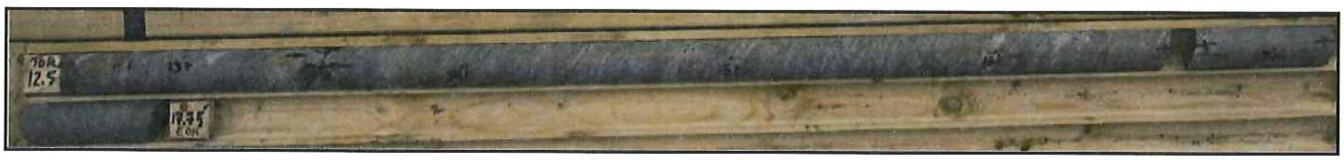
PROJECT No.		11-1191-0025	FILE No.	11-1191-0025.GPJ	
DRAWN	JJL	Nov 2012	SCALE	N/A	REV.
CHECK	AB	Nov 2012	**FIGURE B3**		
APPR	JMAC	Nov 2012			

Golder Associates
SUDBURY, ONTARIO

SUD-MTO PL (NEW) GLDR_LDN.GDT

Borehole GHR-1
Elevation 206.8 m to 205.4 m

Borehole GHR-1a
Elevation 212.3 m to 210.7 m



0 m 0.12 m 0.24 m 0.37 m 0.49 m 0.61 m

PROJECT	HIGHWAY 11 GROUNDHOG RIVER BRIDGE			
TITLE	**CORE PHOTOGRAPHS**			
Golder Associates	PROJECT No.	11-1191-0025		FILE No. ----
	DESIGN	AC	NOV 2012	SCALE AS SHOWN REV.
	CADD	-		
	CHECK	AB	April 2013	**FIGURE B4**
	REVIEW	JMAC	April 2013	