March 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

Preliminary Design
Amherst Island Ferry
Docks Conversion Study
Millhaven, Ontario
G.W.P. 4067-09-00

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT AMHERST ISLAND FERRY DOCKS CONVERSION STUDY G.W.P. 4067-09-00

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 in support of the preliminary design for the expansion of the existing Millhaven and Stella ferry terminals in Loyalist Township, Lennox and Addington County, Ontario.

This report addresses the results of the foundation investigation carried out for the proposed expansion/reconstruction of the ferry terminals.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated July 2011 and associated clarifications, and in Section 5.8 of URS's *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The Amherst Island ferry travels across the northeastern end of Lake Ontario between the Stella Terminal (or Amherst Island) and the Millhaven Terminal (or the mainland) located about 25 km west of Kingston, Ontario. The proposed configuration of the expanded ferry terminals are shown in plan on Drawings 1 and 3, based on the preliminary design drawings dated September 4, 2013 provided to Golder by URS.

The proposed new Amherst Island ferry terminals are to be located at the same sites as the existing terminals in Millhaven and Stella on Highway 33 and Forty Foot Road, respectively, in the general area indicated on the key plan on Drawings 1 and 3.

The elevation of the water surface of Lake Ontario ranged between about 74.2 m and 74.4 m in September 2012 when a portion of the over-water boreholes were drilled, and between about 74.9 m and 75.1 m in August 2013 when the remaining over-water boreholes were drilled. The contour lines shown on Drawings 1 and 3 represent the approximate depth from water surface to lake bottom based on a chart datum for Lake Ontario of 74.2 m as reported by Canadian Hydrographic Services, Fisheries and Oceans, Canada.

Based on the preliminary design drawings of the proposed expanded ferry terminals provided by URS, we understand that the existing on shore structures will be removed and that expanded docks, new ramps, sewage holding tanks and mechanical, storage, office and washroom facilities will be constructed. It is also understood that bulkhead walls will be constructed along the shore line as shown on Drawings 1 and 3.

In general, the terrain in the area of the proposed expanded ferry terminals is relatively flat, with the natural ground surface in the vicinity of the Millhaven terminal between about Elevation 75 m and 77 m and between about Elevation 75 m and 76 m in the vicinity of the Stella terminal.

3.0 INVESTIGATION PROCEDURES

Following the identification of the preferred ferry terminal layout alternatives to be carried forward to preliminary design, Golder and URS met with MTO Foundations on July 9, 2012 to agree on the locations for the proposed in-water foundation boreholes. As per the terms of reference for this project, four (4) in-water borehole locations at each terminal were selected for the preliminary foundation investigation. The field work for the in-water investigation was carried out in two stages.

The first stage took place between September 12 and 27, 2012 at which time six (6) boreholes (Boreholes 12-01, 12-02, 12-02A, 12-04, 12-07 and 12-08) and one (1) dynamic cone penetration test (DCPT 12-03) were advanced at the locations shown on Drawings 1 and 3. The barge equipment employed during the first stage was comprised of a modular floating raft and tug boat supplied and operated by ODS Marine Construction, of Greely, Ontario. On September 27, 2012 after numerous days of delay and stand-by due to inclement weather, scheduling issues with the existing ferry operations and deeper than anticipated water conditions, the DFO window for in-water work closed for the fall/spring spawning season. As a result of the in-water work restriction as well as the deteriorating weather and marine conditions (which were posing health and safety risks to the workers) the field crews were demobilized from the site.

The second and remaining stage of the in-water investigation was carried out between August 12 and 15, 2013 at which time four (4) boreholes (Boreholes 13-02, 13-03, 13-05/13-05A and 13-06) were advanced at the locations shown on Drawings 1 and 3. The barge equipment employed during the second stage was comprised of a triangular “Jack-up 50” barge approximately 17 m long by 20 m wide and an “Ecosse” tug boat supplied and operated by McKeil Marine Ltd. of Hamilton, Ontario. The larger barge equipment utilized for the second stage of the in-water investigation could more readily work in deeper water with less interference with the existing ferry operations and the work was started earlier in the summer when the marine and weather conditions were more favourable.

Photographs showing the equipment used for both stages of the in-water borehole investigation are included in Appendix E.

The on-shore investigation was carried out between September 22 and 24, 2013 at which time four (4) boreholes (Boreholes 13-09, 13-10, 13-11 and 13-12) were advanced at the locations shown on Drawings 1 and 3.

Following completion of all of the field investigation, the design team, in consultation with Loyalist Township and MTO, made minor modifications to the layout of the proposed Millhaven Terminal resulting in some of the boreholes no longer being located within the footprint of the proposed Ferry Terminal expansion at this location.

The drilling investigation was carried out using a combination of CME 55 (first stage over water) and CME-75 (second stage over water) drill rigs (working from the barges) as well as a track-mounted CME 55 drill rig (for the on-land boreholes), supplied and operated by Marathon Drilling Co. Ltd. of Greely, Ontario and Canadian Soil Drilling of Springwater, Ontario. The in-water boreholes were advanced using wash rotary methods and NW-casing contained inside a 100 mm diameter outer casing or HW-casing contained inside a 140 mm diameter outer casing. The outer protective casing extended through the lake water column and was driven into the bottom of the lakebed, in accordance with Golder’s Environmental Protection Plan, dated May 11, 2012, in order to minimize sediments and drill flush water being washed into the lake. The on-shore boreholes were advanced through the overburden using 108 mm inside diameter hollow-stem augers. Soil samples were obtained at depth intervals typically ranging from about 0.75 m to 1.5 m. All soil sampling was performed, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Samples of the bedrock were obtained using an ‘NQ’ or ‘HQ’ size rock core barrel.

The field work was supervised throughout by members of our engineering and technical staff, who confirmed the investigated locations, arranged for the clearance of underground services, supervised the drilling, sampling and in situ testing operations, logged the boreholes and DCPT, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labeled and transported to our Mississauga or Ottawa geotechnical laboratories where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing such as water content, Atterberg limits, organic contents and grain size distribution tests were carried out on selected samples of the overburden soils. Point load index testing and unconfined compression testing was carried out on specimens of the recovered rock core.

The groundwater conditions in the on-land boreholes were observed in the open boreholes during and immediately following the overburden drilling operations. All boreholes were backfilled with cement grout or bentonite upon completion, in accordance with Ontario Regulation 903 (as amended) and Golder’s Environmental Protection Plan.

All of the in-water boreholes were laid out in the field by Hopkins, Cormier and Chitty, a registered Ontario land surveyor in Kingston, Ontario. The elevation of the lake water surface, to which the depths on the in-water Record of Boreholes are referred, was either measured by the surveyor at the start of the borehole drilling or determined by level survey in reference to a temporary benchmark located on each existing ferry dock, (supplied by the surveyor) at the start and completion of drilling. The on-land boreholes were located in the field relative to fixed existing features and the elevations were determined by level survey in reference to temporary benchmarks located on each existing ferry dock, as supplied by the surveyor.

The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized below and are shown on Drawings 1 and 3.

Borehole/DCPT Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground/Water Surface Elevation (m)	Borehole Depth (m)
BH 12-01	4894798.8	285574.5	74.4	9.2
BH 12-02	4894766.6	285562.3	74.4	11.1
BH 12-02A	4894778.6	285562.2	74.3	8.8
DCPT 12-03	4894703.5	285574.3	74.3	11.3
BH 12-04	4894765.9	285536.0	74.4	7.7
BH 13-02	4894766.6	285561.8	75.1	15.2
BH 13-03	4894704.9	285572.0	75.1	15.7
BH 13-09	4894833.5	285551.9	76.3	9.5
BH 13-10	4894777.0	28551.7	77.8	5.9
BH 12-07	4892480.4	288556.7	74.2	8.5
BH 12-08	4892485.0	288581.7	74.4	7.7
BH13-05/13-05A	4892500.9	288505.2	75.0	22.5
BH 13-06	4892502.8	288538.4	74.9	16.3
BH 13-11	4892471.8	288553.2	76.4	9.9
BH 13-12	4892412.9	288585.6	76.4	4.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located in the southern portion of the physiographic region known as the Napanee Plain, as delineated in The Physiography of Southern Ontario[1]. The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[2] indicates that the bedrock within the Napanee Plain consists of grey limestone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay. Well records indicate that the average depth to bedrock within the Napanee Plain is approximately 2 m. However, in many areas, bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the southern portion of the Napanee Plain, and within and adjacent to river valleys throughout the Plain.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the investigation and the results of in situ and laboratory testing are shown on the Record of Boreholes and Drillholes in Appendix A (Millhaven) Appendix and C (Stella).

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic sections on Drawings 2 and 4 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions and top of bedrock will vary between and beyond the borehole locations.

In general, the subsurface strata encountered at the Millhaven site consist of fill and/or organics and/or silts, sands and gravels over limestone bedrock. The fills are variable in composition ranging from sand and gravel to crushed concrete to silty clay and contain varying amounts of shell fragments, organics, wood fragments and timber cribbing. At the Stella site, the subsurface strata encountered are similar, generally consisting of fill and/or organics and/or sands and gravels over limestone bedrock. The fills are variable in composition ranging from sand and gravel to clayey silt and contain concrete fragments and rootlets and timber cribbing. At both the Millhaven and Stella sites, the overburden thickness is variable ranging from as little as 0 m or less than 0.3 m up to about 6.5 m thick at the investigated locations.

A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

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[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*. Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

[2] Map 2544, Ministry of Northern Development and Mines, 1991.

4.2.1 Millhaven – In-Water Boreholes (Boreholes 12-01, 12-02, 12-02A, 12-04, 13-02, 13-03 and DCPT 12-03)

4.2.1.1 Lake Water

The water level in Lake Ontario at the site ranged from about Elevations 74.3 m to 74.4 m during the 2012 investigation, and was at about Elevation 75.1 m during the 2013 investigation. The depth to the lakebed varied from about 3.2 m to 8.2 m at the borehole and DCPT locations (Elevations 71.1 m and 66.9 m, respectively).

The water level elevation was surveyed to a temporary benchmark set up on each existing ferry dock at the start and end of the drilling of each borehole and the depths shown on the Records of Boreholes and Drillholes are referenced to the water surface.

4.2.1.2 Sand and Gravel to Sandy Silt and Gravel to Crushed Concrete Fill

A deposit of sand and gravel to sandy silt and gravel to crushed concrete fill was encountered at lake bottom in Boreholes 12-02 and 12-02A at depths between about 3.2 m and 4.9 m below water surface (Elevations 69.5 m to 71.1 m). This deposit was found to be about 2.4 m thick in Borehole 12-02A and greater than 6.2 m thick in Borehole 12-02 and was not fully penetrated to a depth of 11.1 m below water surface (Elevation 63.3 m) at which depth the borehole was terminated due to rough water conditions that made working conditions unsafe to continue. However, the bottom of the overburden/top of bedrock was encountered at a depth of 11.7 m (Elevation 63.4 m) in the immediately adjacent Borehole 13-02.

The measured SPT ‘N’ values in the sand and gravel to sandy silt and gravel fill are between 0 blows (weight of hammer) and 27 blows per 0.3 m of penetration, indicating that this deposit has a very loose to compact relative density. It was not possible to carry out SPTs in the crushed concrete fill in Borehole 12-02A and coring was required to advance the borehole through the fill at this location.

The fill varies in composition from sand and gravel trace to some silt, trace clay to sandy silt and gravel to crushed concrete, containing organics and wood fragments. The results of grain size distribution tests completed on two selected samples of the sand and gravel fill are shown on Figure B1 in Appendix B.

The measured water content of three samples of the fill ranges between about 1 per cent and 60 per cent. The organic content measured on one sample of the fill is about 12 per cent.

4.2.1.3 Clayey Organic Silt

A deposit of clayey organic silt with sand containing wood fragments and shells was encountered at lake bottom in Borehole 13-03 at a depth of about 8.2 m below water surface (Elevation 66.9 m) and was measured to be about 3.4 m thick.

A single measured SPT ‘N’ value in the organic silt deposit is 0 blows (weight of rods) per 0.3 m of penetration, indicating that this deposit has a very soft relative density.

The results of a grain size distribution test completed on one selected sample of the organic deposit is shown on Figure B2 in Appendix B.

The measured water content of one sample from this deposit is about 198 per cent. The measured organic content of a sample of this deposit is 18 per cent.

4.2.1.4 Sand and Gravel

A deposit of sand and gravel was encountered at lake bottom in Borehole 12-01 at a depth of about 4.3 m below water surface (Elevation 70.1 m) and a deposit of sand and gravel till was encountered underlying the organic deposit in Borehole 13-03 at a depth of about 11.6 m below surface (Elevation 63.5 m). The sand and gravel and sand and gravel till deposits were measured to be about 1.4 m and 0.3 m thick, respectively.

A single measured SPT ‘N’ value in the sand and gravel deposit in Borehole 12-01 is 2 blows per 0.3 m of penetration, indicating that this deposit has a very loose relative density.

The deposit is comprised of sand and gravel, some silt, trace clay to sand and gravel till, and contains organics and shell fragments in Borehole 12-01. The results of a grain size distribution test completed on one selected sample of the deposit from Borehole 12-01 is shown on Figure B3 in Appendix B.

The measured water content of one sample from near the surface of this deposit is about 44 per cent.

4.2.1.5 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes 12-01, 12-02A, 12-04, 13-02 and 13-03 at depths starting between about 3.6 m and 11.9 m below water surface (Elevations 70.8 m to 63.2 m, respectively). Refusal to further penetration of DCPT 12-03 on probable bedrock was encountered at about 11.3 m below water surface (Elevation 63.0 m). Bedrock was not encountered in Borehole 12-02 to a depth of 11.1 m below water surface (Elevation 63.3 m).

Based on the recovered bedrock core samples, the bedrock at the Millhaven site consists of limestone. In general, the bedrock samples are described as fine grained, laminated to thickly bedded, slightly porous, slightly weathered to fresh, grey Limestone with shale interbeds and laminations and clay infilling within joints at some locations. The Rock Quality Designation (RQD) measured on the core samples ranges from about 0 per cent to 93 per cent, but is generally between about 60 per cent and 80 per cent, indicating a rock mass that in general is of fair to good quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 15 per cent and 100 per cent.

Unconfined Compressive Strength (UCS) tests carried out on seven samples of the Limestone bedrock from the in-water boreholes at the Millhaven site measured compressive strengths between about 9 MPa and 106 MPa. The test results which are shown on the Record of Drillhole sheets and summarised on Tables B1 to B7 in Appendix B, indicate that the bedrock is weak (R2) to very strong (R5) (but generally medium strong to strong) as per Table 3.5 of CFEM (2006) reproduced here in Table B9 in Appendix B.

Point load index tests were performed on seventeen selected samples of the rock core recovered from the in-water boreholes at the Millhaven terminal. Point load strength index values are shown on the Record of Drillhole Sheets and on Table B10 in Appendix B. The point load index (Is_{50}) results from the laboratory tests carried out on the samples of the Limestone bedrock range from approximately 2.9 MPa to 7.4 MPa. These index values correspond to UCS values ranging between about 40 MPa and 100 MPa, based on a relationship between Is_{50} and UCS which is given by a correlation factor (k), estimated to be equal to 14 for this site, and calculated as the ratio of the average laboratory UCS and average corresponding point load test index value from all of the drillholes at the Millhaven terminal. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.

Based on the laboratory UCS tests and point load testing results (refer to Table B9 in Appendix B for details on the field estimation of rock hardness and R0, R1, etc. values outlined below), the estimated intact strength of the Limestone bedrock generally ranges from medium strong (R3, 25 MPa < UCS < 50 MPa) to strong (R4, 50 MPa < UCS < 100 MPa); (CFEM, 2006).

4.2.2 Millhaven – On-Shore Boreholes (Boreholes 13-09 and 13-10)

4.2.2.1 Silty Clay Fill

A fill deposit consisting of gravelly silty clay with sand was encountered at ground surface in Borehole 13-09 (Elevation 77.8 m). The bottom of the fill deposit was encountered at a depth of 0.7 m below ground surface, corresponding to Elevation 77.1 m.

One Standard Penetration Test (SPT) "N" value measured within the cohesive fill was 23 blows per 0.3 m of penetration, suggesting a very stiff consistency.

The results of a grain size distribution test carried out on one sample of this fill deposit are shown on Figure B4 in Appendix B.

Atterberg limits testing was carried out on one sample of this cohesive fill and measured a liquid limit of 39 per cent, a plastic limit of 21 per cent and a plasticity index of 18 per cent. These test results, which are plotted on a plasticity chart on Figure B5 in Appendix B, confirm that the cohesive fill material consists of silty clay of medium plasticity.

The measured water content of one sample from this deposit is about 11 per cent.

4.2.2.2 Sandy Gravel Fill

A fill deposit consisting of sandy gravel was encountered at ground surface in Borehole 13-10 and underlying the cohesive fill at a depth of 0.7 m below ground surface (Elevation 77.1 m) in Borehole 13-09. The bottom of the fill deposit was encountered at a depth of 1.6 m to 2.9 m below ground surface (Elevation 76.2 m and 73.4 m, respectively) and the fill deposit was measured to be between 0.9 m and 2.9 m thick.

Standard Penetration Test (SPT) "N" values measured within this fill deposit were between 9 and 48 blows per 0.3 m of penetration, indicating a loose to dense relative density.

The results of grain size distribution tests carried out on two samples of this fill deposit are shown on Figure B6 in Appendix B.

The measured water content of three samples from this deposit are between about 4 per cent and 9 per cent.

4.2.2.3 Wood (Existing Cribwork)

Existing wooden cribwork from the original ferry dock structure was encountered at a depth of about 2.9 m below ground surface (Elevation 73.4 m) in Borehole 13-10 and was found to be about 1.5 m thick.

4.2.2.4 Organic Silty Sand

A deposit of dark grey organic silty sand, trace clay was encountered underlying the wooden cribwork at a depth of about 4.4 m below ground surface (Elevation 71.9 m) in Borehole 13-10. The bottom of the organic silty sand deposit was encountered at a depth of 5.6 m below ground surface (Elevation 70.7 m) and the deposit was measured to be about 1.2 m thick.

One Standard Penetration Test (SPT) "N" value measured within this organic silty sand deposit was 1 blow per 0.3 m of penetration, indicating a very loose relative density.

The measured water content of one sample from this deposit is 58 per cent.

4.2.2.5 Sand and Gravel

A deposit of sand and gravel, trace to some silt, trace clay was encountered underlying the organic silty sand at a depth of about 5.6 m below ground surface (Elevation 70.7 m) in Borehole 13-10. The bottom of this deposit was encountered at a depth of about 6.3 m below ground surface (Elevation 70.0 m) and the deposit was measured to be 0.7 m thick.

The results of a grain size distribution test carried out on one sample of this deposit is shown on Figure B7 in Appendix B.

The measured water content of one sample from this deposit is 11 per cent.

4.2.2.6 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes 13-09 and 13-10 at depths of about 1.6 m and 6.3 m below ground surface, respectively, corresponding to Elevation 76.2 m and 70.0 m.

Based on the recovered bedrock core samples, the bedrock at this location consists of Limestone inter-bedded with Shale. In general, the bedrock samples are described as fine grained, laminated, slightly porous, slightly weathered to fresh, grey Limestone with shale interbeds. The Rock Quality Designation (RQD) measured on the core samples ranges from about 0 per cent to 92 per cent, but is generally less than about 50 per cent in the upper 3 m below the bedrock surface, indicating a rock mass of very poor to excellent (but generally very poor to poor near the bedrock surface) quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 60 per cent and 100 per cent.

Unconfined Compressive Strength (UCS) tests carried out on one sample of the Limestone bedrock from Borehole 13-09 measured a compressive strength of about 71 MPa. The test result which is shown on the Record of Drillhole sheet and summarised on Table B8 in Appendix B, indicate that the bedrock is strong (R4) as per Table 3.5 of CFEM (2006) reproduced here in Table B9 in Appendix B.

Point load index tests were performed on six selected samples of the rock core recovered from the on-shore boreholes at the Millhaven terminal. Point load strength index values are shown on the Record of Drillhole Sheets and on Table B10 in Appendix B. The point load index (Is_{50}) results from the laboratory tests carried out on the samples of the Limestone bedrock range from approximately 2.3 MPa to 5.7 MPa. These index values correspond to UCS values ranging between about 30 MPa and 80 MPa, based on a relationship between Is_{50} and UCS which is given by a correlation factor (k), estimated to be equal to 14 for this site, and calculated as the ratio of the average laboratory UCS and average corresponding point load test index value from all of the drillholes at the Millhaven terminal. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.

Based on the laboratory UCS tests and point load testing results (refer to Table B9 in Appendix B for details on the field estimation of rock hardness and R0, R1, etc. values outlined below), the estimated intact strength of the Limestone bedrock generally ranges from medium strong (R3, 25 MPa < UCS < 50 MPa) to strong (R4, 50 MPa < UCS < 100 MPa); (CFEM, 2006)..

4.2.2.7 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets in Appendix A. Water was encountered at a depth of about 1.8 m below ground surface (Elevation 74.5 m) in Borehole 13-10, while Borehole 13-09 was dry upon completion of the overburden drilling to Elevation 76.2 m. It is noted that the groundwater level in Borehole 13-10 is similar to the water level in the adjacent Lake Ontario.

The water level at the site is expected to fluctuate seasonally in response to changes in the adjacent lake level, precipitation and snow melt, and is expected to be higher during the spring season and periods of precipitation.

4.2.3 Stella – In-Water Boreholes (Boreholes 12-07, 12-08, 13-05/13-05A and 13-06)

4.2.3.1 Lake Water

The water level in Lake Ontario at the site ranged from about Elevation 74.2 m to about Elevation 74.4 m during the 2012 investigation, and from elevation 74.9 m to Elevation 75.0 m Elevation during the 2013 investigation. The depth to the lakebed varied from about 2.8 m to 15.7 m at the borehole locations.

The water level elevation was surveyed to a temporary benchmark set up on each existing ferry dock at the start and end of the drilling of each borehole and the depths shown on the Records of Boreholes and Drillholes are referenced to the water surface.

4.2.3.2 Sand and Gravel to Gravel Fill

A deposit of sand and gravel to gravel fill was encountered at lake bottom in Borehole 12-07 at a depth of about 4.3 m below water surface (Elevation 69.9 m) and was found to be 1.6 m thick. The deposit is comprised of sand and gravel to gravel, trace to some sand trace silt, containing concrete fragments.

A single measured SPT ‘N’ value in the sand and gravel fill is 21 blows per 0.3 m of penetration, indicating that this deposit has a compact relative density.

The results of a grain size distribution test completed on one selected sample of the gravel fill is shown on Figure D1 in Appendix D.

The measured water content of one sample from this deposit is about 5 per cent.

4.2.3.3 Clayey Organic Silt

A deposit of clayey organic silt containing shells was encountered at lake bottom in Boreholes 13-05 and 13-06 at depths ranging from about 11.1 m to 15.7 m below water surface (Elevations 59.3 m to 63.8 m) and was measured to be between about 0.3 m and 1.8 m thick.

A single measured SPT ‘N’ value in the organic deposit is 0 blows (weight of rods) per 0.3 m of penetration, indicating that this deposit has a very soft relative density.

The results of a grain size distribution test completed on one selected sample of the organic deposit is shown on Figure D2 in Appendix D.

The measured water content of one sample from this deposit is about 173 per cent.

4.2.3.4 Sand and Gravel to Sandy Gravel

A deposit of sand and gravel to sandy gravel was encountered at lake bottom in Borehole 12-08 at a depth of about 2.8 m below water surface (Elevation 71.6 m) and underlying the organic deposits in Borehole 13-05 and 13-06 at depths ranging from about 11.4 m to 17.5 m below water surface. The deposit was measured to be between about 0.3 m and 1.1 m thick. The deposit is comprised of sand and gravel to sandy gravel, trace to some silt, trace clay, and was found to contain organics and shell fragments in the samples from Borehole 12-08.

Measured SPT ‘N’ values in this deposit are between 16 blows and 32 blows per 0.3 m of penetration, indicating that this deposit has a compact to dense relative density.

The results of grain size distribution tests completed on three selected samples of the deposit are shown on Figure D3 in Appendix D.

The measured water content of three samples from this deposit is between about 9 per cent and 10 per cent.

4.2.3.5 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes 12-07, 12-08, 13-05A and 13-06 at depths ranging between about 3.1 m and 18.6 m below water surface (Elevations 56.4 m to 71.3 m).

Based on the recovered bedrock core samples, the bedrock at the Stella site consists of Limestone inter-bedded with Shale to shaley limestone. In general, the bedrock samples are described as fine to medium grained, laminated to medium bedded, fine to medium grained, highly weathered to fresh, grey Limestone to shaley limestone. The Rock Quality Designation (RQD) measured on the core samples is between about 0 per cent and 88 per cent, but is generally less than about 30 per cent, indicating a rock mass of very poor to poor quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 0 per cent and 100 per cent.

Unconfined Compressive Strength (UCS) tests carried out on four samples of the Limestone bedrock recovered from the in-water boreholes at the Stella terminal measured compressive strengths between about 23 MPa and 98 MPa. The test results which are plotted on the Record of Drillhole sheets and summarised on Tables D1 to D4 in Appendix D, indicate that the bedrock is weak (R2) to strong (R4) as per Table 3.5 of CFEM (2006) reproduced here in Table D6 in Appendix D.

Point load index tests were performed on nine selected samples of the rock core recovered from the in-water boreholes at the Stella terminal. Point load strength index values are shown on the Record of Drillhole Sheets and on Table D7 in Appendix D. The point load index (Is_{50}) results from the laboratory tests carried out on the samples of the Limestone bedrock range from approximately 3.7 MPa to 8.2 MPa. These index values correspond to UCS values ranging between about 45 MPa and 100 MPa, based on a relationship between Is_{50} and UCS which is given by a correlation factor (k), estimated to be equal to 12 for this site, and calculated as the ratio of the average laboratory UCS and average corresponding point load test index value from all of the drillholes at the Stella terminal. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.

Based on the laboratory UCS tests and point load testing results (refer to Table D6 in Appendix D for details on the field estimation of rock hardness and R0, R1, etc. values outlined below), the estimated intact strength of the Limestone bedrock generally ranges from weak (R2, 5 MPa < UCS < 25 MPa) to strong (R4, 50 MPa < UCS < 100 MPa); (CFEM, 2006).

4.2.4 Stella – On-Shore Boreholes (Boreholes 13-11 and 13-12)

4.2.5 Topsoil

A 0.2 m thick surficial layer of topsoil was encountered at the ground surface in Borehole 13-12.

4.2.5.1 Clayey Silt Fill

A fill deposit consisting of clayey silt with sand and gravel, containing trace roots and rootles was encountered underlying the topsoil in Borehole 13-12 at a depth of about 0.2 m below ground surface (Elevation 76.2 m). The bottom of the clayey silt fill deposit was encountered at a depth of about 0.9 m below ground surface, corresponding to Elevation 75.5 m.

One Standard Penetration Test (SPT) “N” value measured within the cohesive fill was 20 blows per 0.3 m of penetration, suggesting a very stiff consistency.

The result of a grain size distribution test carried out on one sample of this fill deposit is shown on Figure D4 in Appendix D.

The measured water content of one sample from this deposit is about 14 per cent.

4.2.5.2 Sand and Gravel to Sandy Gravel Fill

A fill deposit consisting of sand and gravel to sandy gravel to gravel was encountered at ground surface in Borehole 13-11 and underlying the cohesive fill at a depth of about 0.9 m below ground surface (Elevation 75.5 m) in Borehole 13-12. The bottom of the fill deposit was encountered at a depth of 1.2 m to 2.7 m below ground surface (Elevations 75.2 m and 73.7 m, respectively) and the gravelly fill deposit was measured to be between about 0.3 m and 2.7 m thick.

Standard Penetration Test (SPT) “N” values measured within this fill deposit were between 7 and 25 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The results of grain size distribution tests carried out on two samples of this fill deposit are shown on Figure D5 in Appendix D.

The measured water content of two samples from this deposit are about 3 per cent and 9 per cent.

4.2.5.3 Wood (Existing Cribwork)

Wooden cribwork from the original ferry dock structure was encountered at a depth of about 2.7 m below ground surface (Elevation 73.7 m) in Borehole 13-11 and was found to be about 2.5 m thick.

4.2.5.4 Sandy Gravel to Gravel

A deposit of sandy gravel to gravel, some sand, trace to some silt, trace clay containing trace organics and crushed rock fragments was encountered underlying the timber cribwork at a depth of about 5.2 m below ground surface (Elevation 71.2 m) in Borehole 13-11. The bottom of this deposit was encountered at a depth of about 6.5 m below ground surface (Elevation 69.9 m) and the deposit was measured to be 1.3 m thick.

The result of a grain size distribution test carried out on one sample of this deposit is shown on Figure D6 in Appendix D.

The measured water content of two samples from this deposit are 11 per cent and 13 per cent.

4.2.5.5 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes 13-11 and 13-12 at depths of about 6.5 m and 1.2 m below ground surface, respectively, corresponding to Elevations 69.9 m and 75.2 m.

Based on the recovered bedrock core samples, the bedrock at this location consists of Limestone inter-bedded with Shale. In general, the bedrock samples are described as laminated, fine grained, slightly porous, slightly weathered to fresh, grey Limestone. The Rock Quality Designation (RQD) measured on the core samples is between about 0 per cent and 31 per cent, indicating a rock mass of very poor to poor quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 55 per cent and 100 per cent.

An Unconfined Compressive Strength (UCS) test carried out on one sample of the Limestone bedrock measured a compressive strength of about 74 MPa. The test result which is shown on the Record of Drillhole sheet and summarised on Table D5 in Appendix D, indicate that the bedrock is strong (R4) as per Table 3.5 of CFEM (2006) reproduced here in Table D6 in Appendix D.

Point load index tests were performed on six selected samples of the rock core recovered from the on-shore boreholes at the Stella terminal. Point load strength index values are shown on the Record of Drillhole Sheets and on Table D7 in Appendix D. The point load index (Is_{50}) results from the laboratory tests carried out on the samples of the Limestone bedrock range from approximately 2.7 MPa to 9.5 MPa. These index values correspond to UCS values ranging between 30 MPa and 110 MPa, based on a relationship between Is_{50} and UCS which is given by a correlation factor (k), estimated to be equal to 12 for this site, and calculated as the ratio of the average laboratory UCS and average corresponding point load test index values from all of the drillholes at the Stella terminal. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.

Based on the laboratory UCS tests and point load testing results (refer to Table D6 in Appendix D for details on the field estimation of rock hardness and R0, R1, etc. values outlined below), the estimated intact strength of the Limestone bedrock generally ranges from medium strong (R3, 25 MPa < UCS < 50 MPa) to strong (R4, 50 MPa < UCS < 100 MPa); (CFEM, 2006).

4.2.5.6 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets in Appendix C. Water was encountered at a depth of about 1.7 m below ground surface (Elevation 74.7 m) in Borehole 13-11, while Borehole 13-12 was dry upon completion of the overburden drilling to Elevation 75.2 m. It is noted that the groundwater level in Borehole 13-11 is similar to the water level in the adjacent Lake Ontario.

The water level at the site is expected to fluctuate seasonally in response to changes in the adjacent lake level, precipitation and snow melt, and is expected to be higher during the spring season and periods of precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. J. Paul Dittrich, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

FOUNDATION DESIGN REPORT
AMHERST ISLAND FERRY DOCKS CONVERSION STUDY
G.W.P. 4067-09-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the preliminary design of the the expansion of the existing Millhaven and Stella ferry terminals in Loyalist Township, Lennox and Addington County, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigations at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out preliminary design of the foundations for the proposed ferry terminal expansions. Where comments are made on construction, they are provided to highlight those aspects that could affect the preliminary design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Based on the Preliminary Design Drawings provided by URS, it is understood that the existing ferry terminals will be expanded to accommodate a large, end-loading vessel and new shelters, washrooms, mechanical and storage structures will be constructed and improvements will be made to the parking and marshalling areas.

It is our understanding that the existing ferry terminals were expanded from the original timber crib and rock fill configuration to the current size in 1992 by adding an outer concrete panel wall. The concrete panel wall foundations reportedly consist of W310x129 steel H-piles socketed 2.7 m into the limestone bedrock and tremie grouted in place on approximate 3.3 m to 4.8 m centre-to-centre spacing with tie rods connected to concrete anchor blocks constructed within the existing rock fill. According to the design drawings the area between the old timber cribbing and rock fill and the new outer terminal walls was backfilled with compacted granular A and “Dredged Class A material”.

6.2 Foundation Options

The Millhaven and Stella terminal expansions are to include the construction of new berthing walls, main ramp and back-up ramp structures, as well as a number of ancillary structures including an office/washroom/passenger amenity building, a mechanical & storage building and a sewage storage tank at each site.

Shallow foundations are not considered to be a feasible foundation alternative for the support of the berthing walls, main ramp or back-up ramp structures at these sites for the following reasons:

- Significant depth of water (up to about 8 m deep at Millhaven; up to about 15 m deep at Stella);
- Generally poor and variable thickness overburden soils (varying from non-existent to up to about 3.5 m thick very soft clayey organic silts to up to about 6.5 m thick of very loose to compact sand and gravel fills containing organics);
- Significant depth to bedrock (up to about 13.5 m below finished surface at Millhaven; up to about 20.5 m below finished surface at Stella);
- Significant lateral loading on berthing walls (i.e. from wave action, ice loads, ferry impact loads and retained soil);

- Significant axial loading and settlement sensitivity on main ramp and back-up ramp.

Given the above, only deep foundations will be discussed as an option for these structures, the details of which are presented in Section 6.3.

The office/washroom/passenger amenity building, the mechanical & storage building and the sewage storage tanks could be founded on either deep foundations or shallow foundations depending on their location relative to the subsurface conditions and depth to bedrock. The details of the options for these structures are presented in Section 6.4.

6.3 Berthing Walls, Main Ramp and Back-Up Ramp Foundations

Based on the Terminal Berthing Wall drawings provided by URS, the proposed design of the Berthing Walls comprises the use of pre-cast concrete panels and steel HP piles. It is understood that this is the preferred option from a structural perspective and that the steel H-piles would act as "soldier piles" and the concrete panels would be installed in between the channels of the piles to form the berthing walls. In order to provide adequate fixity and support of the steel H-piles at this site (given the variable composition and thickness of the overburden), the H-piles must be installed within tremie concrete filled sockets in the bedrock. This type of foundation design is considered appropriate for the conditions at this site.

Given that the berthing walls will be founded on steel H-piles placed within tremie concreted rock sockets, this type of foundation is also recommended for support of the Main Ramp and Back-up Ramp foundations. Although other types of deep foundations could be considered (including driven steel H-piles, driven steel tube piles, drilled steel casings, micropiles or concrete caissons) to support the berthing walls and ramp foundations, none offer an appreciable advantage over the H-piles socketed into bedrock and efficiencies will be realized in the design, construction and overall cost through the use of similar foundation types for all of these structures. Tables 1 and 2 following the text of this report present a comparison of the advantages, disadvantages, costs and risks/consequences for the various foundation alternatives for the berthing walls and ramps/backup ramps, respectively.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to found the berthing walls as well as the main ramp and back-up ramp foundations on rock socketed steel H-piles.

6.3.1 Rock Socketed H-Pile Foundations

The type of H-pile section most appropriate for supporting the walls and ramps will depend on the axial loading from the ramp structures as well as on the lateral loads to be resisted by, and the structural design requirements of, the berthing walls. For the purposes of this report, two H-pile sections have been considered:

- HP 310x110 (standard HP section for MTO bridge work)
- HP 310x132 (slightly heavier section if required for increased lateral rigidity)

The minimum required strength of the concrete recommended to fill the annular space between the H-Piles and the drilled hole of the rock sockets, from a foundations perspective, is 35 MPa. This minimum strength requirement should also be reviewed by the structural engineer.

The following sections provide details of the recommendations for supporting the berthing walls and ramps on rock socketed H-pile foundations at the Millhaven and Stella sites.

6.3.1.1 Frost Protection

The estimated depth of frost penetration at this site is 1.5 m based on OPSD 3090.101 *Foundation Frost Depths for Southern Ontario.* Pile caps supporting the ramp structures should be provided with a minimum of 1.5 m of soil cover for frost protection. Pile caps that do not support a structure (i.e. those at the top of the berthing walls) and that are backfilled with non-frost susceptible soils (i.e. Granular B Type II) do not require a minimum cover for frost protection given that the historic water levels (as provided by URS) indicate that the lake level is expected to remain about 1.6 m below the bottom of the berthing wall pile caps during winter months.

The required thickness of conventional soil cover for frost protection of pile caps should also be provided in the horizontal direction, around the perimeter of any pile caps adjacent to a vertical and exposed face (such as the edge of a berthing wall) that could be subjected to freezing.

If adequate soil cover cannot be provided for the pile caps (in the horizontal and vertical directions), rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action, however rigid styrofoam insulation cannot be used below the water level in the lake.

6.3.1.2 Founding Elevations

Drawings 2 and 4 show an approximate bedrock surface profile along several sections through the proposed berthing wall areas at the Millhaven and Stella sites, respectively. The rock sockets for the H-pile foundations should extend into “sound” bedrock below observed zones of weathering or highly fractured zones which is estimated to be about 1.0 m below the top of bedrock. However it is noted that the depth of the fractured/weathered zone as well as the variation in the elevation of the bedrock surface at the ramp locations and along the alignments of the berthing walls will need to be confirmed at the detail design-build stage. The length of the rock socket below “sound” bedrock for the berthing walls will be governed by the required resistance to lateral loading.

In order to accommodate the proposed H-pile sections and provide adequate space for installation of the tremie concrete, the diameter of the rock sockets will need to be at least 0.6 m. The limestone bedrock is weak to very strong (with unconfined compressive strengths typically in the range of about 25 MPa to 75 MPa), and so the rock sockets will have to be advanced into the bedrock by use of a rotary drilling system and down-the-hole (DTH) hammer such as the Symmetrix drilling system or equivalent.

Depending on the tolerance of nearby buildings and the existing ferry terminals to vibration, vibration monitoring may be appropriate during bedrock drilling as discussed in Section 6.9.6.

6.3.1.3 Axial Geotechnical Resistance/Reaction

Steel H-piles socketed a minimum of 1 m below the bedrock surface (i.e. to found on “sound” bedrock) should be designed based on a factored axial geotechnical resistance at Ultimate Limit States (ULS) and a geotechnical reaction at Serviceability Limit States (SLS for 25 mm of settlement) as given below.

Foundation Type	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS[1]
HP 310x110	2,000 kN	N/A
HP 310x132	2,400 kN	N/A

Note: 1. For pile foundations founded on the limestone bedrock, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

As described in Section 6.3.1.6, rock sockets deeper than 1 m below the surface of the bedrock may be required to provide sufficient resistance to lateral loads.

6.3.1.4 Downdrag Loads

So long as all existing clayey and organic soils are sub-excavated from within the limits of the new construction prior to backfilling behind the berthing walls and pile installation, the potential for downdrag loads on the piles should be negligible.

6.3.1.5 Resistance to Lateral Loads - Backfill

Resistance to lateral loading for the piles supporting the ramp structures can be derived from a combination of the backfill behind the berthing walls (through which the piles will be installed) as well as the bedrock socket at the base of the pile. It is noted that the backfill surrounding the pile elements must extend laterally a distance of at least ten (10) pile diameters (i.e. the zone of influence around a laterally loaded pile based on Mezazigh and Levache (1998)) away from the perimeter limits of the pile caps in order to rely on the lateral resistances of the backfill provided below. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below (CFEM, 1992, as noted in Section C6.8.7.1 (Table C6.5) and in Section C6.8.7.3 of the *Commentary to CHBDC*).

For cohesionless soils:

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

where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
n_h is the constant of subgrade reaction (kPa/m);
z is the depth (m); and
B is the pile diameter / width (m).

As discussed further in Section 6.8, it is recommended that all existing clayey and organic soils be sub-excavated and replaced by new backfill comprised of Granular B Type II behind the berthing walls. It is further recommended that compactive effort be applied to the below water fills by use of a vibroflot or other suitable method to provide adequate densification of the end-dumped below water fills. Based on this assumption, the following value of n_h may be assumed in the structural analyses of the lateral pile response, using a thickness of backfill based on the depth to bedrock shown on the interpreted stratigraphic sections on Drawings 2 and 4.

Soil Unit	n_h (kPa/m)
Compacted Granular B Type II	4,400

Given the variable thickness, composition and generally poor nature of the overburden soils that will remain outside of the berthing walls, and the potential for future scour and/or the effects of dredging, reliance on the lateral resistance of these soils for design of the berthing walls is not recommended. Resistance to lateral loads on the berthing walls can be provided by the tie rods/dead-man anchor blocks as indicated on the preliminary design drawings (and discussed in Section 6.6).

6.3.1.6 Resistance to Lateral Loads - Bedrock

Resistance to lateral loading for the piles supporting the ramp structures as well as the piles supporting the berthing walls can be derived from the rock socketed H-piles encased in concrete with a minimum embedment below the surface of the bedrock of 2 m at the Millhaven site, and 4 m at the Stella site based on the results of lateral pile analyses incorporating a lateral load of 365 kN on the steel H-Pile at the surface of the bedrock as supplied by the structural engineer.

The lateral load response of a single pile may be calculated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h, (kPa/m) for the limestone bedrock. It is anticipated that the rock will remain in the elastic range for the design loading; however, this assumption should be checked once the design is finalized and the maximum lateral loads on a single pile are available. For loading within the elastic range, closed form solutions are applicable for the estimation of the coefficient of horizontal subgrade reaction, k_h, as follows:

$$k_h = \frac{4\pi(1-\nu)E_h}{(3-4\nu)(1+\nu)}\frac{1}{\ln(r_o/r_i)}$$

where

ν is the Poisson's ratio = 0.2;

r_o is the radius of 'zero' deformation; typically 10 to 15 pile or rock socket diameters (m);

r_i is the pile or rock socket radius (m); and,

E_h is the lateral rock mass elastic modulus (kPa).

Location	Depth below Bedrock Surface	Lateral Rock Mass Elastic Modulus (E_h)	k_h (MN/m/m)
Millhaven Site	0 m to 1 m	750 MPa	1,000
	>1 m	4,200 MPa	5,500
Stella Site	0 m to 1 m	475 MPa	600
	>1 m	1,000 MPa	1,300

Note: Diameter of rock socket assumed to be 0.6 m

The lateral rock mass moduli indicated above have been estimated based on consideration of the intact UCS strengths of the bedrock as well as the RQD values of the bedrock as assessed from the rock core.

6.3.1.7 Group Effects

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R, as follows:

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

Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading d = Pile Diameter/width	**Horizontal Subgrade Reaction Reduction Factor, R**
4 d	1.00
1 d	0.50

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

6.3.1.8 Lateral Pile Capacity - Backfill

The passive resistance of the portion of the pile embedded in the berthing wall backfill has been estimated based on typical values provided in the CHBDC for a steel H-pile in a granular material. For a single HP 310 vertical pile, the factored lateral resistance at ULS is estimated to be about 100 kN while the lateral reaction at SLS (for 10 mm of horizontal deflection) is estimated to be about 35 kN. As noted in Section 6.3.1.5, the backfill surrounding the pile elements must extend laterally a distance of at least ten (10) pile diameters away from the perimeter limits of the pile caps in order to rely on the lateral resistances at ULS and SLS provided.

Given the variable thickness, composition and generally poor nature of the overburden soils that will remain outside of the berthing walls, and the potential for future scour and/or the effects of dredging, reliance on the lateral resistance of these soils for design of the berthing walls is not recommended. Resistance to lateral loads on the berthing walls can be provided by the tie rods/dead-man anchor blocks as indicated on the preliminary design drawings (and discussed in Section 6.6).

6.3.1.9 Lateral Pile Capacity - Bedrock

The passive resistance of the portion of the pile socketed into bedrock has been assessed using a Rock Mass Rating (RMR) profile for the bedrock based on the RQD values and UCS strength values measured on the rock core recovered from the boreholes. The ultimate lateral capacity of the bedrock with a minimum embedment into bedrock of 2 m at the Millhaven site, and 4 m at the Stella site is outlined below.

Location	Ultimate Lateral Capacity (MPa)
Millhaven Site	5
Stella Site	1

The capacity per metre length of pile within the bedrock (in kN) can be determined by multiplying the Ultimate Lateral Capacity, given above, by the diameter of the pile socket.

A geotechnical resistance factor of 0.5 should be applied to the calculated values of the ultimate lateral/passive resistance of the pile, based on the methods described above, for the section of the pile in the bedrock.

6.3.1.10 Comments on Design for Lateral Loads

If adequate resistance to lateral loads cannot be developed from the soil surrounding the piles, or if piles cannot be located laterally a distance of at least ten (10) pile diameters away from adjacent piles or the berthing walls, as noted in Sections 6.3.1.5 and 6.3.1.7, consideration could be given to the use of battered piles, or tie back anchors and anchor blocks to provide additional lateral resistance. Alternatively, the berthing walls could be designed to support the additional lateral loads induced by the piles by decreasing the distance between piles in the berthing wall or using piles with a stiffer cross section in these areas.

6.4 Passenger Amenity Building, Storage Building and Sewage Tank Foundations

A brief discussion of the different foundation options and their appropriateness for the various ancillary structures based on location and thickness of overburden is provided below. A comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs for each structure is provided in Table 3 following the text of this report.

- **Shallow Foundations on Overburden:** The thickness of overburden at the proposed ancillary structures varies from 0 m (non-existent) to up to about 2.5 m and its composition ranges from native sand and gravel containing organics and shell fragments to variable fills including sands and gravels, crushed concrete and clayey silts/silty clays. These soils are not considered suitable as founding strata for shallow foundations for the proposed structures.
- **Shallow Foundations on Bedrock:** The depth to the bedrock surface at the proposed ancillary structures below the underside of the foundations (considering the requirements for frost protection) varies from 0 m up to about 6.5 m. Where bedrock is expected at shallow depth (i.e. at the locations of the Sewage Tanks at both sites and at the location of the Passenger Amenity building at the Stella site), shallow footings founded directly on bedrock are feasible and the preferred type of foundation for these structures.
- **Shallow Foundations "perched" on Compacted Granular Pads:** Where the depth to bedrock below the underside of the foundations (considering the requirements for frost protection) is greater than

about 1 m (i.e. at the locations of the Mechanical & Storage Buildings at both sites and at the location of the Passenger Amenity building at the Millhaven site), the structures could be supported on shallow foundations "perched" on compacted granular pads constructed within the backfill behind the ferry terminal berthing walls. For this alternative, the compacted granular pads must have a minimum thickness of 1 m. The pads would consist of OPSS. Prov. 1010 (Aggregates) Granular 'A' material extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS 501 (Compacting) and Special Provision (SP) 105S21. However, given the anticipated variable thickness of backfill behind the berthing walls in the vicinity of these structures and the difficulty with compacting the backfill below the lake level, there is a risk of these foundations experiencing differential settlements.

- **Deep Foundations (H-Piles, Tube Piles or Caissons seated on Bedrock):** Drilled concrete caissons seated on bedrock or steel H-piles or tube piles driven to bedrock are a feasible alternative to support the ancillary structures where the depth to bedrock below the underside of a pile cap (considering the requirements for frost protection) is greater than about 3 m (i.e. at the locations of the Mechanical & Storage Buildings at both sites and at the location of the Passenger Amenity building at the Millhaven site). The use of deep foundations to support these structures would avoid the requirement to provide significant compactive effort to the backfill behind the berthing walls and would minimize the risk of differential settlement at these structures.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the Sewage Tanks at both sites and the Passenger Amenity building at the Stella site on shallow foundations on bedrock. In addition, the preferred option from a foundations perspective is to found the Mechanical & Storage Buildings at both sites and the Passenger Amenity building at the Millhaven site on deep foundations.

6.4.1 Shallow Foundations

The following sections provide details of the recommendations for supporting the ancillary structures at the Millhaven and Stella sites on shallow foundations.

6.4.1.1 Frost Protection

The estimated depth of frost penetration at this site is 1.5 m based on OPSD 3090.101 *Foundation Frost Depths for Southern Ontario*.

The RQD of the upper portion of the bedrock in the boreholes advanced furthest on-shore and in the general vicinity of the proposed Passenger Amenity building at the Stella site and the Sewage Tanks at both sites is generally low (RQD less than about 15%). As such, shallow foundations constructed directly on the bedrock in these areas could be susceptible to frost heave and should be provided with a minimum of 1.5 m of soil cover for frost protection.

At the other ancillary structure locations, if the structures are supported on shallow foundations founded on a compacted Granular 'A' pad, all footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

The required thickness of conventional soil cover for frost protection of footings should also be provided in the horizontal direction, around the perimeter of any footings adjacent to a vertical and exposed face (such as the edge of a berthing wall) that could be subjected to freezing.

If adequate soil cover cannot be provided for the footings (in the horizontal and vertical directions), rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action, however rigid styrofoam insulation cannot be used below the water level in the lake.

6.4.1.2 *Founding Elevations*

For support of the proposed Passenger Amenity building, mechanical and storage buildings, and sewage holding tanks, strip or spread footings or mat foundations should be founded below any existing fill and directly on the limestone bedrock or on compacted Granular 'A' pads within the new backfill behind the ferry terminal walls.

The following founding elevations are recommended for shallow foundations supporting the proposed ancillary structures at both sites.

Structure	Estimated Finished Grade/Surface Elevation	Approximate Strip/Spread Footing or Mat Founding Elevation	Founding Stratum	Approximate Maximum Excavation Depth Relative to Finished Grade
Passenger Amenity building (Millhaven)	76.7 m	75.2 m[1]	Compacted Granular 'A' Pad over Berthing Wall Backfill	1.5 m
Mechanical and Storage Building (Millhaven)	76.7 m	75.2 m[1]	Compacted Granular 'A' Pad over Berthing Wall Backfill	1.5 m
Sewage Holding Tank (Millhaven)	77.5 m	73.5 m	Limestone Bedrock	4.0 m (up to about 2.7 m of bedrock excavation required)
Passenger Amenity building (Stella)	77.4 m	75.2 m[1]	Limestone Bedrock	2.2 m
Mechanical and Storage Building (Stella)	76.1 m	74.6 m	Compacted Granular 'A' Pad over Berthing Wall Backfill	1.5 m
Sewage Holding Tank (Stella)	77.6 m	73.6 m	Limestone Bedrock	4.0 m (up to about 1.6 m of bedrock excavation required)

Note: 1. Requires construction of minimum 1 m thick compacted Granular A pad below this elevation.

Shallow foundation construction at the sewage holding tanks and the Passenger Amenity building at the Stella site will require sub-excavation and removal of all overburden materials, including any existing fills and exposure of the bedrock surface. Additional bedrock excavation will be required to reach the founding level at the sewage holding tanks. The founding surface of the bedrock shall be free from all debris and any loose/shattered rock and should be inspected by a Quality Verification Engineer following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* prior to foundation construction.

If shallow foundations are adopted for the mechanical & storage buildings at both sites and at the location of the Passenger Amenity building at the Millhaven site, all existing subgrade soils/fills in the vicinity of these structures shall be removed and replaced with Granular B Type II (below the lake level). Compactive effort should be applied to the below water fills by use of a vibroflot or other suitable method to provide adequate densification of the end-dumped below water fills. A minimum 1 m thick Granular A pad compacted to 100% of the materials Standard Proctor maximum dry density shall be provided below the foundations. It is noted that construction of this compacted engineered fill pad must be carried out in the dry and will require temporary dewatering.

6.4.1.3 Geotechnical Resistance/Reaction

Strip or spread footings or mat foundations placed on the properly prepared limestone bedrock founded at or below the design elevations given in the preceding section, should be designed based on a factored geotechnical resistances at Ultimate Limit States (ULS) of 2,000 kPa. For shallow foundations founded on the properly prepared and inspected bedrock, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

Strip or spread footings founded on a compacted Granular ‘A’ pad within the berthing wall backfill at or below the design elevations given in the preceding section, should be designed based on a factored geotechnical resistances at Ultimate Limit States (ULS) of 400 kPa and geotechnical reactions at Serviceability Limit States (SLS for 25 mm of settlement) of 200 kPa for footings up to about 1.2 m wide. Mat foundations could be considered, but a lower geotechnical reaction at SLS may apply to limit settlements to 25 mm. As noted previously, given the potential for variation in the thickness of fills across the structure foundation footprints, differential settlements may occur, the magnitude of which will have to be considered at detailed design when additional boreholes/probeholes are advanced to better delineate the depth to bedrock in the critical areas.

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 Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC).*

6.4.1.4 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings the coefficient of friction, tan δ can be taken as follows:

Interface Materials	Coefficient of Friction (tan δ)
Cast-in-Place Concrete Footing on Bedrock	0.70
Cast-in-Place Concrete Footing on Compacted Granular 'A' Pad	0.58

The values presented above represent unfactored values.

6.4.1.5 Slabs-on-Grade

Sewage Holding Tanks (Both Terminals) and Passenger Amenity Building at Stella Terminal

For the above noted structures (i.e. for the tanks and building that will be designed with shallow footings founded on bedrock or on relatively thin compacted Granular A over bedrock), if the entire layer of existing fill and native soil is sub-excavated to the bedrock surface and replaced with compacted OPSS Granular A engineered fill, the average k_{v1} is estimated to be 40 MPa/m for a 0.3 m x 0.3 m square footing founded on the engineered fill. The value of k_s for a given slab on grade foundation B meters wide and mB meters long can be calculated based on k_{v1} by using the following formula.

$$k_s = k_{v1}\left(\frac{b+0.3}{2b}\right)^2$$

where

K_s is the modulus of subgrade reaction (kPa/m);

K_{v1} is the is the modulus of subgrade reaction (kPa/m) for a 0.3 m x 0.3 m footing; and

b is the foundation width (meters).

Mechanical and Storage Buildings (Both Terminals) and Passenger Amenity Building at Millhaven Terminal

For structures that will be founded on piles behind the berthing walls, if the entire layer of existing soil is sub-excavated to the bedrock surface and replaced with OPSS Granular B Type II engineered fill end dumped below the water level and properly placed and compact above the water level, the average k_{v1} is estimated to be 25 MPa/m for a 0.3 m x 0.3 m square footing founded on the engineered fill. The value of k_s for a given slab on grade foundation B meters wide and mB meters long can be calculated based on k_{v1} by using the following formula.

$$k_s = k_{v1}\left(\frac{b+0.3}{2b}\right)^2$$

where

K_s is the modulus of subgrade reaction (kPa/m);

K_{v1} is the is the modulus of subgrade reaction (kPa/m) for a 0.3 m x 0.3 m footing; and

b is the foundation width (meters).

Since the level of compaction achievable below the water level is not known there is some risk of differential settlement over the length of the slab on grade and between the slab on grade and the pile supported building foundations which could require future maintenance. Alternatively, to mitigate the risk of differential settlements the structures could be designed with a structural slab

The geotechnical bearing resistances and moduli of subgrade reaction provided in this section are based on uniform, vertical and concentric loading conditions.

6.4.2 Deep Foundations

Deep foundations are not recommended to support the sewage holding tanks at either site nor the Passenger Amenity building at the Stella site. However, deep foundations are recommended to support the mechanical & storage buildings at both sites and the Passenger Amenity building at the Millhaven site, and the following types can be considered:

- 0.6 m diameter concrete filled caissons seated on bedrock;
- HP 310x79 steel piles driven to bedrock; or,
- Closed-ended, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in) driven to bedrock.

Given the anticipated relatively light loads of the ancillary structures, small pile sections have been selected for consideration and socketing of the piles into bedrock is not considered necessary.

The following sections provide details of the recommendations for supporting the appropriate ancillary structures at the Millhaven and Stella sites on deep foundations.

6.4.2.1 Frost Protection

The estimated depth of frost penetration at this site is 1.5 m based on OPSD 3090.101 *Foundation Frost Depths for Southern Ontario.* All pile caps should be provided with a minimum of 1.5 m of soil cover for frost protection.

The required thickness of conventional soil cover for frost protection of pile caps should also be provided in the horizontal direction, around the perimeter of any pile caps adjacent to a vertical and exposed face (such as the edge of a berthing wall) that could be subjected to freezing.

If adequate soil cover cannot be provided for the pile caps (in the horizontal and vertical directions), rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action, however rigid styrofoam insulation cannot be used below the water level in the lake.

6.4.2.2 Founding Elevations

The following provides estimated elevations for the top of the bedrock and approximate lengths of pile elements at the appropriate ancillary structures.

Structure	Estimated Finished Grade/Surface Elevation	Approximate Underside of Pile Cap Elevation	Pile Tip Founding Stratum	Estimate Elevation of Bedrock Surface[1]	Approximate Length of Pile Elements[2]
Passenger Amenity building (Millhaven)	76.7 m	75.2 m[1]	Limestone Bedrock	71.5 m to 73.5 m	1.7 m to 3.7 m
Mechanical and	76.7 m	75.2 m[1]	Limestone Bedrock	68.7 m	6.5 m

Storage Building (Millhaven)					
Mechanical and Storage Building (Stella)	76.1 m	74.6 m	Limestone Bedrock	71.3 m	3.3 m

Note: 1. Approximate only based on closest available borehole information. Requires confirmation during detailed design.

2. Minimum pile length should be taken as 3.0 m

6.4.2.3 Axial Geotechnical Resistance/Reaction

Concrete caissons seated on bedrock or steel piles driven to the top of bedrock should be designed based on the factored axial geotechnical resistance at ULS and a geotechnical reaction at SLS as given below.

Foundation Type	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS[1]
0.6 m diameter Concrete Caissons	1,600 kN	N/A
HP 310x79 Steel Piles driven to bedrock	1,400 kN	N/A
324 mm diameter Steel Pipe Piles driven to bedrock	1,400 kN	N/A

Note: 1. For pile foundations founded on the limestone bedrock, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

For the concrete caisson options, it is noted that the base of the caisson/top of limestone bedrock would have to be properly cleaned and inspected in order to rely on the value of ULS provided above.

For the driven steel pile options, it is noted that rock points would be required to ensure proper seating on the sloping bedrock.

6.4.2.4 Downdrag Loads

So long as all existing clayey and organic soils are sub-excavated from within the limits of the new construction prior to backfilling behind the berthing walls and pile installation, the potential for downdrag loads on the piles should be negligible.

6.4.2.5 Resistance to Lateral Loads

Resistance to lateral loading can be derived from the backfill behind the berthing walls through which the caissons seated on bedrock or the steel piles driven to bedrock will be installed. It is noted that the backfill surrounding the pile elements must extend laterally a distance of at least ten (10) pile diameters away from the perimeter limits of the pile caps in order to rely on the lateral resistances provided below. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below (CFEM, 1992, as noted in Section C6.8.7.1 (Table C6.5) and in Section C6.8.7.3 of the *Commentary to CHBDC*).

For cohesionless soils:

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

where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
n_h is the constant of subgrade reaction (kPa/m);
z is the depth (m); and
B is the pile diameter / width (m).

As discussed further in Section 6.8, it is recommended that all existing clayey and organic soils be sub-excavated and replaced by new backfill comprised of Granular B Type II behind the berthing walls. It is further recommended that compactive effort be applied to the below water fills by use of a vibroflot or other suitable method to provide adequate densification of the end-dumped below water fills. Based on this assumption, the following value of n_h may be assumed in the structural analyses of the lateral pile response, using a thickness of backfill based on the depth to bedrock shown on the interpreted stratigraphic sections on Drawings 2 and 4.

Soil Unit	**n_h (kPa/m)**
Compacted Granular B Type II	4,400

6.4.2.6 Group Effects

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R, as follows:

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

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

6.4.2.7 Lateral Pile Capacity

The passive resistance of the pile embedded in the berthing wall backfill has been estimated based on typical values provided in the CHBDC for a steel H-pile in a granular material. For a single HP 310 vertical pile, the factored lateral resistance at ULS is estimated to be about 100 kN while the lateral reaction at SLS (for 10 mm of horizontal deflection) is estimated to be about 35 kN. As noted in Section 6.3.1.5, the backfill surrounding the pile elements must extend laterally a distance of at least ten (10) pile diameters away from the perimeter limits of the pile caps in order to rely on the lateral resistances at ULS and SLS provided.

6.4.2.8 Comments on Design for Lateral Loads

If adequate resistance to lateral loads cannot be developed from the soil surrounding the piles, or if piles cannot be located laterally a distance of at least ten (10) pile diameters away from adjacent piles or the berthing walls, as noted in Sections 6.3.1.5, 6.3.1.7 and 6.3.1.10, consideration could be given to the use of battered piles, or tie back anchors and anchor blocks to provide additional lateral resistance. Alternatively, the berthing walls could be designed to support the additional lateral loads induced by the piles by decreasing the distance between piles in the berthing wall or using piles with a stiffer cross section in these areas.

6.5 Seismic Considerations

6.5.1 Passenger Amenity and Storage Buildings and Tanks

The 2012 Ontario Building Code (OBC 2012) contains seismic analysis and design methodology applicable to the design of the buildings and tanks. The seismic site classification methodology outlined in the code is based on the average subsurface conditions within the upper 30 m below ground surface in accordance with Section 4.1.8.4 of the OBC.

The site classification can be assessed based on measurements of shear wave velocity or based on SPT N-values. As described in Section 4.2, the soils encountered at the on-shore borehole locations are generally very stiff/compact to dense. The soils encountered in the off-shore boreholes are generally very soft/very loose to soft/loose, however they will sub-excavated and replaced with Granular B Type II but the level of compaction achievable below the water level is unknown. Based on these soil types, this site should be classified as Site Class D (Stiff soil) for on-shore structures and as Site Class E (Soft soil) for off-shore structures for engineering purposes, based on OBC.

If footings for a proposed structure are to be located within 3 m of the bedrock surface, the site can be classified as Site Class B (Rock) for engineering purposes.

6.5.2 Seismic Analysis Coefficient

Based on the latitude and longitude of the site, (44.1871 N and 76.804 W), the peak ground acceleration (PGA) is reported to be equal to 0.112g, and the 5% damped spectral response acceleration values, $S_a(T)$ at the site based on the information obtained from the NRCan website for a probability of exceedance of 2% in 50 years (as required by the OBC design guidelines) are provided below.

2%/50 years (0.000404 per annum) probability			
Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
0.275 g	0.170 g	0.095 g	0.030 g

6.5.3 Berthing Walls

6.5.3.1 Site Coefficient

For seismic design of the berthing walls and ramps, the Site Coefficient, S, for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC may be taken as 1.0; consistent with Soil Profile Type I.

6.5.3.2 Seismic Analysis Coefficient

Based on the latitude and longitude of the site, (44.1871 N and 76.804 W), the peak horizontal acceleration (PHA) is reported to be equal to 0.79g at the site based on the information obtained from the NRCan website for a probability of exceedance of 10% in 50 years (as required by the CHBDC guidelines). Based on a PHA<0.08 and according to Table 4.1 of the CHBDC, this site is located in Seismic Performance Zone 1 and the corresponding site-specific zonal acceleration ratio, A, is 0.05. Given this assessment, and in accordance with Section 4.4.5.1 of the CHBDC, seismic analysis is not required for a structure located in Seismic Performance Zone 1.

6.6 Lateral Earth Pressures for Design

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

The following recommendations are made concerning design of the berthing walls:

- All existing clayey and organic soils present within the footprint of the terminals should be sub-excavated and replaced prior to berthing wall construction and backfilling.
- Select free-draining granular fill, in accordance with OPSS Granular B Type II gradation specifications (or Granular A for pad construction below shallow foundations), should be used as backfill behind the berthing walls.
- All below water (i.e. below lake level) granular fills should be subjected to compactive effort by use of a vibroflot or other suitable method to provide adequate densification of the end-dumped granular materials.
- All granular backfill placed above the lake level should be placed in maximum 200 mm loose lifts and uniformly compacted to at least 95 per cent of the SPMDD of the material. If the backfill soils are to support settlement sensitive structures, the level of compaction should be increased to at least 100% SPMDD. Heavy compaction equipment should not be used within the lateral distance behind any structure equal to the current height of the fill above the base of the structure.
- An unbalanced water head behind the berthing walls of at least 0.5 m should be included in the structural analysis to account for changes in lake levels.
- If the berthing walls are permitted to yield sufficiently for "active" earth pressure conditions to occur (possibly in the area of the dead-man anchors), an "active" lateral earth pressure coefficient (K_a) should be used in design. In this case, the appropriate geotechnical design parameters are provided below for a triangular lateral earth pressure distribution.

- If the berthing walls are not designed to permit sufficient movement for "active" conditions to occur (possibly in the area where adjacent berthing walls are connected by tie-rods), an "at-rest" lateral earth pressure coefficient (K_o) should be used in design. In this case, the appropriate geotechnical design parameters are provided below for a triangular lateral earth pressure distribution.

- Typically, wall design allows for lateral movement of the walls to be resisted by any soil present in front of the wall and, in this case, passive lateral earth pressures may be used in the geotechnical design of the wall. However, at this site, given the variable thickness, composition and generally poor nature of the overburden soils that will remain outside of the berthing walls, as well as the potential for future scour to occur along the face of the berthing walls from propeller wash of the proposed larger-use ferries, reliance on the passive lateral resistance of these soils for design of the walls is not recommended.

- At the locations of the dead-man anchors, the passive resistance provided by the wedge of compacted granular fill in front of the pre-cast concrete anchor blocks may relied on so long as the fill in front of the anchor blocks comprises Granular A compacted to at least 100% SPMDD. The movements required to fully mobilize passive resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize full passive resistance; if this is the case, at-rest earth pressures should be assumed for geotechnical design. Assuming that the retaining wall is permitted to yield sufficiently to develop "passive" earth pressure conditions to occur at the locations of the dead-man anchors, a "passive" lateral earth pressure coefficient (K_p) may be used in design as summarized below. A resistance factor equal to 0.5 should be applied to the passive resistance for ULS conditions according to CFEM 2006.

Fill Type	Soil Unit Weight (kN/m^3)	Coefficients of Static Lateral Earth Pressure		
		Active, K_a	At-Rest, K_o	Passive, K_p[1]
Granular 'A'	22	0.27	0.43	3.7
Granular 'B' Type II	21	0.27	0.43	N/A
Existing Crushed Concrete Fill, Sand and Gravel Fill and native Sand and Gravel	19	0.36	0.53	N/A

Note: 1. Applicable for design of the dead-man anchors only.

- It should be noted that the earth pressure coefficients provided above assume that berthing walls are backfilled with free-draining, compacted granular backfill and that the backfill surface is horizontal (i.e. not sloping). Any sloping backfill or anticipated surcharge loading within a distance from the back face of the wall equal to the wall height, including traffic and foundation loads, should be included in the design.

- Lateral pressures from surcharge loading shall be taken into account in accordance with Section 6.9.5 of CHBDC and its commentary.

- All berthing wall backfill materials and general design / construction methods should also meet the requirements specified by the berthing wall designer.

6.6.1 Seismic Lateral Earth Pressures

Seismic loading must be taken into account in accordance with Section 4.6.4 of *CHBDC*, as it can result in increased lateral earth pressures acting on the berthing walls.

The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

where K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3)

- taken as soil unit weights given above for fill materials
- taken as 19 kN/m^3 for the native materials

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

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) is 0.05. The site-specific peak horizontal ground acceleration (PHA) is 0.079g based on the NRC website. The Site Coefficient (S) may be taken as 1.0 in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on the subsurface conditions at the site, no significant amplification of the ground motion is expected.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.05. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary.*

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case B	
	Granular A	**Granular B Type II**
Yielding Wall	0.26	0.26
Non-Yielding Wall	0.29	0.29

Notes:

1. These seismic K_{AE} values include the effect of wall friction, and assume that the back of the wall is vertical and the ground surface behind the wall is flat.
2. The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.05. This corresponds to displacements of up to approximately 15 mm at this site.

It is noted that for the very low zonal acceleration ratio for this site, the seismic K_{AE} values are similar to or less than the static values of K_a and K_o reported above.

6.7 Shoreline Erosion Protection

All shoreline within the project limits, beyond the extents of the berthing walls, should be provided with erosion protection (i.e. rip-rap) of a suitable size and thickness in order to protect the shore and/or adjacent slopes from undermining/erosion by wave action. The shoreline erosion protection should be constructed in accordance with OPSS 511 – Rock Protection.

As part of the rip-rap design and installation, provision should be made to ensure that measures (such as the use of a geotextile separator) are adopted to protect the loss of fine materials from the existing slopes through the erosion protection.

Based on observations of the existing shoreline in the area of the project, the existing rip-rap ranges in size from about 0.15 m up to 0.6 m and appears to be performing well and not allowing active erosion to occur.

6.8 Subgrade Preparation and Backfilling

All layers of topsoil and clayey or organic fills (on-shore) and all deposits of clayey organic silts (off-shore) should be removed prior to any new fill placement. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

6.8.1 Removal of Clayey and Organic Materials

Based on the information from the boreholes obtained during the field investigation, layers of topsoil and/or clayey organic fills up to about 1.2 m thick can be expected in some of the existing on-shore areas. Deposits of very soft organic clayey silt up to about 3.4 m thick was encountered at some of the boreholes advanced in the off-shore areas. All topsoil, clayey organic fills and deposits of organic clayey silt must be removed from the plan limits of the proposed works prior to fill placement for the new construction.

6.8.2 Backfill Material and Placement

Backfill behind the berthing walls should comprise Granular B Type II. Rock fill is not recommended for backfilling for the following reasons:

- Potential for post-construction settlement of rock fill placed by end-dumping below the lake level:
  - estimated to be up to about 250 mm (short-term) and up to about 25 mm (long-term) for the >11.5 m thick fill required at the Millhaven site.
  - estimated to be up to about 350 mm (short-term) and up to about 35 mm (long-term) for the >17.5 m thick fill required at the Stella site.
- Potential for loss of fines from any fill material placed above the rock fill (and above lake level) due to fluctuations in lake water levels and vibrations from wave action and/or ferry impact which could lead to the formation of voids in the near surface backfill and additional settlement in the long-term.
- Rock fill would create obstructions and cause problems with pile installation for support of ramps and other ancillary structures located within the limits of the berthing walls.

In order to improve the density of the Granular B Type II placed by end-dumping below the lake level and further reduce the potential for any post-construction settlements, it is recommended that the granular fills be subjected to compactive effort by use of a vibroflot or other suitable method to provide adequate densification of the end-dumped granular materials.

All other fill placed above the lake level should comprise either Granular B Type II or alternative Granular A (for pad construction below footings) placed in maximum 200 mm loose lifts and uniformly compacted to at least 95 per cent of the SPMDD of the material. If the backfill soils are to support settlement sensitive structures, the level of compaction should be increased to at least 100% SPMDD.

6.8.3 PVC Sleeve Protection

Based on the Terminal Berthing Wall drawings provided by URS, the proposed design of the Berthing Wall includes the construction of pre-cast concrete anchor blocks joined to the berthing wall pile cap by tie-rods. In order to protect the tie-rods from the loading imposed by the compaction of the overlying granular materials, as well as from any potential settlement of the underlying fills, it is recommended that a PVC protective sleeve be installed around the tie-rods. Given the depth of fill required behind the berthing walls (greater than 11.5 m deep at Millhaven and 17.5 m deep at Stella) it is recommended that the diameter of the PVC sleeves should be at least 50 mm larger than the diameter of the tie-rod.

6.9 Design and Construction Considerations

6.9.1 Excavation and Groundwater Control

It is noted that heterogeneous fills comprised of crush concrete and other materials as well as timber cribbing from the original dock structure were encountered in the boreholes advanced as part of the investigation at both the Millhaven and Stella sites. It is anticipated that existing tie-back rods/anchors will also be buried within the fills that comprise the current dock structures at the sites. These materials (along with the existing dock structure elements themselves) will have to be removed as part of the new construction and prior to the installation of the H-piles and pre-cast concrete panels forming the berthing walls to avoid encountering obstructions during construction. In addition, these existing materials will also need to be removed prior to the backfilling in the areas of the ramps and new ancillary structures to avoid the pile foundations for the new structures encountering obstructions during installation.

Above the lake level, in the current on-shore areas, excavations for new foundation construction will extend through organic clayey silt/silty clay fills and sand and gravel fills as well as into the limestone bedrock. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fills are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

The level of the water surface in Lake Ontario at the site ranged from about Elevations 74.3 m to 75.1 m at the time of the investigations. Based on the Terminal Berthing Wall drawings provided by URS, the proposed design of the Berthing Walls includes the construction of pre-cast concrete anchor blocks joined to the berthing wall pile cap by tie-rods. The proposed bottom of the pre-cast anchor blocks is estimated to be at about Elevation 73.6 m, or as much as about 1.5 m below the lake level. As such, some form of dewatering will be required to install the anchor blocks, the tie-rods and to construct the compacted granular wedges in-front of the blocks.

Although the current wall design includes the installation of a single row of cast-in-place concrete panels at the base of the berthing walls to achieve a better seal with the bedrock surface, it is likely that some amount of seepage will occur through the joints between adjacent pre-cast panels in the wall. However, the rate of seepage through the panels and through and up to the top of the granular backfill (at about elevation 73.6 m prior to the anchor block installation) may be possible to be controlled by pumping from a number of properly filtered sumps behind the berthing walls. This will need to be confirmed at detail design. It is however anticipated that a Permit-to-Take-Water (PTTW) will be required for control of the water seepage through the berthing walls at the sites.

6.9.2 Temporary Protection Systems

It is our understanding that the current ferry service is to remain in operation during the construction of the expanded ferry terminals. In order to accomplish this, the construction will need to be carried out in a series of stages to avoid conflicts with the existing service.

To facilitate the construction of the new terminals in stages, some form of temporary protection systems will be required, the details of which will need to be determined at the detail design stage.

The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 *(Construction Specification for Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that the adjacent structures, ramps and other critical components of the existing terminals can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor.

6.9.3 Bedrock Socket Formation for Rock Socketed Pile Foundations

In order to accommodate the proposed H-pile sections forming the berthing wall ‘soldier piles’ as well as the H-piles supporting the ancillary structures and provide adequate space for installation of the tremie concrete, the diameter of the bedrock sockets will need to be at least 0.6 m. The limestone bedrock is weak to very strong (with unconfined compressive strengths typically in the range of about 25 MPa to 75 MPa), and so the rock sockets will have to be advanced into the bedrock by use of a rotary drilling system and down-the-hole (DTH) hammer such as the Symmetrix drilling system or equivalent.

A Non-Standard Special Provision (NSSP) will have to be included in the Contract Documents to warn the Contractor that the limestone bedrock is weak to very strong.

6.9.4 Tie Back Anchor Blocks – Alternative Design Options

Based on the limited boreholes advanced during the on-shore investigations, it appears that the elevation of the bedrock surface increases at locations further on-shore, away from the shoreline at both the Millhaven and Stella sites. Using the available information, a very preliminary estimate of the location at which the bedrock surface may rise above Elevation 73.6 m (the approximate proposed bottom of the anchor blocks) has been carried out, the results of which are presented on the sketches shown in Appendix F. As can be seen from this preliminary assessment, it appears that there is a risk that a large number of the anchor blocks for the portions of the berthing walls located close to the existing shoreline will not be able to be installed per the current design.

Given the above, the following design alternatives could be considered for these areas of the sites, subject to additional drilling at the detail design stage to confirm the bedrock surface elevation in these areas:

- Steepen the angle of the tie-rods and convert them to tie-back anchors socketed into the limestone bedrock. An unfactored grout-to-rock bond value of 1,500 kPa could be used for preliminary design of the anchors into sound limestone bedrock.
- Replace the current anchor blocks with short sections of steel H-piles embedded in 0.6 m diameter rock sockets filled with concrete. The equipment required to install these types of ‘grouted dead-men’ anchors would be essentially the same as that required to install the rock socketed H-piles in the other areas of the site, so efficiencies in the costs could be realized.
- Limit the extent of the berthing walls to those areas where the bedrock surface is deep enough to accommodate the installation of the anchor blocks per the current design. In the other areas, change the design to a rock fill slope constructed at 1.25H:1V. It is noted that some additional investigation may be required to confirm the feasibility of this design from a stability perspective, however, any unsuitable foundation soils, if encountered, could be sub-excavated and replaced as necessary.

6.9.5 Potential for Migration of Soil Particles Through Panel Wall

As noted previously, it is likely that some amount of seepage will occur through the joints between adjacent pre-cast panels in the berthing wall. Depending on the magnitude and frequency of fluctuations in the lake level adjacent to the berthing wall, changes in the water level from the front face to the rear face of the wall could result in the migration of fine particles from the backfill, through the panel joints. Over time, this migration could lead to the formation of voids behind the wall and eventually settlements at the ground surface/paved areas.

In order to minimize the chance of this phenomenon occurring, it is recommended that an appropriate geotextile filter fabric be installed behind the wall to limit the migration of fine grained particles.

6.9.6 Vibration Monitoring During Pile Installation

Where deep foundations are installed, pile driving and rock socket drilling (likely with the use of a down-the-hole hammer to break up the bedrock) will be required. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered acceptable for adjacent structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by DTH drilling and pile driving activities will reach this threshold level and, therefore, vibration monitoring for the existing terminal structures may not be required during construction at the sites. However, the requirements for vibration monitoring will have to be assessed at the detail design stage.

6.9.7 Uplift of Buried Sewage Holding Tanks

Given the anticipated high groundwater level at the location of the Sewage Tanks relative to the proposed underside of tank elevation, the Factor of Safety against buoyancy for an empty tank should be confirmed at detailed design. Based on an assumed minimum concrete tank wall thickness of 150 mm and a maximum groundwater level at Elevation 75.0 m, the Factor of Safety against buoyancy is estimated to be approximately 1.5 assuming the top of the sewage holding tanks is at about finished grade.

6.10 Scope of Work Required for Detail Design

Additional detail foundation investigation will be required at the ferry terminal sites in order to obtain information to better define changes in the subsurface conditions, in particular the elevation of the bedrock surface and to confirm the design recommendations provided in this report.

It is our understanding that the Detail Design of this project will be administered as a Design Build contract. The terms of reference for the Design Build RFP should outline the requirements for NSSPs that will need to be incorporated into the contract documents as indicated in Section 6.9 as well as the recommendations for additional foundation investigation as indicated below.

The following scope of work is recommended at a minimum for detail design foundation investigation:

- **Along the Off-Shore Berthing Walls Alignments (at Millhaven and Stella) -** additional boreholes (with rock coring) should be carried out at locations between the existing boreholes, and in areas where existing boreholes are not located within the footprint of the proposed terminal due to changes in the dock alignment following completion of the borehole investigation, to confirm the variation in the bedrock surface elevation as well as better define the extents and volumes of organics or other unsuitable materials that will need to be removed as part of construction. The number of boreholes required to be determined by the Design-Build Contractor.
- **At the Passenger Amenity Building at Millhaven** – a minimum of two (2) additional boreholes (with rock coring) on the north and south sides of the footprint of the proposed structure are recommended to identify the elevation and variation in bedrock surface in this area to confirm the preferred foundation alternative and assess the length of piles that will be required to support the structure.
- **At the Passenger Amenity Building Facility at Stella** – a series of test pits are recommended on the south side of the footprint of the proposed structure to identify the elevation of the shallow bedrock surface and confirm the founding stratum for the building footings and the sewage tank. This investigation is also recommended in order to better quantify the volume of bedrock excavation that will be required in this area.
- **Along the Near-Shore Berthing Wall Alignments (at Millhaven and Stella)** – a series of shallow boreholes and/or test pits is recommended to identify the elevation of the bedrock surface as well as the composition of the overburden in these areas to confirm the design alternatives for anchoring the proposed berthing walls or for changing the design and reducing the length of berthing walls by constructing rock fill slopes in these areas.

Soil and Groundwater aggressivity testing should be carried out at the detail design stage to determine the requirements for corrosion protection on exposed steel (H-Piles, tie-rods, etc.) and to determine appropriate cement types.

7.0 CLOSURE

This Foundation Design Report was prepared by Matthew Kelly, P.Eng., and reviewed by Mr. Paul Dittrich, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
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MARCH 24, 2014
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F. J. HEFFERNAN
March 24, 2014
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MWK/JPD/FJH/jl

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Ontario Provincial Standard Specifications (OPSS)

OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1002	Material Specification for Aggregates - Concrete
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation Frost Depths for Southern Ontario

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP105S21	Amendment to OPSS 501 – Construction Specification for Construction

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – Berthing Walls

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Steel H-piles (socketed into bedrock) with Concrete Lagging Panels	• Feasible and preferred option.	• High axial geotechnical resistance at ULS and geotechnical reaction at SLS for piles socketed into bedrock. • High lateral capacity given H-piles socketed into bedrock. • H-pile cross-section readily accommodates installation of concrete lagging panels.	• Drilling of temporary liners required in order to create socket in bedrock. • Potential for encountering obstructions in existing fill during drilling of temporary liners.	• DTH hammer drilling available for advancing liners and creating 0.6 m diameter socket into bedrock. • Conventional construction techniques for installing H-piles and placing tremie concrete in rock socket.	• Low relative cost for H-piles. • Additional costs for advancing temporary liners and drilling rock sockets with DTH hammer. • Additional costs for placing tremie concrete in rock sockets.
Continuous Caisson Wall, 0.6 m to 1.2 m diameter (socketed into bedrock)	• Feasible.	• Very high axial geotechnical resistance at ULS and geotechnical reaction at SLS for caissons socketed into bedrock. • Very high lateral capacity given large cross-section of caisson and given that caissons socketed into bedrock. • Adjacent caissons form contiguous wall as construction proceeds.	• Potential for encountering obstructions in existing fill during drilling of caissons. • Permanent steel liners required for caisson construction through water. • Potential for difficulty creating socket in bedrock for larger diameter caissons.	• Specialized construction equipment and methods for advancing caissons through overburden and drilling sockets into bedrock. • DTH hammer drilling available for caissons up to 0.75 m in diameter.	• Very high cost per m length of wall. • Additional costs for permanent steel liners. • Additional costs for specialized equipment especially for caisson diameters larger than 0.75 m.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Steel Sheet Pile Walls (driven to refusal on bedrock)	• Not feasible.	• Steel sheet piles form contiguous and interlocking wall as construction proceeds. • High axial geotechnical resistance at ULS and geotechnical reaction at SLS for piles driven to refusal on bedrock.	• Very low or negligible lateral resistance of driven sheet piling at many locations due to thinness of overburden and inability to toe sheet piles into bedrock. • Potential for encountering obstructions in existing fill during driving of sheet piles.	• Conventional construction equipment and methods for driven steel sheet pile installation.	• Low relative cost alternative for piling.
Steel H-piles or Steel Pipe (tube) Piles (driven to refusal on bedrock) with Concrete Lagging Panels	• Not feasible.	• High axial geotechnical resistance at ULS and geotechnical reaction at SLS for piles driven to refusal on bedrock.	• Very low or negligible lateral resistance of driven piles at many locations due to thinness of overburden and inability to toe driven piles into bedrock. • Potential for encountering obstructions in existing fill during piling. • Special detail required to affix concrete panels to steel pipe pile option.	• Conventional construction equipment and methods for driven steel H-piles and pipe piles.	• Low relative cost alternative for piling. • Additional costs for special detail to affix concrete panels to steel pipe piles.
Conventional Cantilever Retaining Wall with Strip footings founded on bedrock	• Not feasible.	• High axial geotechnical resistance at ULS and geotechnical reaction at SLS for footings on bedrock.	• Not practical/possible to construct off-shore (underwater); would require cofferdam and full dewatering of terminal area. • Shear key or anchors in bedrock may be required to develop sufficient lateral resistance to support berthing walls.	• Difficult and impractical to construct due to depth of water and depth to bedrock.	• Very high construction costs as cofferdam and dewatering would be required.

TABLE 2 – COMPARISON OF FOUNDATION ALTERNATIVES –Ramps and Backup Ramps

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Steel H-piles (socketed into bedrock)	• Feasible and preferred option.	• High axial geotechnical resistance at ULS and geotechnical reaction at SLS for piles socketed into bedrock. • High lateral capacity given H-piles socketed into bedrock. • Similar equipment and construction techniques as preferred option for berthing walls.	• Drilling of temporary liners required in order to create socket in bedrock. • Potential for encountering obstructions in existing fill during drilling of temporary liners.	• DTH hammer drilling available for advancing liners and creating 0.6 m diameter socket into bedrock. • Conventional construction techniques for installing H-piles and placing tremie concrete in rock socket.	• Low relative cost for H-piles. • Additional costs for advancing temporary liners and drilling rock sockets with DTH hammer. • Additional costs for placing tremie concrete in rock sockets. • Efficiencies and cost savings possible if same option adopted for berthing walls.
Drilled Steel Casings (0.40 m to 0.6 m diameter) or Micropiles (up to 0.3 m diameter)	• Feasible.	• Relatively high axial geotechnical resistance at ULS and geotechnical reaction at SLS for piles socketed into bedrock (dependent on length of rock socket). • More readily able to penetrate obstructions if encountered in existing fill.	• Lateral capacity dependent on diameter, amount of steel in cross-section and configuration of piles.	• Specialized drilling and grouting equipment for advancing drilled steel casings and constructing micropiles.	• Higher relative costs than H-pile option. • Additional costs for mobilization of micropile or steel casing drilling equipment.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Caissons (0.6 m to 0.75 m diameter) advanced to top of limestone bedrock	• Feasible.	• Higher axial capacities than for driven steel piles options, requiring fewer elements per ramp structure.	• Potential for encountering obstructions in existing fill during drilling of caissons. • Temporary liners required for caisson construction through granular soils below the water level. • Cleaning of caisson bases and inspection of bases required to ensure a clean base/top of rock prior to concreting. • Inspection difficult below water; requires use of underwater camera.	• Conventional construction methods for caisson foundations.	• Higher cost relative to other pile foundation options. • Additional costs for temporary liners. • Additional costs for mobilization of caisson drilling equipment.
Steel H-piles or Steel Pipe (tube) Piles (driven to refusal on bedrock)	• May not be feasible depending on lateral capacities required.	• High axial geotechnical resistance at ULS and geotechnical reaction at SLS for piles driven to refusal on bedrock.	• Potential for encountering obstructions in existing fill during piling. • Very low or negligible lateral resistance for piles located adjacent to berthing walls due to inability to toe driven piles into bedrock.	• Conventional construction equipment and methods for driven steel H-piles and pipe piles.	• Low relative cost alternative for piling. • Additional costs for mobilization of pile driving equipment.
Spread/strip footings founded on compacted granular pad	• Feasible (but not preferred).	• Relatively straight forward construction.	• Lower ULS and SLS values as compared with footings on bedrock. • High risk of differential settlement due to difficulties associated with fill placement and compaction below water level and variations in depth to bedrock/thickness of fill	• Conventional construction techniques.	• Lowest relative cost of all foundation options. • Additional costs for dewatering to allow granular pad construction.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
			required below structure footprints. • Temporary dewatering below the adjacent lake level would be required for construction/compaction of granular pads.		
Spread/Strip footings founded on bedrock	• Not feasible.	• High axial geotechnical resistance at ULS and geotechnical reaction at SLS for footings on bedrock.	• Not practical/possible to construct off-shore (underwater); would require cofferdam and full dewatering of terminal area.	• Difficult and impractical to construct due to depth of water and depth to bedrock.	• Very high construction costs as cofferdam and dewatering would be required.

TABLE 3 – COMPARISON OF FOUNDATION ALTERNATIVES – Ancillary Structures at Millhaven and Stella Terminals

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Spread/strip footings founded on Bedrock	• Feasible and preferred option for support of Sewage Holding Tanks at both sites and Crew Facilities at Stella site. • Not feasible option for support of Mechanical & Storage Buildings at both sites and Crew Facilities at Millhaven site as bedrock too deep.	• High geotechnical resistance at ULS and geotechnical reaction at SLS for footings on bedrock. • Lower risk of settlement than for shallow foundations founded on compacted granular pads.	• Significant bedrock excavation (to a depth of up to about 2.7 m below bedrock surface) will be required to achieve adequate frost protection and to fully bury Sewage Holding Tanks. • Groundwater seepage anticipated at Tanks as excavations extend below adjacent lake level; pumping from filtered sumps is expected to provide adequate groundwater control.	• Conventional excavation and construction techniques. • Some bedrock excavation through slightly weathered to fresh weak to very strong limestone will be required.	• Estimated cost is \$600/m^3 for construction of spread footings • Additional cost for bedrock excavation at Tanks required.
Spread/strip footings founded on compacted granular pad	• Feasible option (but not preferred) for support of Mechanical & Storage Buildings at both sites and Crew Facilities at Millhaven site. • Not feasible for support of Sewage Holding Tanks at both sites or Crew Facilities at Stella site.	• Relatively straight forward construction.	• Lower ULS and SLS values as compared with footings on bedrock. • High risk of differential settlement due to difficulties associated with fill placement and compaction below water level and variations in depth to bedrock/thickness of fill required below structure footprints. • Temporary dewatering below the adjacent lake level would be required for construction/compaction of granular pads.	• Conventional construction techniques.	• Similar cost to shallow foundations founded on bedrock. • Additional costs for dewatering to allow granular pad construction.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Steel H-piles or Steel Pipe (tube) Piles driven to refusal on limestone bedrock	• Feasible and preferred option for support of Mechanical & Storage Buildings at both sites and Crew Facilities at Millhaven site. • Not feasible for support of Sewage Holding Tanks at both sites or Crew Facilities at Stella site.	• Pile foundations minimize differential settlement within each structure and between adjacent structures/berthing walls. • Pile foundations will limit interaction/loading from structures onto adjacent berthing walls. • Potentially no dewatering required since pile cap construction likely above current lake level. • Less compactive effort required for granular backfill below structures.	• Potential for encountering obstructions in existing fill during pile driving. • Vibration monitoring may be required for piles driven through new or existing fill to refusal on bedrock. • Variable pile lengths across some structure footprints and possibly short pile lengths at some locations.	• Conventional construction methods for driven steel H-pile or pipe (tube) pile foundations.	• Estimated cost is approximately $250/m length for pile installation and $600/m^3 for pile cap construction.
Caissons (0.6 m diameter) advanced to top of limestone bedrock	• Feasible (but not preferred option) for support of Mechanical & Storage Buildings at both sites and Crew Facilities at Millhaven site. • Not feasible for support of Sewage Holding Tanks at both sites or Crew Facilities at Stella site.	• Higher axial capacities than for driven steel piles options, requiring fewer elements per structure. • Pile foundations minimize differential settlement within each structure and between adjacent structures/berthing walls. • Potentially no dewatering required since pile cap construction likely above current lake level. • Less compactive effort required for granular backfill below structures.	• Temporary liners required for caisson construction through granular soils below the water level. • Cleaning of caisson bases and inspection of bases required to ensure a clean base/top of rock prior to concreting. • Inspection difficult below water; requires use of underwater camera.	• Conventional construction methods for caisson foundations.	• Estimated cost is approximately $1000/m length for caisson installation and $600/m^3 for pile cap construction. • Additional costs for temporary liners.

MINISTRY OF TRANSPORTATION, ONTARIO

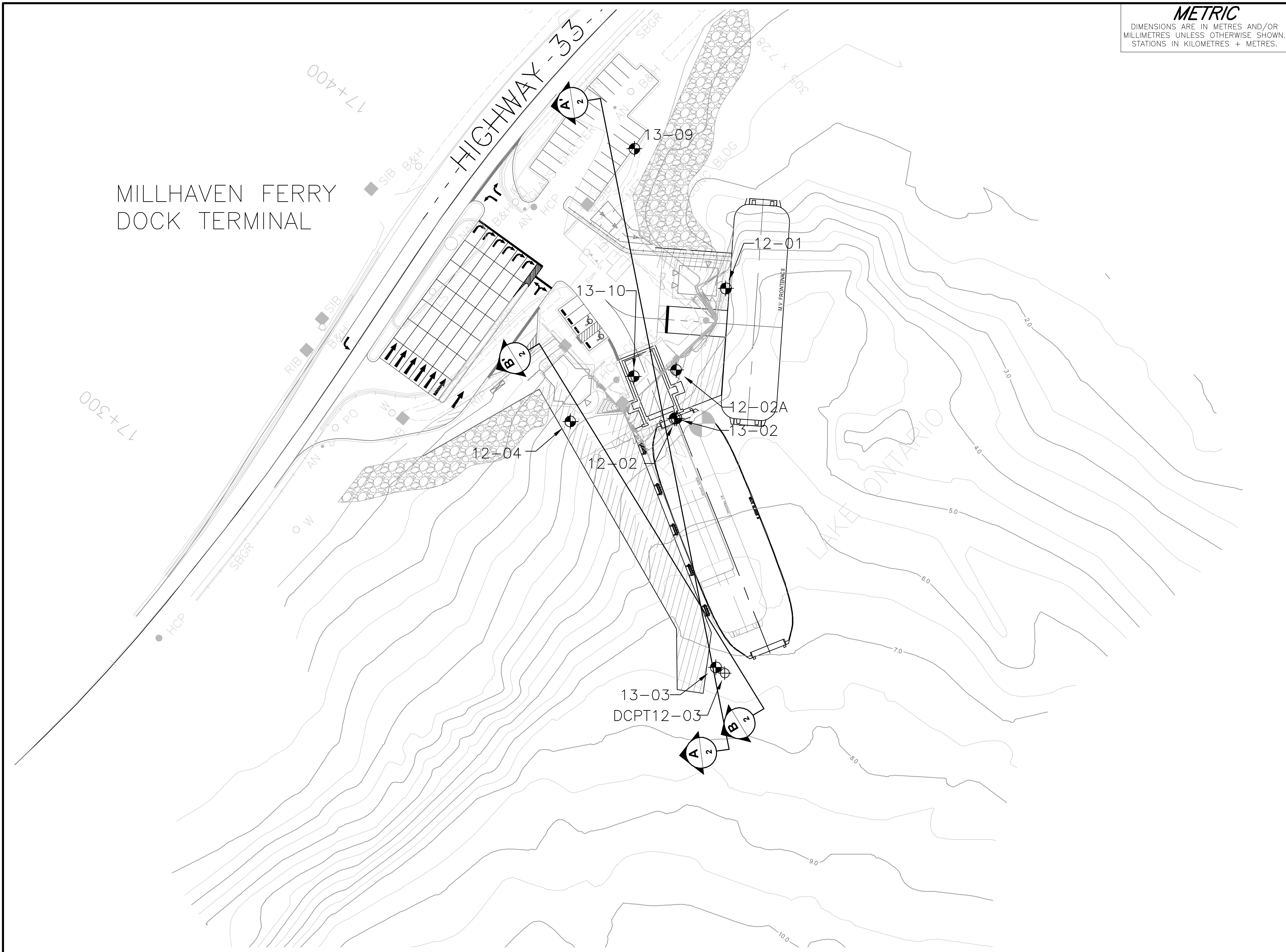
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
G.W.P. No. 4067-09-00

SHEET

AMHERST ISLAND
MILLHAVEN TERMINAL CONVERSION
BOREHOLE LOCATION PLAN

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

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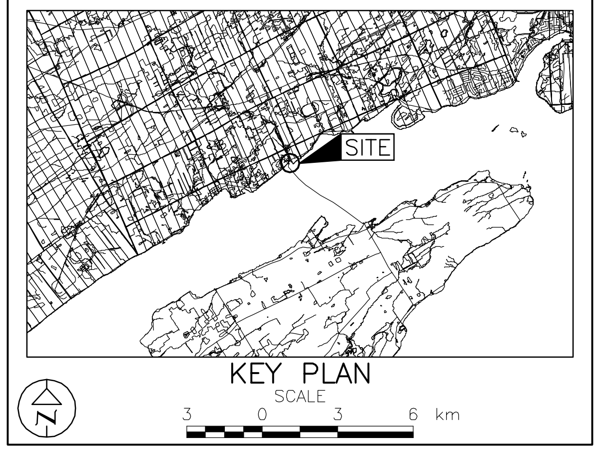

KEY PLAN
SCALE
3 0 3 6 km

LEGEND

▨	Proposed New Ferry Dock
—4.0—	Water Depth Contours (0.5 m Interval)
◓	Borehole Investigation
⊕	Dynamic Cone Penetration Test

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
12-01	74.4	4894798.8	285574.5
12-02	74.4	4894766.6	285562.3
12-02A	74.3	4894778.6	285562.2
12-04	74.4	4894765.9	285536.0
13-02	75.1	4894766.6	285561.8
13-03	75.1	4894704.9	285572.0
13-09	77.8	4894833.5	285551.9
13-10	76.3	4894777.0	285551.7

DCPT CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
DCPT12-03	74.3	4894703.5	285574.3

NOTES

Water depths shown are based on chart datum elevation of 74.2 m in Lake Ontario, as reported by the Canadian Hydrographic Services, Fisheries and Oceans Canada.

REFERENCE

Base plans provided in digital format by URS, file no. PLAN.dwg, received April 02, 2012 and CONCEPTS-TO GOLDER 10 JULY 2012.dwg, received July 10, 2012. X-Design - Millhaven.dwg and X-Design - Stella.dwg, received August 29, 2013

NO.	DATE	BY	REVISION
1	**14/03/24**	**JFC**	**Change in terminal layout and South end of proposed dock alignment by URS**

Geocres No. 31C-223

HWY.		PROJECT NO. 11-1111-0115	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Mar. 2014	SITE:
DRAWN: DD/JFC	CHKD. JPD	APPD. FJH	DWG. 1

PLOT DATE: March 24, 2014
FILENAME: T:\Projects\2011\11-1111-0115 (URS, Amherst Island)\-DC-Preliminary Report\1111110115DC001.dwg

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CONT No.
G.W.P. No. 4067–09–00

SHEET

AMHERST ISLAND
MILLHAVEN TERMINAL CONVERSION
SOIL STRATA

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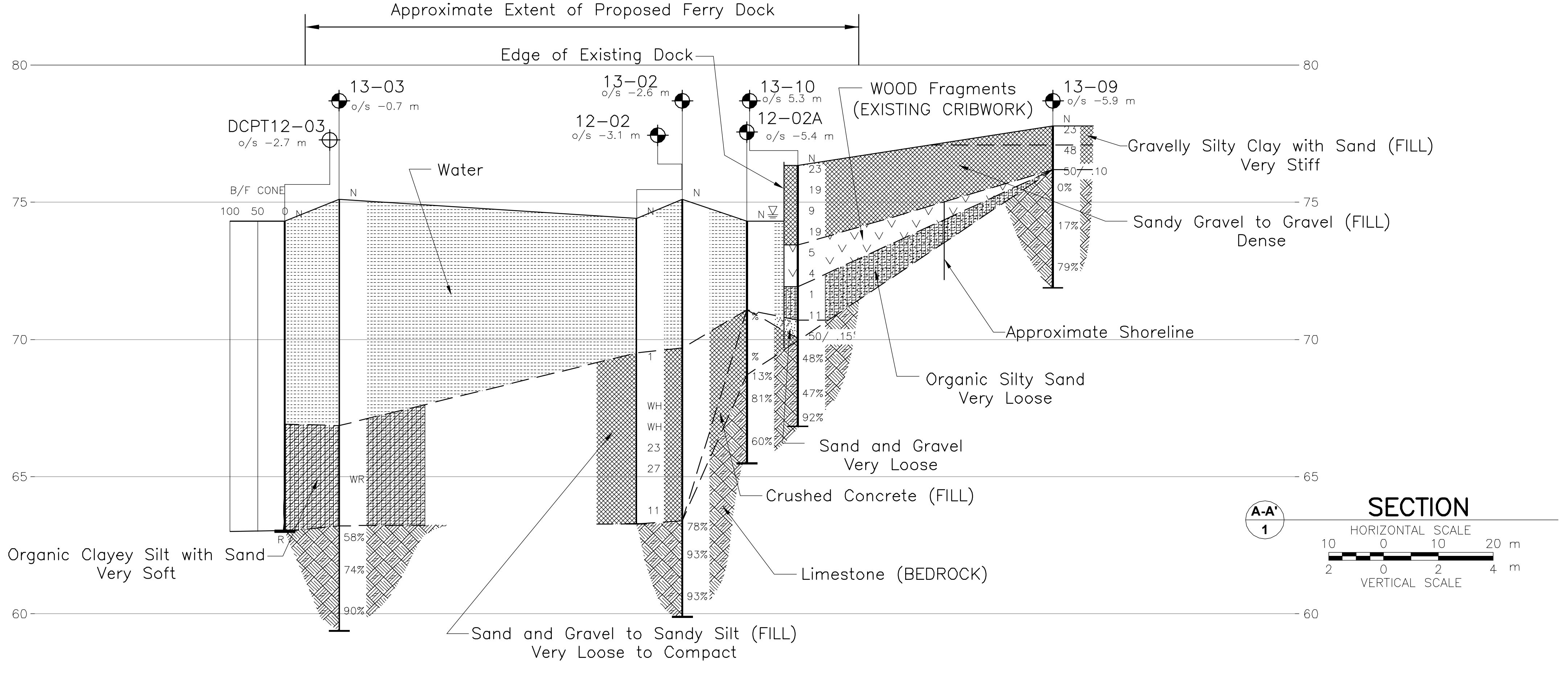
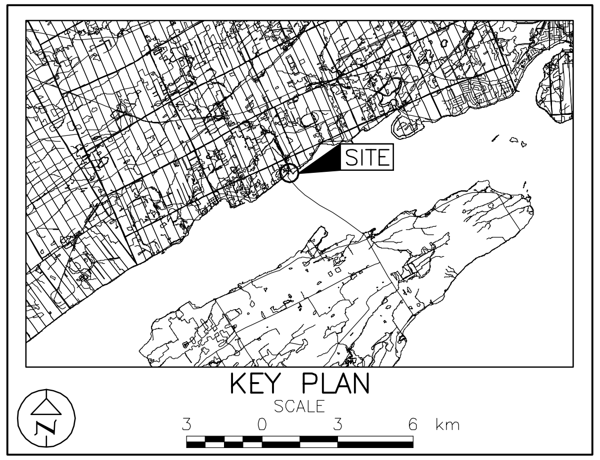




LEGEND

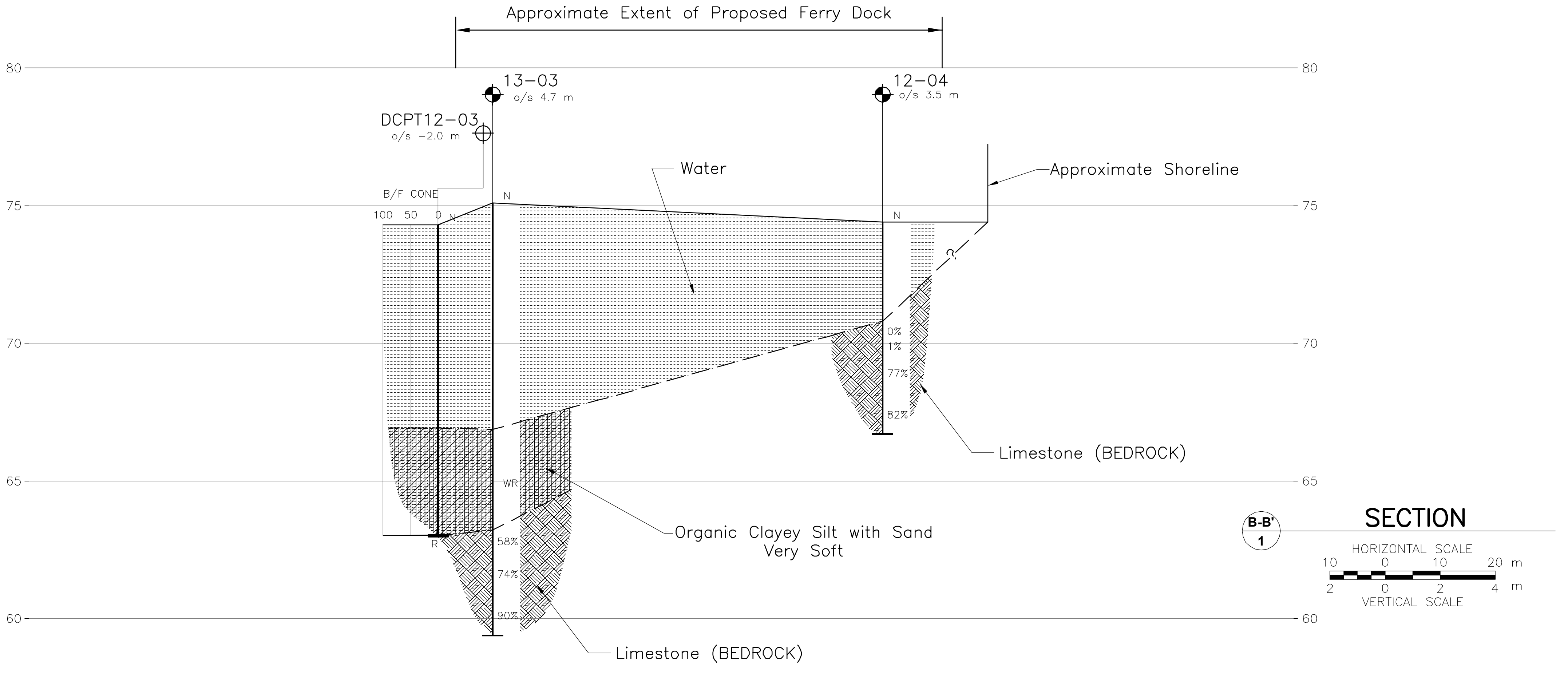
⊕	Borehole Investigation
⊕	Dynamic Cone Penetration Test
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
R	Refusal on inferred Bedrock

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
12–01	74.4	4894798.8	285574.5
12–02	74.4	4894766.6	285562.3
12–02A	74.3	4894778.6	285562.2
12–04	74.4	4894765.9	285536.0
13–02	75.1	4894766.6	285561.8
13–03	75.1	4894704.9	285572.0
13–09	77.8	4894833.5	285551.9
13–10	76.3	4894777.0	285551.7

DCPT CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
DCPT12–03	74.3	4894703.5	285574.3



REFERENCE

Base plans provided in digital format by URS, file no. PLAN.dwg, received April 02, 2012 and CONCEPTS–TO GOLDER 10 JULY 2012.dwg, received July 10, 2012.

NO.	DATE	BY	REVISION

Geocres No. 31C–223

HWY.		PROJECT NO. 11–1111–0115	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Nov. 2013	SITE:
DRAWN: DD/JFC	CHKD. JPD	APPD. FJH	DWG. 2

PLOT DATE: January 10, 2014
FILENAME: T:\Projects\2011\11-1111-0115 (URS, Amherst Island)\-DC-Preliminary Report\1111110115DC002.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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

CONT No.
G.W.P. No. 4067-09-00

SHEET

AMHERST ISLAND
STELLA TERMINAL CONVERSION
BORHOLE LOCATION PLAN

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

PLAN

SCALE
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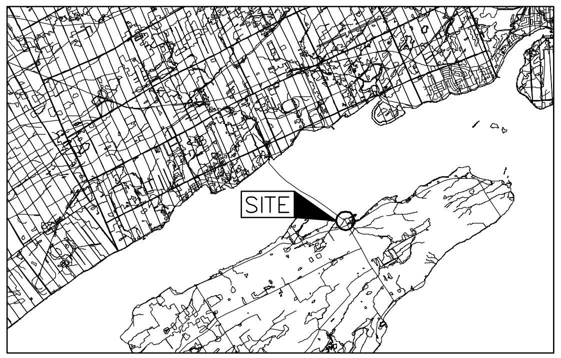


KEY PLAN

SCALE
3 0 3 6 km

LEGEND

- ▨ Proposed New Ferry Dock
- —4.0— Water Depth Contours (0.5 m Interval)
- ⊕ Borehole Investigation
- ⊕ Dynamic Cone Penetration Test

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
12-07	74.2	4892481.7	288555.8
12-08	74.4	4892485.0	288581.7
13-05_13-05A	75.0	4892500.9	288505.2
13-06	74.9	4892502.8	288538.4
13-11	76.4	4892471.8	288553.2
13-12	76.4	4892412.9	288585.6

NOTES

Water depths shown are based on chart datum elevation of 74.2 m in Lake Ontario, as reported by the Canadian Hydrographic Services, Fisheries and Oceans Canada.

REFERENCE

Base plans provided in digital format by URS, file no. PLAN.dwg, received April 02, 2012 and CONCEPTS-TO GOLDER 10 JULY 2012.dwg, received July 10, 2012. X-Design – Millhaven.dwg and X-Design – Stella.dwg, received August 29, 2013

NO.	DATE	BY	REVISION

Geocres No. 31C-223

HWY.		PROJECT NO. 11-1111-0115	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Nov. 2013	SITE:
DRAWN: DD/JFC	CHKD. JPD	APPD. FJH	DWG. 3

PLOT DATE: January 10, 2014
FILENAME: T:\Projects\2011\11-1111-0115 (URS, Amherst Island)\-DC-Preliminary Report\1111110115DC003.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

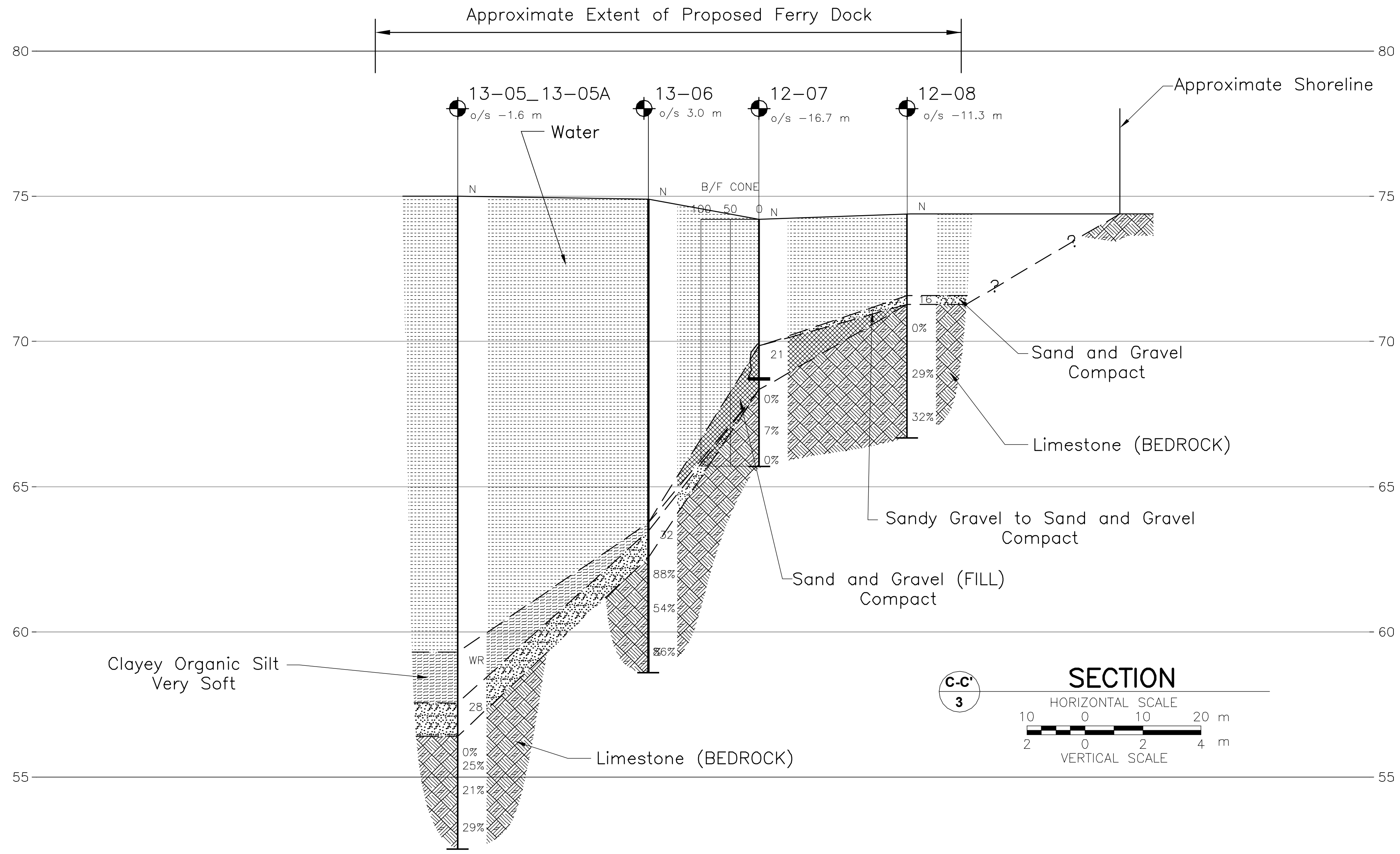
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
G.W.P. No. 4067–09–00

SHEET

AMHERST ISLAND
STELLA TERMINAL CONVERSION
SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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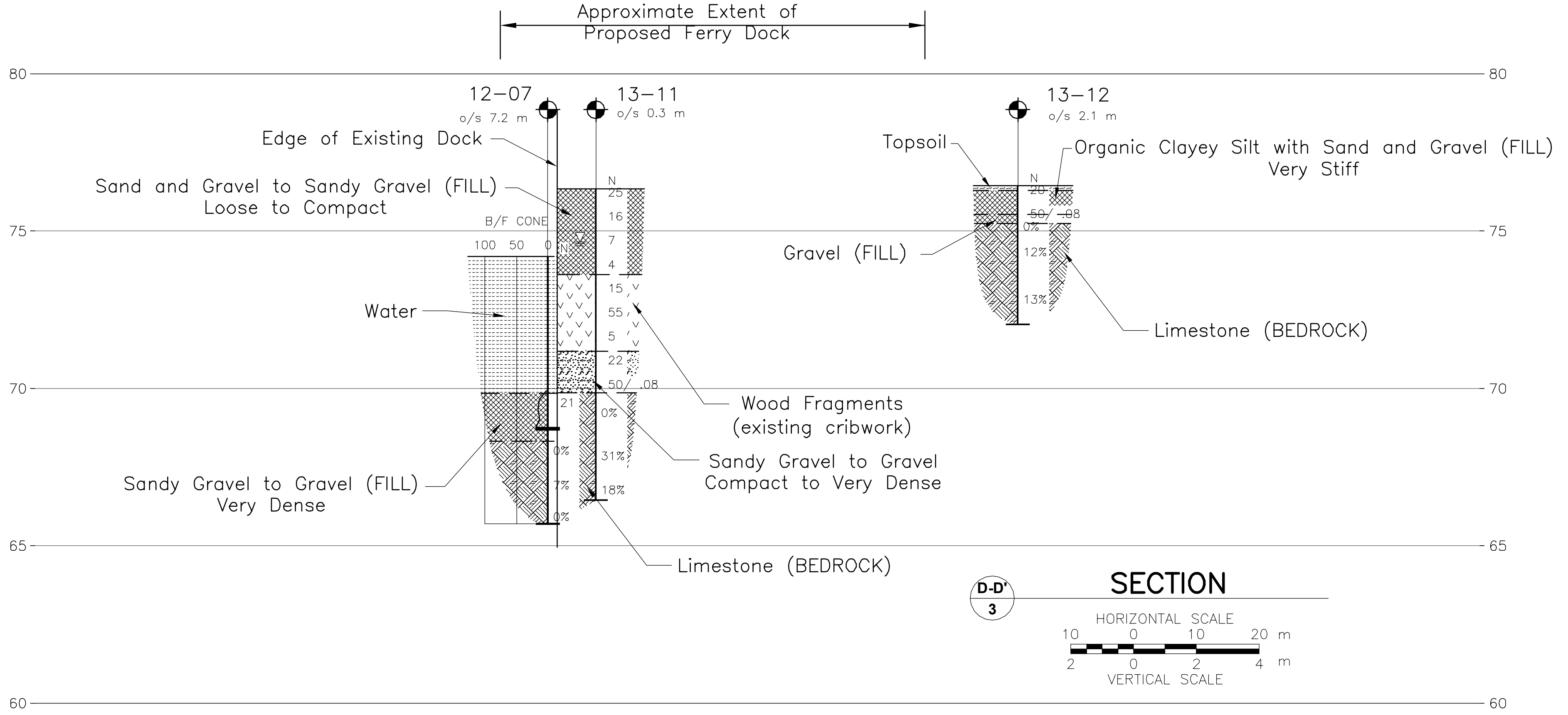

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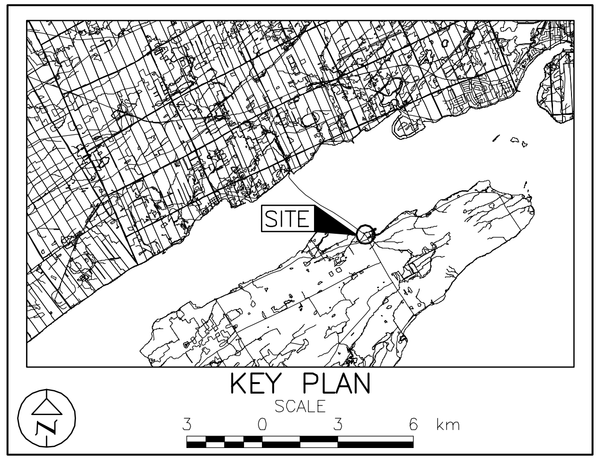


KEY PLAN

LEGEND

●	Borehole Investigation
⊕	Dynamic Cone Penetration Test
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 on inferred Bedrock

BOREHOLE CO–ORDINATES			
No.	ELEVATION	NORTHING	EASTING
12–07	74.2	4892481.7	288555.8
12–08	74.4	4892485.0	288581.7
13–05_13–05A	77.2	4892500.9	288505.2
13–06	77.1	4892502.8	288538.4
13–11	76.4	4892471.8	288553.2
13–12	76.4	4892412.9	288585.6

NO.	DATE	BY	REVISION

Geocres No. 31C–223

HWY.		PROJECT NO. 11–1111–0115	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: 12/18/2012	SITE:
DRAWN: MR/DD	CHKD. JPD	APPD. FJH	DWG. 4

PLOT DATE: January 10, 2014
FILENAME: T:\Projects\2011\11–1111–0115 (URS, Amherst Island)\–DC–Preliminary Report\1111110115DC004.dwg

APPENDIX A

Record of Boreholes and Drillholes – Millhaven Terminal

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
FoS	factor of safety

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

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
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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, 60º 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) Non-Cohesive (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.

V. MINOR SOIL CONSTITUENTS

Per cent 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 (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

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

Golder Associates

RECORD OF BOREHOLE No 12-01

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
LOCATION N 4894798.8 ;E 285574.5
ORIGINATED BY MS/DM
DIST
HWY 33
BOREHOLE TYPE CME-55 Barge Mounted, NW Casing
COMPILED BY MWK
DATUM Geodetic
DATE September 12, 2012
CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
74.4	WATER SURFACE													
0.0	Water						74 73 72 71							
70.1 4.3	SAND and GRAVEL, some silt, trace clay, containing organics and shell fragments Very loose Brown Wet		1	SS	2		70 69		43.5		47	35	16	2
68.7 5.7	LIMESTONE (BEDROCK) Bedrock cored from 5.7 m to 9.2 m depth. Refer to Record of Drillhole for bedrock coring details.		1	NQ	REC 79%						RQD = 0%			
			2	NQ	REC 100%		68 67				RQD = 58%			
			3	NQ	REC 83%		66				RQD = 22%			
65.2 9.2	END OF BOREHOLE													

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 12-01

SHEET 1 OF 1

LOCATION: N 4894798.8 ;E 285574.5

DRILLING DATE:

DATUM: Geodetic

DRILL RIG: CME-55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

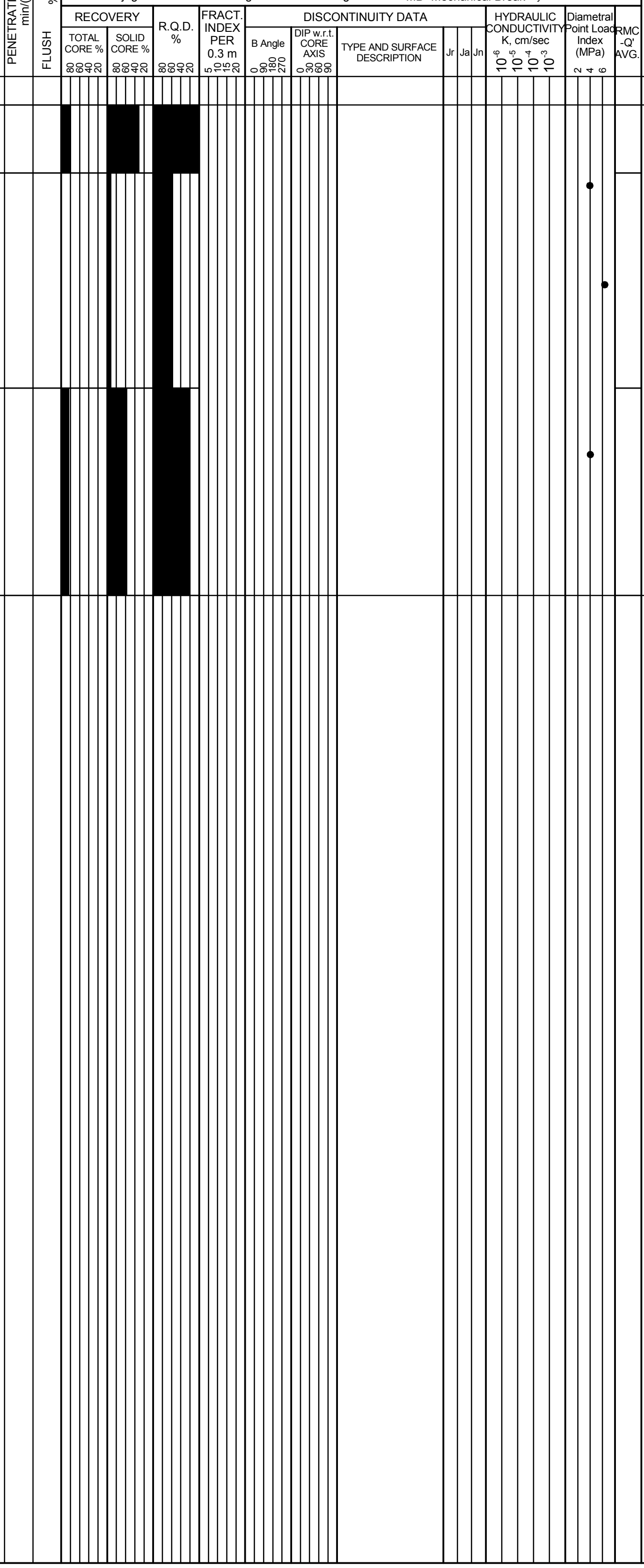
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

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.	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		68.67		
6	NQ Rock Core NW Casing	LIMESTONE, fine grained, laminated, slightly porous Slightly weathered to fresh Grey Strong to very strong		5.73	1	
					2	(Axial)
7						
8					3	UCS = 103 MPa (Axial)
9				65.21 9.19		
		END OF DRILLHOLE				
10						
11						
12						
13						
14						
15						



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MS/DM

CHECKED: JPD

RECORD OF BOREHOLE No 12-02

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115			
G.W.P. 4067-09-00	LOCATION N 4894766.6 ;E 285562.3	ORIGINATED BY MS
DIST ____ HWY 33	BOREHOLE TYPE CME-55 Barge Mounted, NW Casing	COMPILED BY MWK
DATUM Geodetic	DATE September 15, 2012	CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE — DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
74.4	WATER SURFACE										
0.0	Water										
69.5 4.9	Sand and gravel, some silt, containing wood fragments and organics (FILL) Very loose Brown Wet		1	SS	1						
			2	SS	WH			44	40	16	0
			3	SS	WH		OC = 12.0%				
66.2 8.2	Sandy silt and gravel (FILL) Compact Brown Wet		4	SS	23						
65.5 8.9	Sand and gravel, trace silt, trace clay (FILL) Compact Grey Wet		5	SS	27			52	41	5	2
			6	SS	11						
63.3 11.1	END OF BOREHOLE										

ELEVATION SCALE

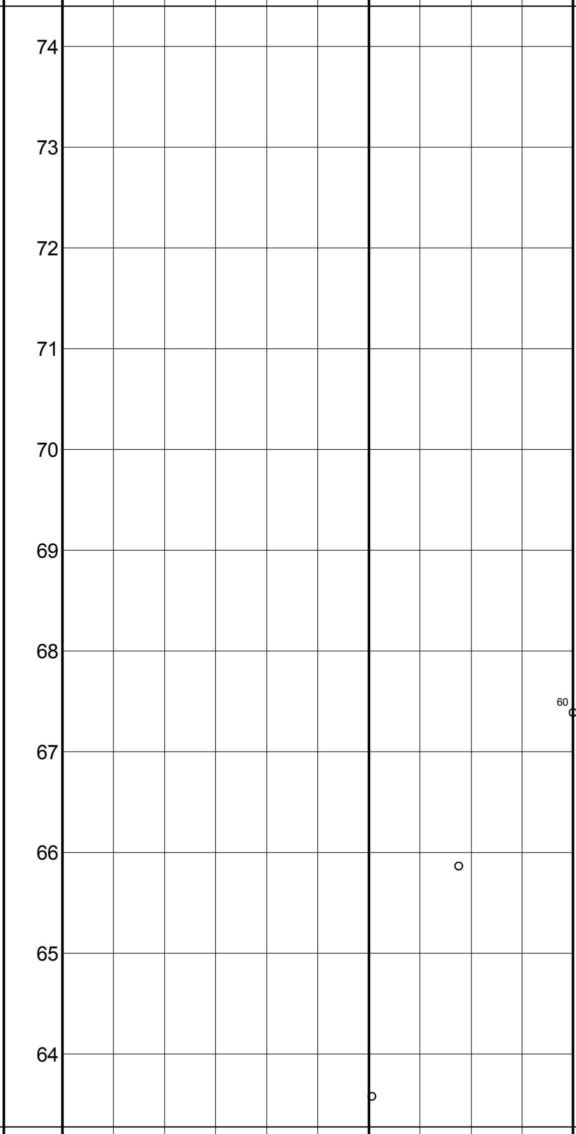
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

RECORD OF BOREHOLE No 12-02A

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
DIST
HWY 33
DATUM Geodetic
LOCATION N 4894778.6 ;E 285562.2
BOREHOLE TYPE CME-55 Barge Mounted, NW Casing
DATE September 26 to 27, 2012
ORIGINATED BY MS/DM
COMPILED BY MWK
CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	SAMPLES – TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
74.3	WATER SURFACE					
0.0	Water					
71.1 3.2	Crushed concrete (FILL)		NQ	REC 39%		
70.6 3.7	No recovery					
70.1 4.2	Crushed concrete (FILL)		NQ	REC 13%		
68.7 5.6	LIMESTONE (Bedrock) Bedrock cored from 5.6 m to 8.8 m depth Refer to Record of Drillhole for bedrock coring details	2	NQ	REC 100%		RQD = 13%
		3	NQ	REC 100%		RQD = 81%
		4	NQ	REC 82%		RQD = 60%
65.5 8.8	END OF DRILLHOLE					

STRAT PLOT

ELEVATION SCALE: 74, 73, 72, 71, 70, 69, 68, 67, 66

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

GR SA SI CL

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 12-02A

SHEET 1 OF 1

LOCATION: N 4894778.6 ;E 285562.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

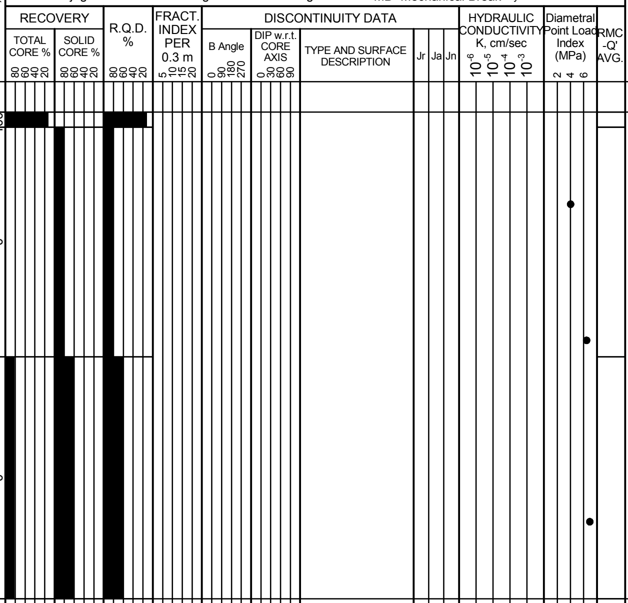
DRILL RIG: CME-55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	COLOUR % RETURN FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		68.71 5.59	2		100	
6	NQ Rock Core NW Casing	LIMESTONE, thinly to medium bedded, fine grained, slightly porous Slightly weathered to fresh Grey Strong			3		0	(Axial)
7				67.09 7.21				(Axial)
		LIMESTONE, thinly bedded, fine grained, Fresh to slightly weathered Grey Strong Clay infilling in joints			4		0	UCS = 53.5 MPa
8								(Axial)
				65.49 8.81				
9		END OF DRILLHOLE						
10								
11								
12								
13								
14								
15								



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MS/DM

CHECKED: JPD

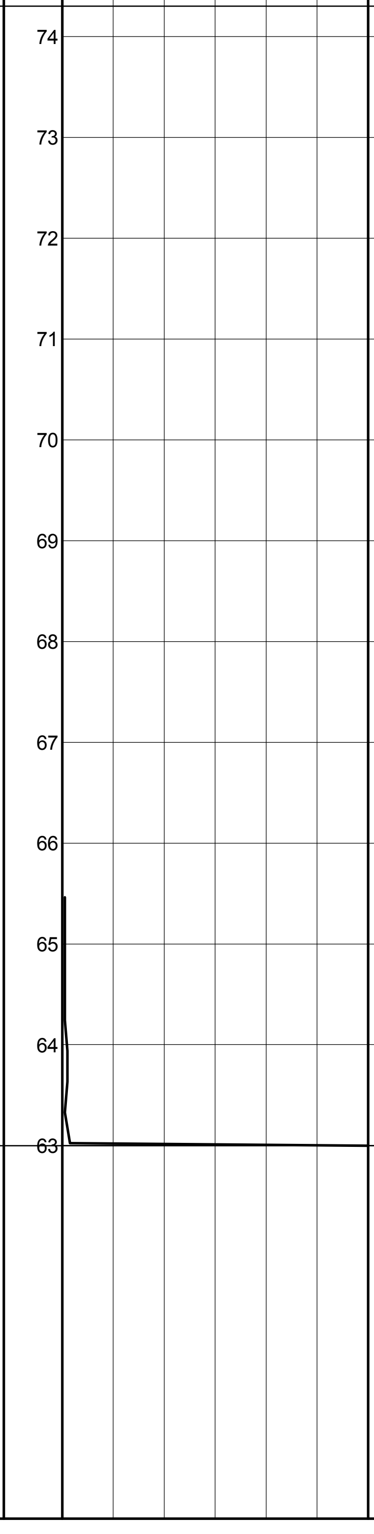
RECORD OF DCPT No DCPT12-03

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
DIST ____ HWY 33
DATUM Geodetic
LOCATION N 4894703.5 ;E 285574.3
BOREHOLE TYPE CME-55 Barge Mounted
DATE September 21, 2012
ORIGINATED BY MS
COMPILED BY MWK
CHECKED BY JPD

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
74.3	WATER SURFACE										
0.0	Water										
66.9 7.4	Very loose Overburden										
63.0 11.3	END OF DCPT DCPT BOUNCING REFUSAL ON INFERRED BEDROCK										



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

RECORD OF BOREHOLE No 12-04

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115

G.W.P. 4067-09-00

LOCATION N 4894765.9 ;E 285536.0

ORIGINATED BY MS/DM

DIST

HWY 33

BOREHOLE TYPE CME-55 Barge Mounted, NW Casing

COMPILED BY MWK

DATUM Geodetic

DATE September 13, 2012

CHECKED BY JPD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES						
74.4	WATER SURFACE									
0.0	Water					74 73 72 71				
70.8 3.6	LIMESTONE (Bedrock) Bedrock cored from 3.6 m to 7.7 m depth. Refer to Record of Drillhole for bedrock coring details.	1	NQ	REC 68%						RQD = 0%
		2	NQ	REC 78%		70				RQD = 1%
		3	NQ	REC 100%		69				RQD = 77%
		4	NQ	REC 100%		68 67				RQD = 82%
66.7 7.7	END OF BOREHOLE									

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 12-04

SHEET 1 OF 1

LOCATION: N 4894765.9 ;E 285536.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: CME-55

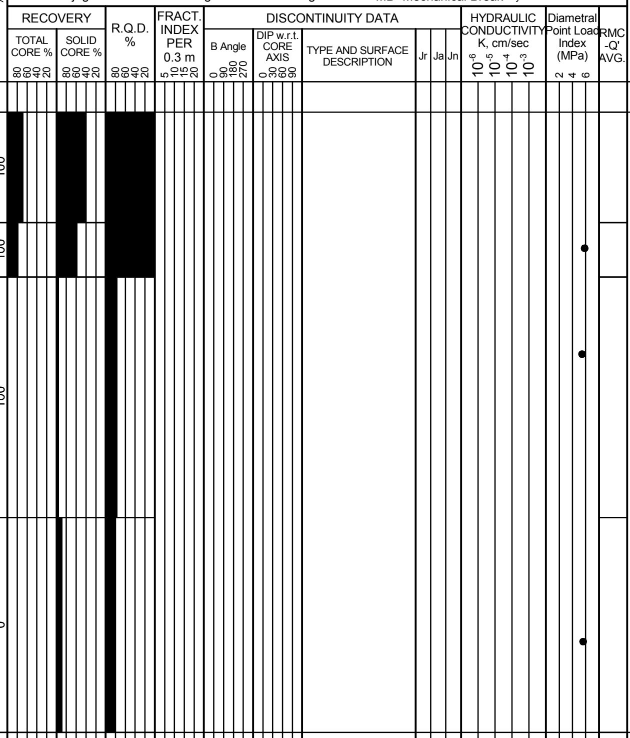
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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.	PENETRATION RATE min/(m)	COLOUR % RETURN	FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		70.80					
4	NQ Rock Core NW Casing	LIMESTONE, fine grained, laminated, slightly porous Slightly weathered to fresh Grey Medium strong to strong		3.60	1			100	
					2			100	
5					3			100	UCS = 41.3 MPa (Axial)
6									
7				66.70	4			0	(Axial)
		END OF DRILLHOLE		7.70					



4 5 6 7 8 9 10 11 12 13

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MS/DM

CHECKED: JPD

GTA-RCK 004 11-1111-0115.GPJ GAL-MISS.GDT 1/10/14

RECORD OF BOREHOLE No 13-02

SHEET 1 OF 2

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
LOCATION N 4894766.6 ;E 285561.8
ORIGINATED BY PH
DIST
HWY 33
BOREHOLE TYPE CME-75 Barge Mounted, NW Casing
COMPILED BY MWK
DATUM Geodetic
DATE August 12 and 13, 2013
CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.1	WATER SURFACE						
0.0	Water					75	
						74	
						73	
						72	
						71	
						70	
69.7 5.4	Overburden Refer to Record of Borehole 12-02 for details of overburden stratigraphy					69	
						68	
						67	
						66	
						65	
						64	
63.4 11.7	LIMESTONE (BEDROCK) Bedrock cored from 11.6 m to 15.2 m depth. Refer to Record of Drillhole 13-02 for bedrock coring details.	1	HQ	REC 100%		63	RQD = 78%
		2	HQ	REC 99%		62	RQD = 93%
		3	HQ	REC 97%		61	RQD = 93%

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

○$^{3\%}$ STRAIN AT FAILURE

Golder Associates

PROJECT 11-1111-0115

RECORD OF BOREHOLE No 13-02

SHEET 2 OF 2

METRIC

G.W.P. 4067-09-00

LOCATION N 4894766.6 ;E 285561.8

ORIGINATED BY PH

DIST

HWY 33

BOREHOLE TYPE CME-75 Barge Mounted, NW Casing

COMPILED BY MWK

DATUM Geodetic

DATE August 12 and 13, 2013

CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
59.9		▨	3	HQ			60							
15.2	END OF BOREHOLE													

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

$○^{3\%}$ STRAIN AT FAILURE

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-02

SHEET 1 OF 1

LOCATION: N 4894766.6 ;E 285561.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

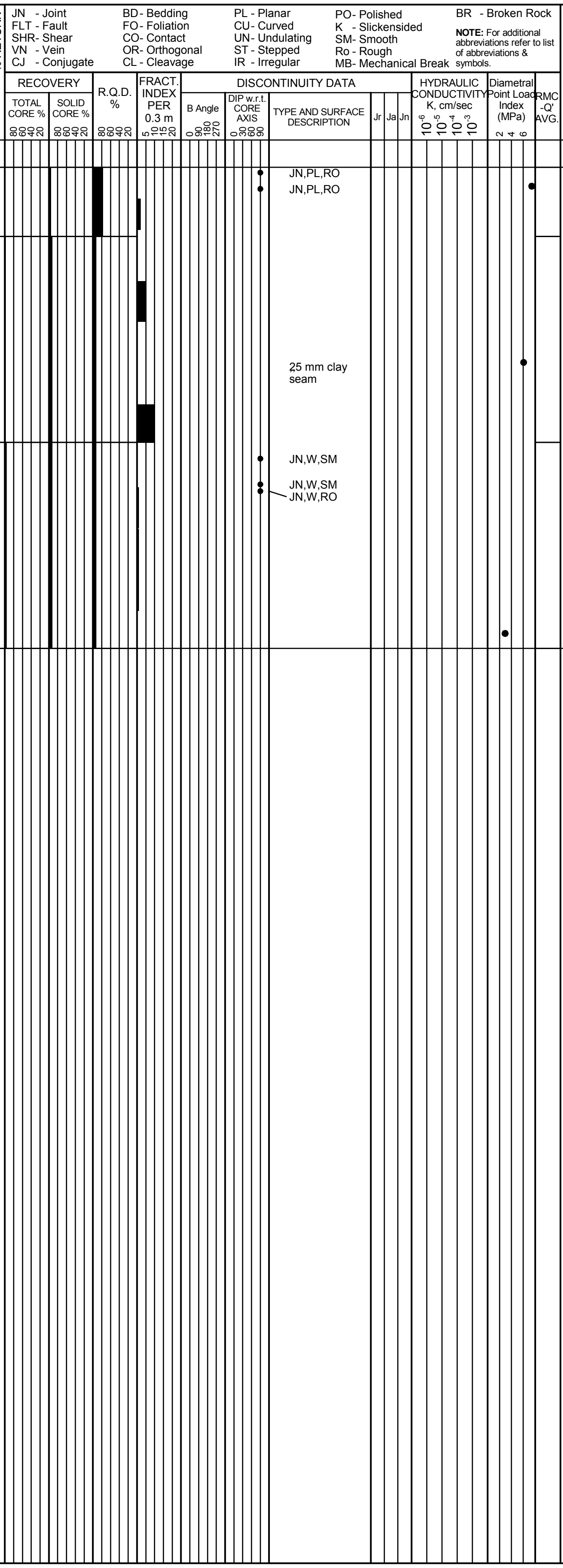
DRILL RIG: CME-75

DRILLING CONTRACTOR: Canadian Soil Drilling

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		63.44					
12	HQ Rock Core NQ Casing	LIMESTONE, fine grained slightly porous Fresh Medium to thickly bedded Grey Weak to very strong		11.66	1		100	JN,PL,RO JN,PL,RO	(Axial) UCS=64.2 MPa
13					2		50	25 mm clay seam	(Axial)
14					3		0	JN,W,SM JN,W,SM JN,W,RO	UCS=8.6 MPa
15				59.89 15.21					(Axial)
		END OF DRILLHOLE							
16									
17									
18									
19									
20									
21									



DEPTH SCALE

1 : 50

LOGGED: PH

CHECKED: JPD

RECORD OF BOREHOLE No 13-03

SHEET 1 OF 2

METRIC

PROJECT	11-1111-0115				
G.W.P.	4067-09-00	LOCATION	N 4894704.9 ;E 285572.0	ORIGINATED BY	PH
DIST	HWY 33	BOREHOLE TYPE	CME-75 Barge Mounted, NW Casing	COMPILED BY	MWK
DATUM	Geodetic	DATE	August 13, 2013	CHECKED BY	JPD

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR SA SI CL
75.1	WATER SURFACE										
0.0	Water						75				
							74				
							73				
							72				
							71				
							70				
							69				
							68				
66.9 8.2	CLAYEY ORGANIC SILT with SAND, containing wood fragments and shells Very soft Dark brown						67				
							66				
							65				O. C. = 17.9%
			1	SS	WR				197.8 ○		0 31 46 23
							64				
63.5	Sand and gravel (TILL)										
63.2 11.9	LIMESTONE (BEDROCK) Bedrock cored from 11.9 m to 15.7 m depth. Refer to Record of Drillhole 13-03 for bedrock coring details.						63				
			1	HQ	REC 98%						RQD = 58%
							62				
			2	HQ	REC 98%						RQD = 74%
							61				
			3	HQ	REC 64%						

Continued Next Page

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

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

Golder Associates

Foundation Design

PROJECT 11-1111-0115

RECORD OF BOREHOLE No 13-03

SHEET 2 OF 2

METRIC

G.W.P. 4067-09-00

LOCATION N 4894704.9 ;E 285572.0

ORIGINATED BY PH

DIST

HWY 33

BOREHOLE TYPE CME-75 Barge Mounted, NW Casing

COMPILED BY MWK

DATUM Geodetic

DATE August 13, 2013

CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
		▨	3	HQ	REC 64%		60				RQD = 90%
59.4 15.7	END OF BOREHOLE										

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-03

SHEET 1 OF 1

LOCATION: N 4894704.9 ;E 285572.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

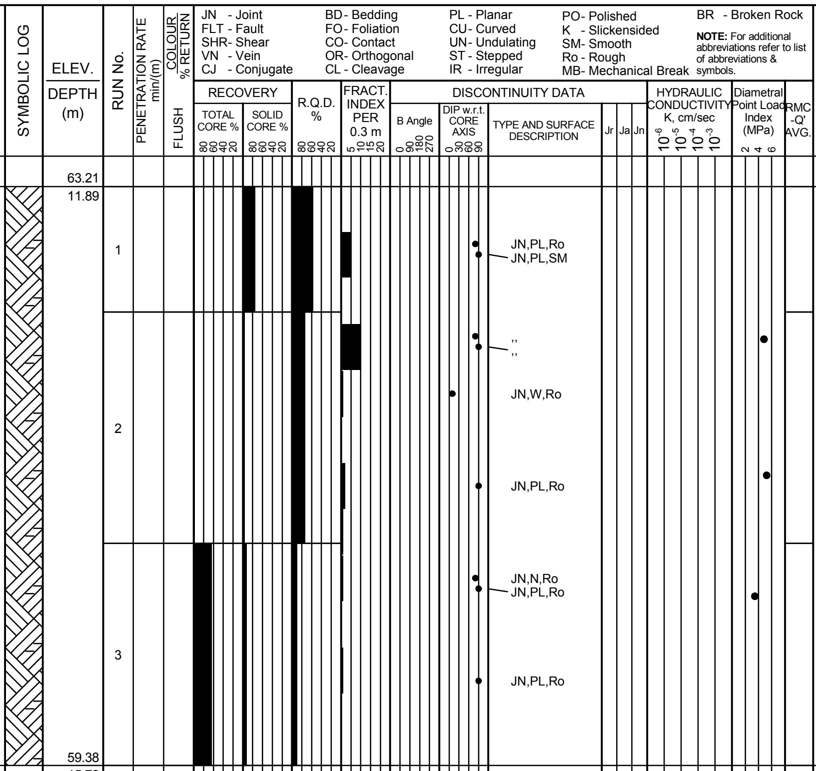
DRILL RIG: CME-75

DRILLING CONTRACTOR: Canadian Soil Drilling

DATUM: Geodetic

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	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE	63.21			
12	HQ Rock Core HW Casing	LIMESTONE, fine grained slightly porous Fresh Medium to thickly bedded Grey Medium strong to very strong Thin laminations of black shale	11.89	1	JN,PL,Ro JN,PL,SM	UCS=55.0 MPa
13				2	,, ,, JN,W,Ro	(Axial)
14					JN,PL,Ro	(Axial)
				3	JN,N,Ro JN,PL,Ro	(Axial)
15			59.38		JN,PL,Ro	UCS=106.1 MPa
		END OF DRILLHOLE	15.72			
16						
17						
18						
19						
20						
21						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PH

CHECKED: JPD

RECORD OF BOREHOLE No 13-09

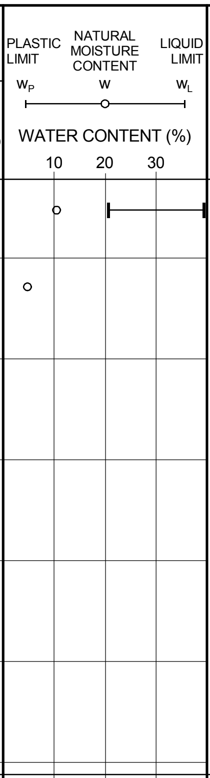
SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
LOCATION N 4894833.5 ;E 285551.9
ORIGINATED BY TWB
DIST
HWY 33
BOREHOLE TYPE CME-55 Track Mounted
COMPILED BY MWK
DATUM Geodetic
DATE October 22 and 23, 2013
CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
77.8	GROUND SURFACE							
0.0	Gravelly Silty Clay with Sand (FILL) Very stiff Dark brown Moist	1	SS	23				21 47 19 13
77.1 0.7	Sandy gravel to gravel containing crushed rock, trace silt (FILL) Dense Grey to brown Moist	2	SS	48		77		
		3	SS	50/ .10				
76.2 1.6	LIMESTONE (BEDROCK) Bedrock cored from 1.6 m to 5.9 m depth. Refer to Record of Drillhole 13-09 for bedrock coring details.	1	RC	REC 69%		76 75		RQD = 0%
		2	RC	REC 45%		74		RQD = 17%
		3	RC	REC 100%		73 72		RQD = 79%
71.9 5.9	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of overburden driling.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100



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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-09

SHEET 1 OF 1

LOCATION: N 4894833.5 ;E 285551.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

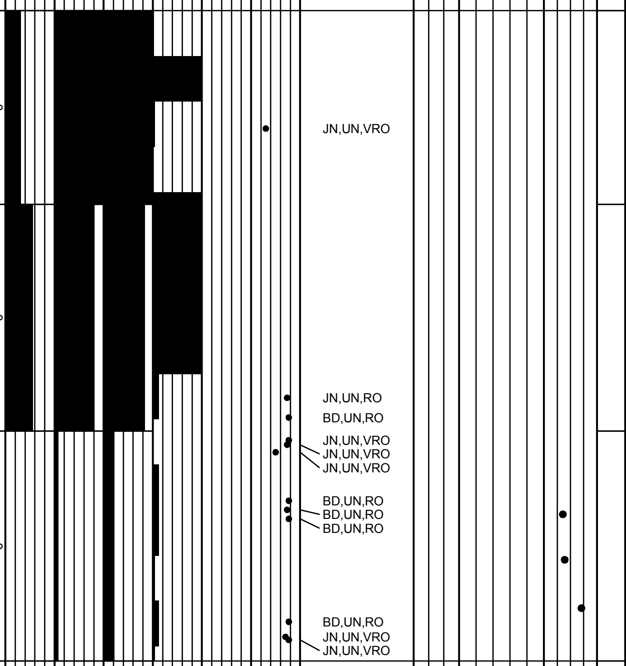
DRILL RIG: CME-55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	COLOUR % RETURN FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE	76.18				
	NQ ROCK CORING	LIMESTONE with shale interbeds Slightly weathered to fresh Laminated Grey Fine grained, slightly porous Medium strong to strong	1.60	1		0	
3				2		0	
4–5				3		0	UCS=70.9 MPa (axial) (axial) (axial)
			71.88				
6		END OF BOREHOLE	5.90				



JN,UN,VRO
JN,UN,RO
BD,UN,RO
JN,UN,VRO
JN,UN,VRO
JN,UN,VRO
BD,UN,RO
BD,UN,RO
BD,UN,RO
BD,UN,RO
JN,UN,VRO
JN,UN,VRO

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TWB

CHECKED: JPD

RECORD OF BOREHOLE No 13-10

SHEET 1 OF 1

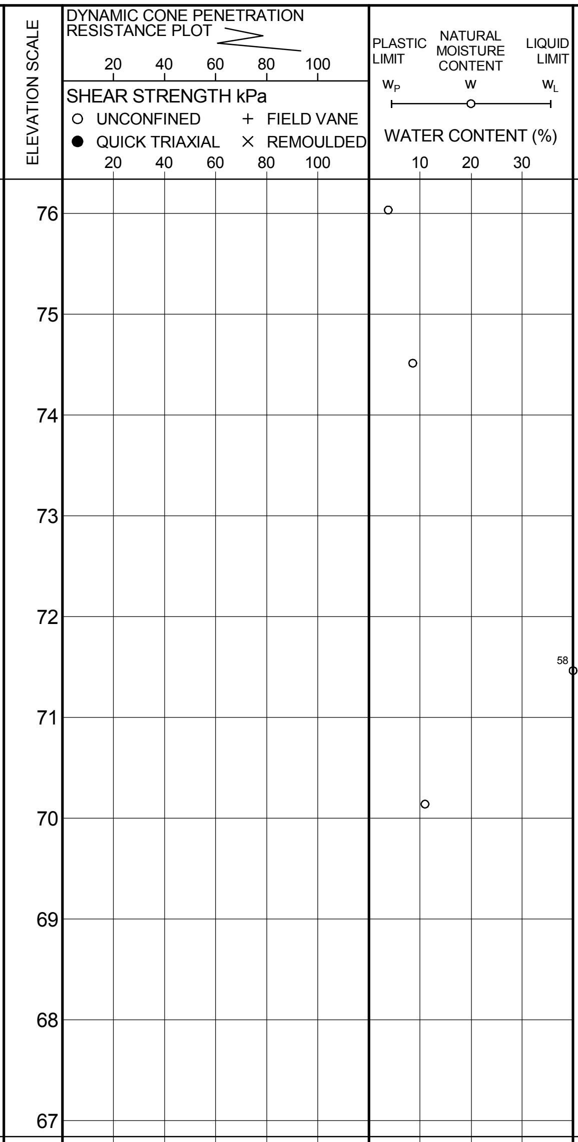
METRIC

PROJECT 11-1111-0115			
G.W.P. 4067-09-00	LOCATION N 4894777.0 ;E 285551.7	ORIGINATED BY TWB	
DIST	HWY 33	BOREHOLE TYPE CME-55 Track Mounted	COMPILED BY MWK
DATUM Geodetic	DATE October 22, 2013	CHECKED BY JPD	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
76.3	GROUND SURFACE									
0.0	Sandy gravel, trace silt, trace clay (FILL) Loose to compact Grey Moist	1	SS	23						
		2	SS	19			65	29	4	2
		3	SS	9	∇		76	20	3	1
		4	SS	19						
73.4 2.9	WOOD fragments (EXISTING CRIBWORK)	5	SS	5						
		6	SS	4						
71.9 4.4	Organic SILTY SAND, trace clay Very loose Dark grey Wet	7	SS	1						
70.7 5.6	SAND and GRAVEL, trace to some silt, trace clay Compact Dark grey Wet	8	SS	11			59	32	6	3
		9	SS	50/ .15						
70.0 6.3	LIMESTONE (BEDROCK) Bedrock cored from 6.3 m to 9.5 m depth. Refer to Record of Drillhole 13-10 for bedrock coring details.	1	RC	REC 100%			RQD = 48%			
		2	RC	REC 98%			RQD = 47%			
		3	RC	REC 100%			RQD = 92%			
66.8 9.5	END OF BOREHOLE									

NOTE:

1. Water encountered during drilling at a depth of 1.8 m (Elev. 74.5 m) below ground surface.



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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-10

SHEET 1 OF 1

LOCATION: N 4894777.0 ;E 285551.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 22, 2013

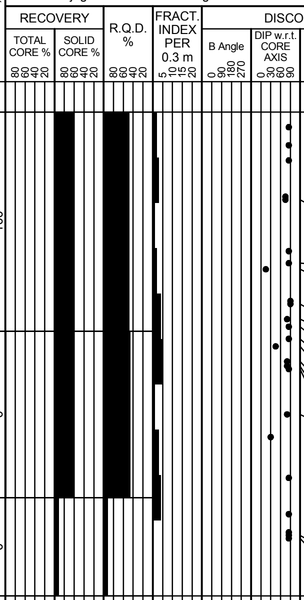
DRILL RIG: CME-55

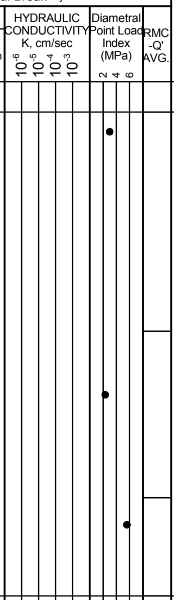
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE	70.04				
	NQ ROCK CORING	LIMESTONE with shale interbeds Slightly weathered to fresh Laminated Grey Fine grained, slightly porous Medium strong to strong	6.30			JN,UN,VRO JN,UN,VRO JN,UN,RO	(axial)
7				1	100	JN,UN,RO JN,UN,RO JN,UN,VRO JN,UN,RO JN,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO	
8				2	0	BD,UN,RO BD,UN,RO JN,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO JN,UN,RO JN,UN,RO	(axial)
		~ 3 mm dark grey clayey silt seam at 8.6 m depth				BD,UN,RO	
9			66.84	3	0	JN,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO	(axial)
		END OF BOREHOLE	9.50				
10							
11							
12							
13							
14							
15							
16							





DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TWB

CHECKED: JPD

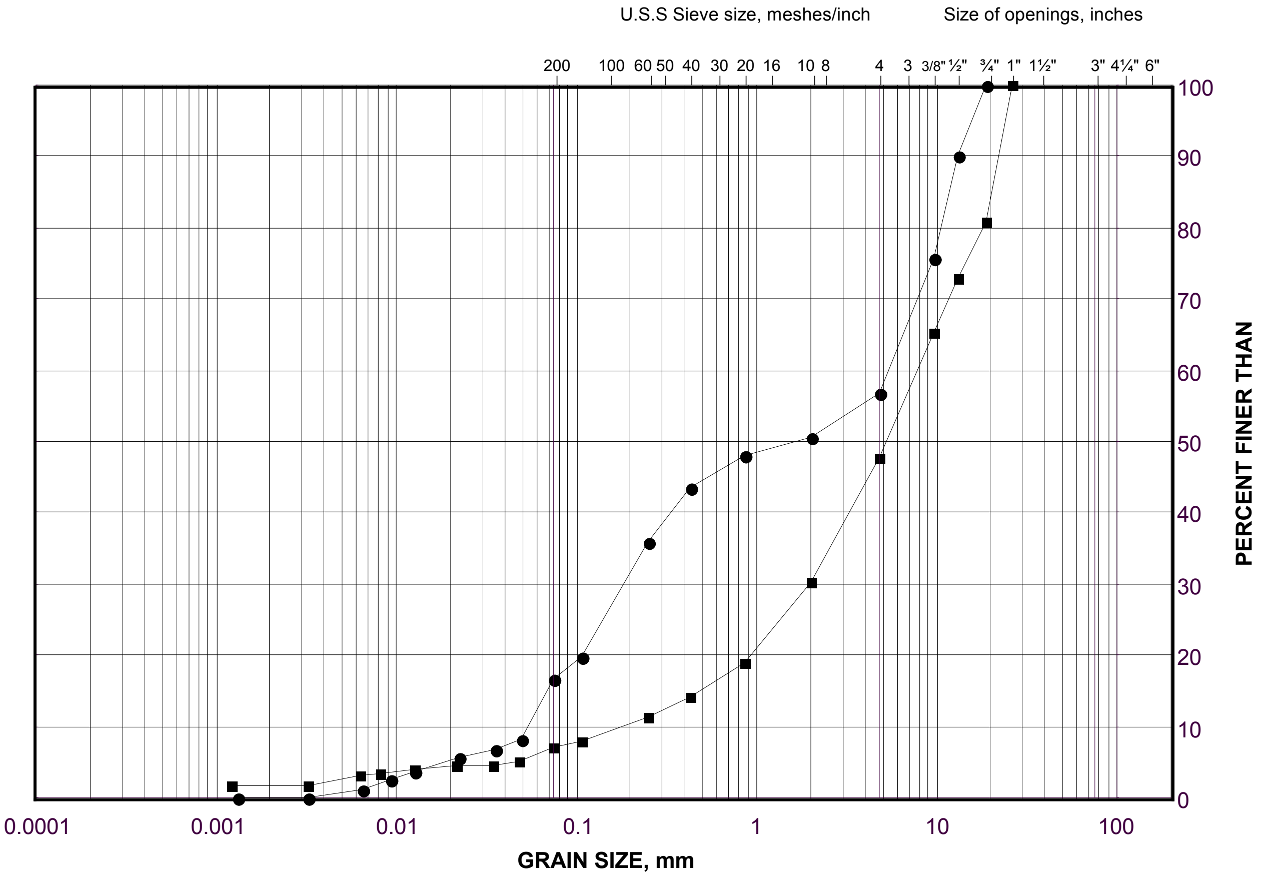
APPENDIX B

Laboratory Test Results – Millhaven Terminal

GRAIN SIZE DISTRIBUTION

Sand and Gravel (FILL)

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-02	2	67.4
■	12-02	5	65.1

Project Number: 11-1111-0115

Checked By: MWK

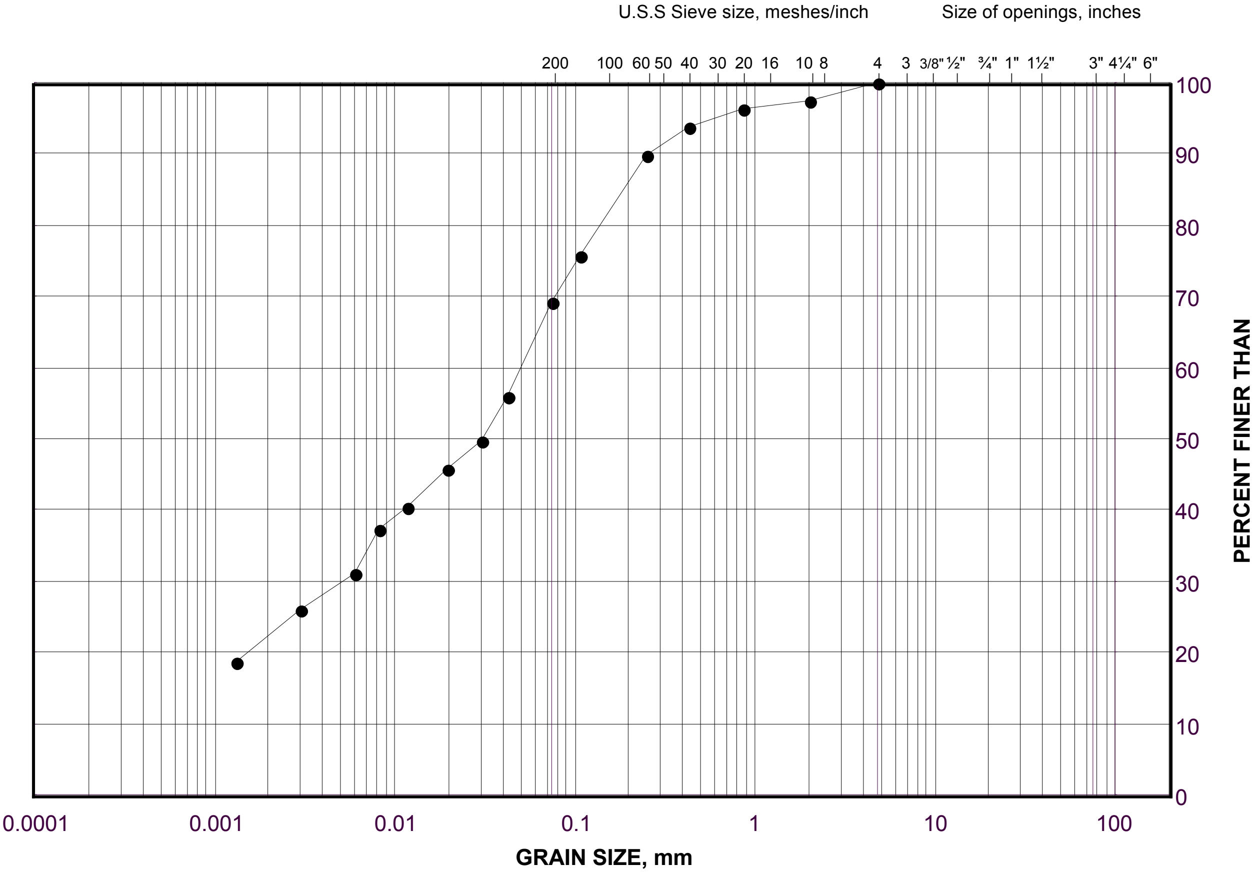
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Clayey Organic Silt with Sand

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-03	1	64.8

Project Number: 11-1111-0115

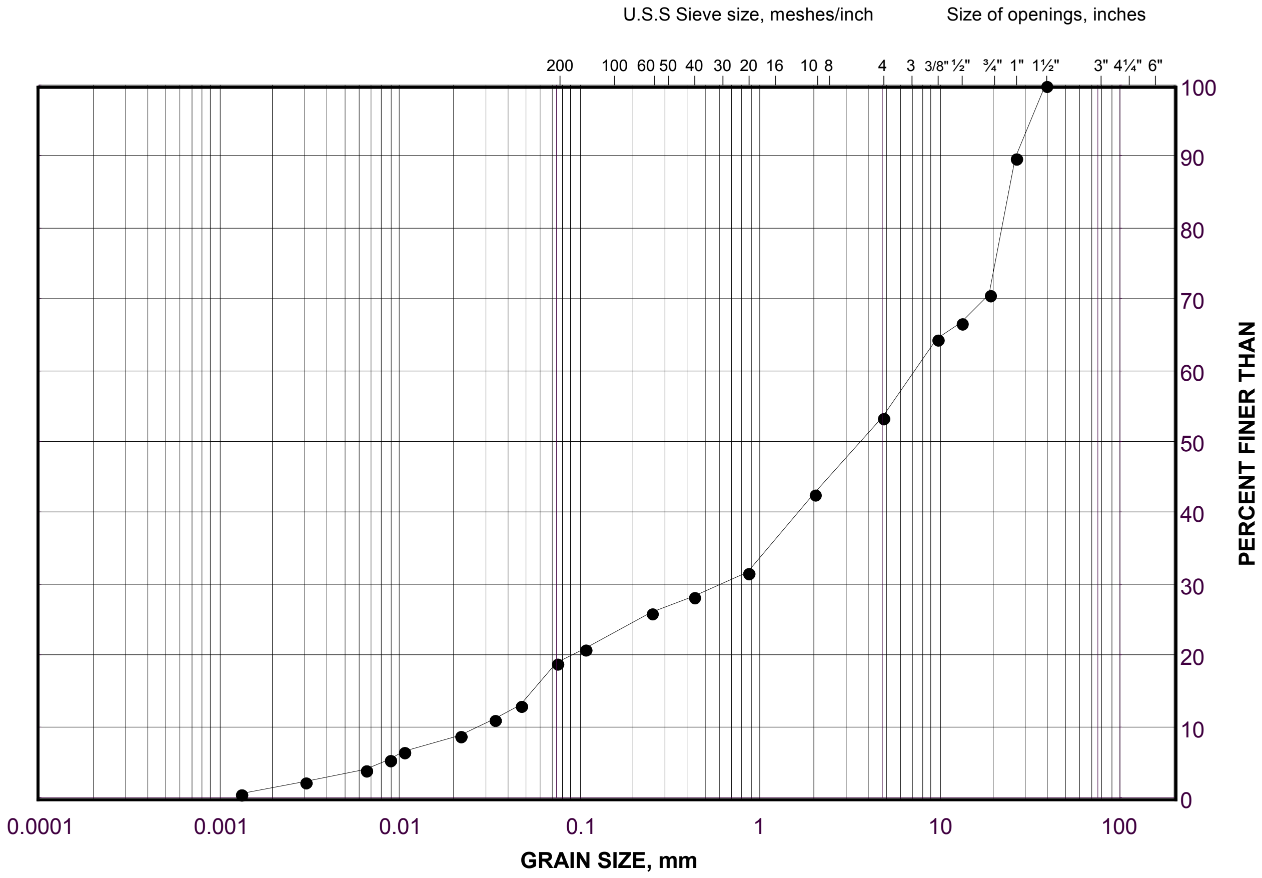
Checked By: MWK

Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION
Sand and Gravel

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-01	1	69.8

Project Number: 11-1111-0115

Checked By: MWK

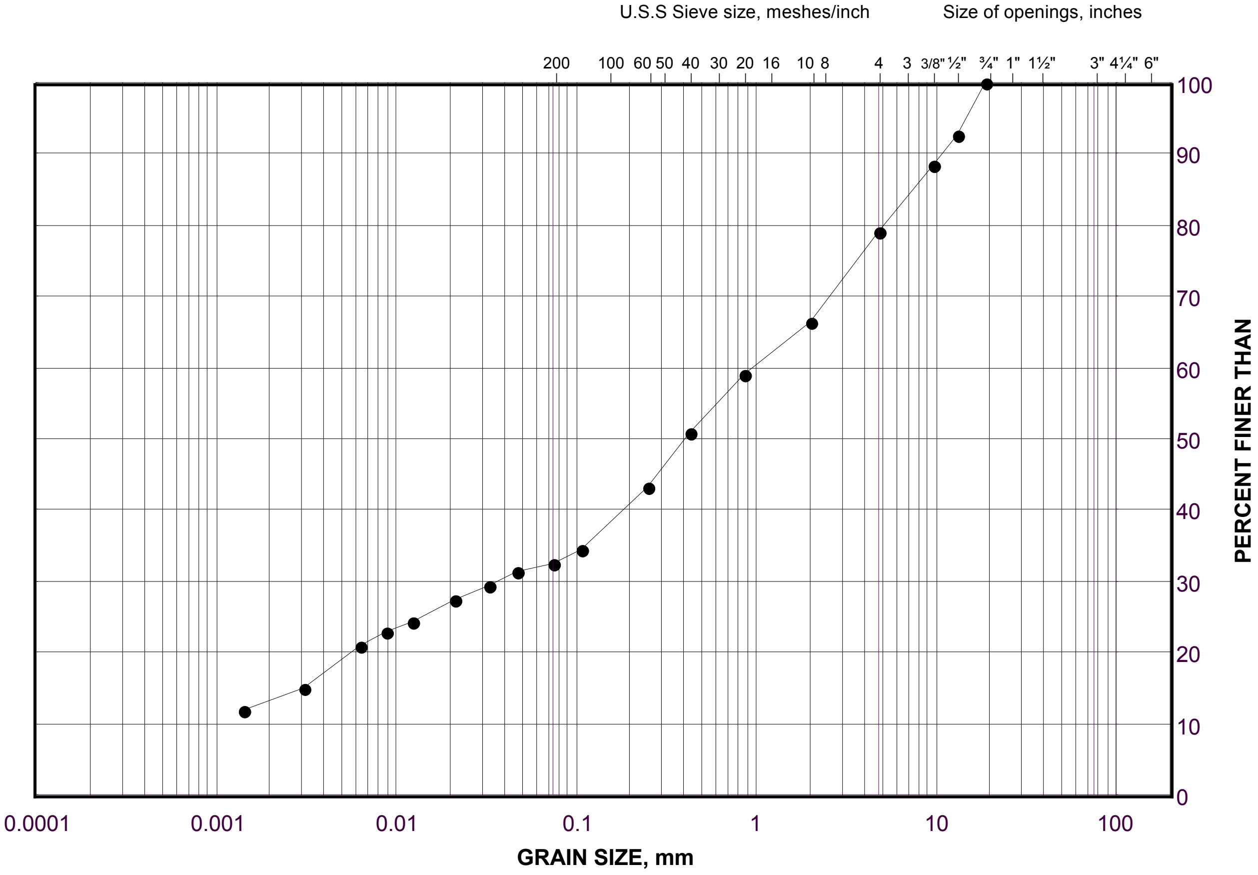
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Gravelly Silty Clay with Sand (FILL)

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-09	1	77.5

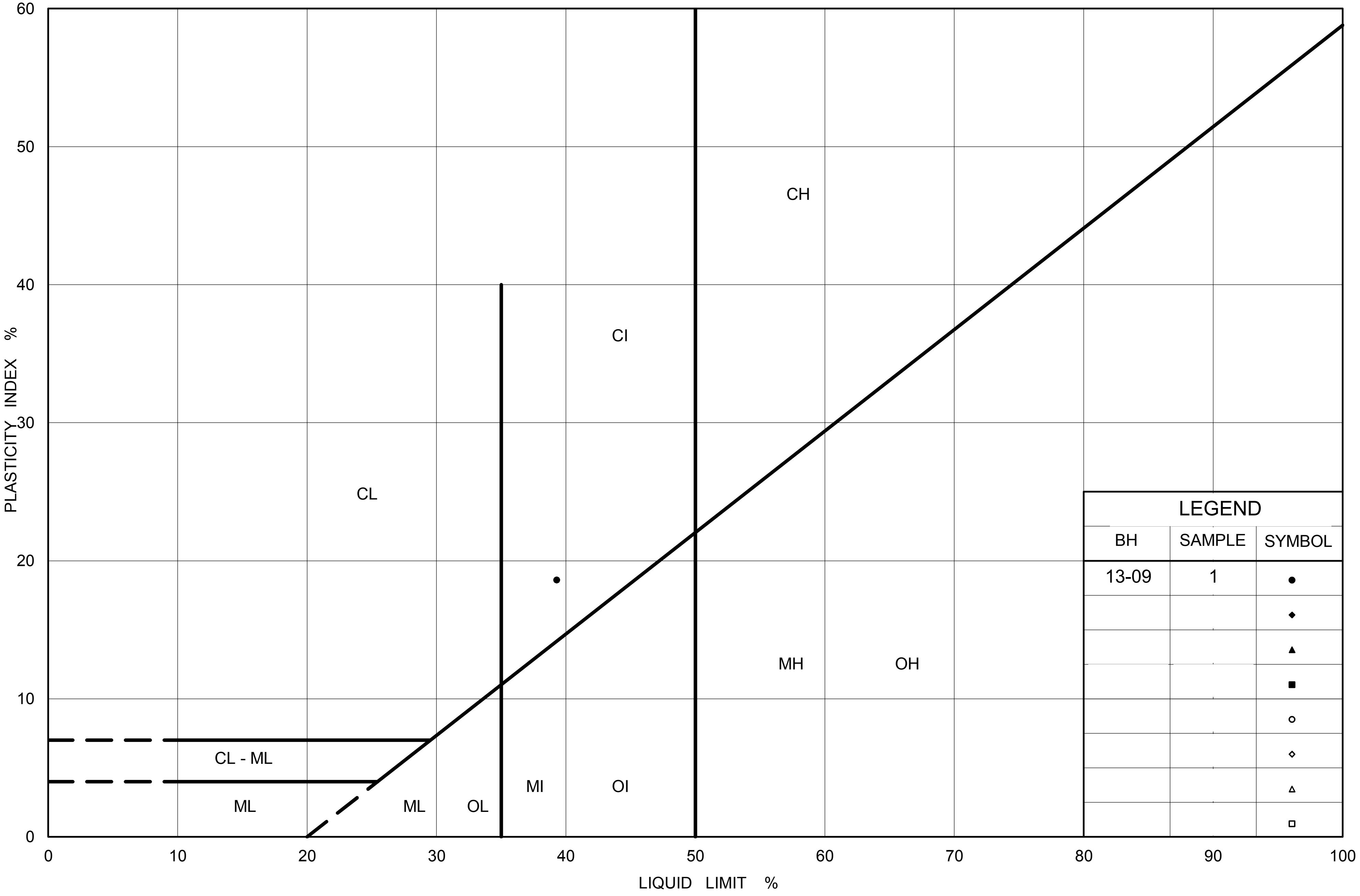
Project Number: 11-1111-0115

Checked By: MWK

Golder Associates

Date: 19-Nov-13

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay (FILL)

Figure No. B5

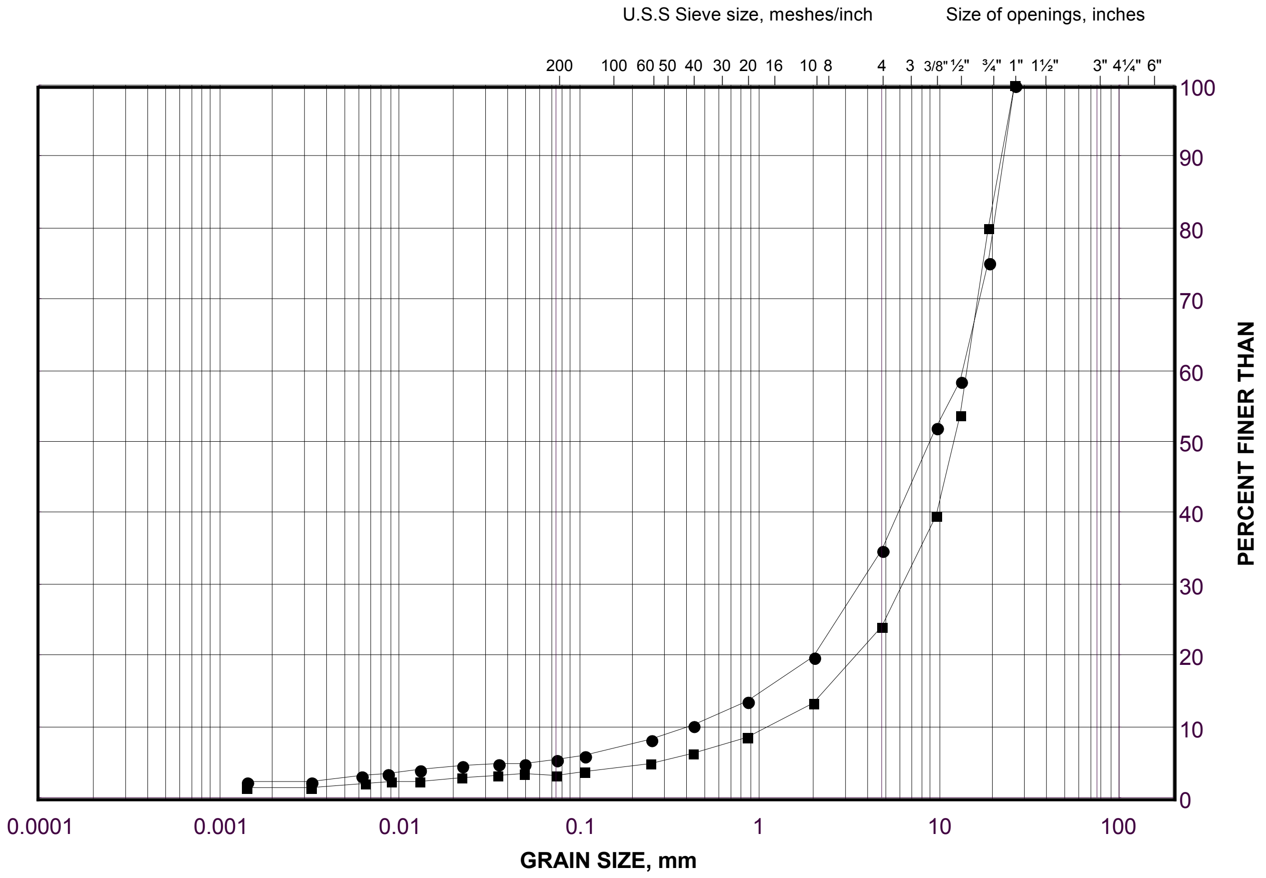
Project No. 11-1111-0115

Checked By: MWK

GRAIN SIZE DISTRIBUTION

Sandy Gravel (FILL)

FIGURE B6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-10	2	75.3
■	13-10	3	74.5

Project Number: 11-1111-0115

Checked By: MWK

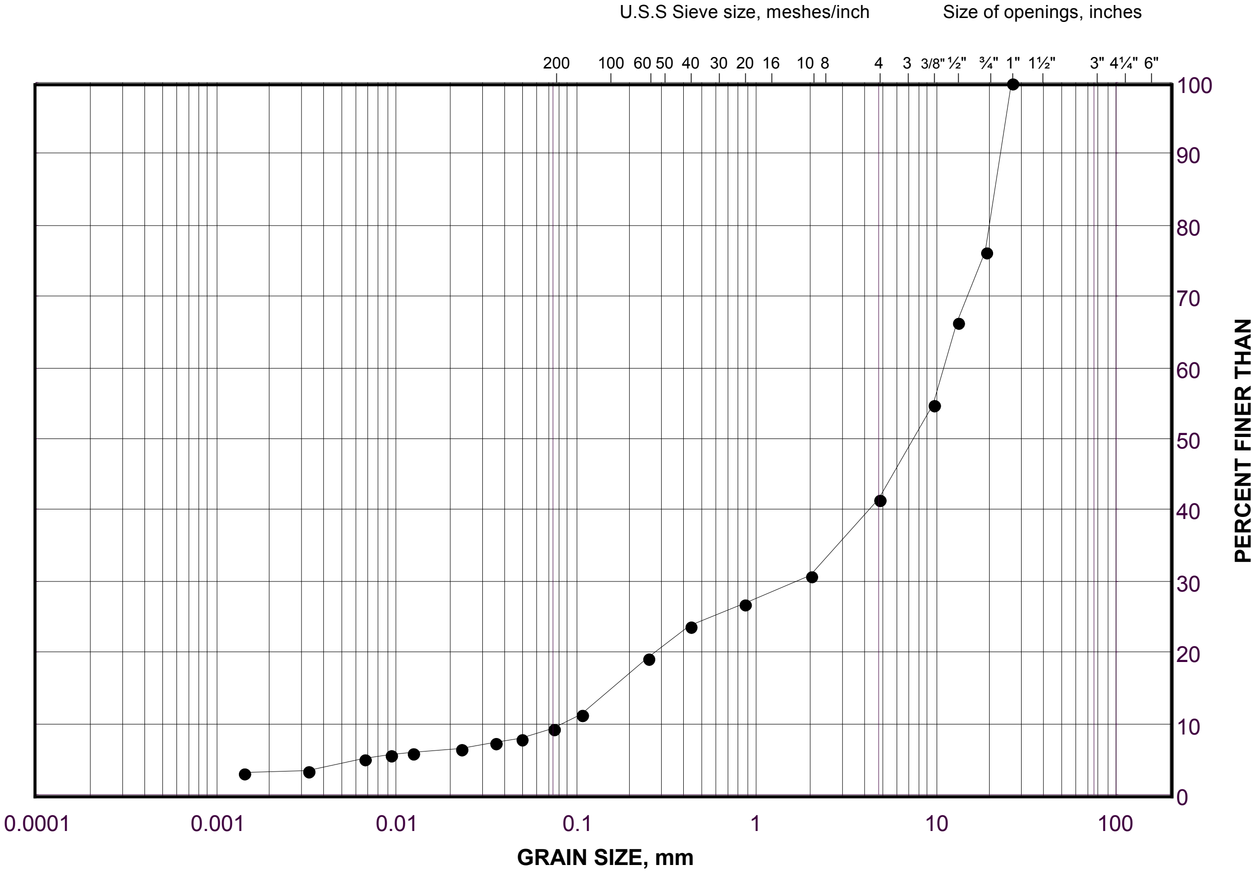
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Sand and Gravel

FIGURE B7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-10	8	70.5

Project Number: 11-1111-0115

Checked By: MWK

Golder Associates

Date: 19-Nov-13

TABLE B1 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	12-01	SAMPLE DEPTH, m	7.8

TEST CONDITIONS

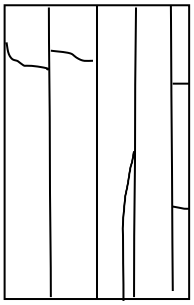
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.32

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.93	WATER CONTENT, (specimen) %	0.09
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m^3	26.44
SAMPLE AREA, cm^2	17.50	DRY UNIT WT., kN/m^3	26.42
SAMPLE VOLUME, cm^3	191.28	SPECIFIC GRAVITY	-
WET WEIGHT, g	515.95	VOID RATIO	-
DRY WEIGHT, g	515.49		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	103.0

REMARKS:

DATE: 11/12/2012

Checked By: MWK

Golder Associates

TABLE B2 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 4
BOREHOLE NUMBER	12-02A	SAMPLE DEPTH, m	7.62-7.76

TEST CONDITIONS

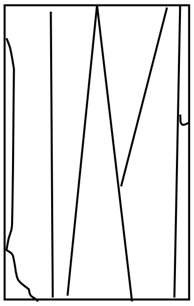
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.26

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.65	WATER CONTENT, (specimen) %	0.12
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m^3	26.23
SAMPLE AREA, cm^2	17.47	DRY UNIT WT., kN/m^3	26.20
SAMPLE VOLUME, cm^3	186.03	SPECIFIC GRAVITY	-
WET WEIGHT, g	497.76	VOID RATIO	-
DRY WEIGHT, g	497.16		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	53.5

REMARKS:		DATE:	11/15/2012

Checked By: MWK

Golder Associates

TABLE B3 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	12-04	SAMPLE DEPTH, m	5.0

TEST CONDITIONS

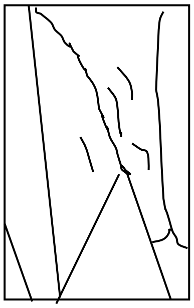
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.64	WATER CONTENT, (specimen) %	0.22
SAMPLE DIAMETER, cm	4.68	UNIT WEIGHT, kN/m^3	26.09
SAMPLE AREA, cm^2	17.23	DRY UNIT WT., kN/m^3	26.04
SAMPLE VOLUME, cm^3	183.34	SPECIFIC GRAVITY	-
WET WEIGHT, g	488.04	VOID RATIO	-
DRY WEIGHT, g	486.97		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	41.3

REMARKS:		DATE:	11/12/2012

Checked By: MWK

Golder Associates

TABLE B4 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 1
BOREHOLE NUMBER	13-02	SAMPLE DEPTH, m	12.0-12.2

TEST CONDITIONS

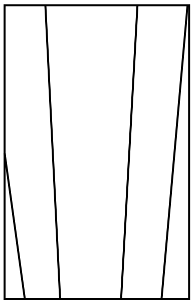
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.19

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.34	WATER CONTENT, (specimen) %	0.19
SAMPLE DIAMETER, cm	6.09	UNIT WEIGHT, kN/m^3	26.37
SAMPLE AREA, cm^2	29.08	DRY UNIT WT., kN/m^3	26.32
SAMPLE VOLUME, cm^3	387.88	SPECIFIC GRAVITY	-
WET WEIGHT, g	1043.50	VOID RATIO	-
DRY WEIGHT, g	1041.52		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	64.2

REMARKS:		DATE:	9/26/2013

Checked By: MWK

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TABLE B5 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	13-02	SAMPLE DEPTH, m	13.8-14.0

TEST CONDITIONS

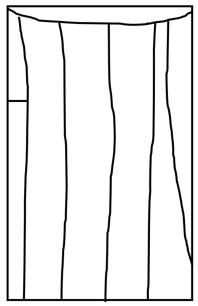
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.13

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	12.97	WATER CONTENT, (specimen) %	0.41
SAMPLE DIAMETER, cm	6.08	UNIT WEIGHT, kN/m^3	26.16
SAMPLE AREA, cm^2	29.04	DRY UNIT WT., kN/m^3	26.06
SAMPLE VOLUME, cm^3	376.66	SPECIFIC GRAVITY	-
WET WEIGHT, g	1005.30	VOID RATIO	-
DRY WEIGHT, g	1001.20		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	8.6

REMARKS:		DATE:	9/26/2013

Checked By: MWK

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TABLE B6 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 1
BOREHOLE NUMBER	13-03	SAMPLE DEPTH, m	12.5-12.7

TEST CONDITIONS

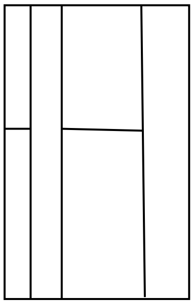
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.16

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.16	WATER CONTENT, (specimen) %	0.12
SAMPLE DIAMETER, cm	6.09	UNIT WEIGHT, kN/m^3	26.33
SAMPLE AREA, cm^2	29.13	DRY UNIT WT., kN/m^3	26.30
SAMPLE VOLUME, cm^3	383.19	SPECIFIC GRAVITY	-
WET WEIGHT, g	1029.10	VOID RATIO	-
DRY WEIGHT, g	1027.87		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	55.0

REMARKS:		DATE:	9/26/2013

Checked By: MWK

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TABLE B7 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	13-03	SAMPLE DEPTH, m	15.0-15.2

TEST CONDITIONS

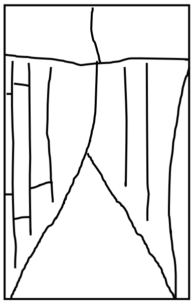
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.18

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.29	WATER CONTENT, (specimen) %	0.15
SAMPLE DIAMETER, cm	6.09	UNIT WEIGHT, kN/m^3	26.52
SAMPLE AREA, cm^2	29.13	DRY UNIT WT., kN/m^3	26.48
SAMPLE VOLUME, cm^3	387.15	SPECIFIC GRAVITY	-
WET WEIGHT, g	1047.30	VOID RATIO	-
DRY WEIGHT, g	1045.73		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	106.1

REMARKS:

DATE: 9/26/2013

Checked By: MWK

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TABLE B8 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	13-09	SAMPLE DEPTH, m	4.63-4.83

TEST CONDITIONS

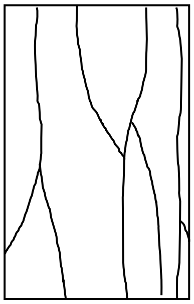
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.75	WATER CONTENT, (specimen) %	0.23
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	26.42
SAMPLE AREA, cm^2	17.57	DRY UNIT WT., kN/m^3	26.36
SAMPLE VOLUME, cm^3	188.90	SPECIFIC GRAVITY	-
WET WEIGHT, g	509.05	VOID RATIO	-
DRY WEIGHT, g	507.88		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	70.9

REMARKS:		DATE:	10/28/2013

Checked By: MWK

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TABLE B9
Field Estimation of Rock Hardness
(Representation of Intact Rock Strength)

Grade	Description	Field Identification	Approx. Range of UCS (MPa)
R0	Extremely weak rock	Indented by thumbnail.	0.25 – 1
R1	Very weak rock	Material can be shaped with a pocket knife or can be peeled by a pocket knife. Crumbles under firm blows of pick (or point) of geological hammer.	1.0 – 5.0
R2	Weak rock	Knife cuts material but too hard to shape into triaxial specimens or material can be peeled by a pocket knife with difficulty. Shallow indentations (< 5 mm) made by firm blow with pick (or point) of geological hammer.	5.0 – 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife. Hand held specimens can be fractured with *single* firm blow of geological hammer.	25 – 50
R4	Strong rock	Hand held a specimen requires *more than one* blow of geological hammer to fracture it.	50 – 100
R5	Very strong rock	Specimen requires many blows of geological hammer to break intact rock specimens (or to fracture it).	100 – 250
R6	Extremely strong rock	Specimen can only be chipped under repeated hammer blows, rings when hit.	> 250

NOTES:

1. Hand held specimens should have height ≅ 2 times the diameter.
2. Materials having a uniaxial compressive strength (UCS) of less than about 0.5 MPa and cohesionless materials should be classified using soil classification systems.
3. Rocks with a uniaxial compressive strength below 25 MPa (i.e., below R2) are likely to yield highly ambiguous results under point load testing.

REFERENCES:

1. Brown (1981). “Suggested Methods for Rock Characterization Testing and Monitoring”, International Society for Rock Mechanics.

2. Hoek, E., Kaiser, P.K., Bawden, W.F. (1995). “Support of Underground Excavations in Hard Rock”, Balkema, Rotterdam.

TABEL B10 - POINT LOAD TEST RESULTS ON ROCK SAMPLES

PROJECT NO. 11-1111-0115
TITLE URS / Ferry Terminals / Amhearst Island
DATE November, 2012

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core [(2)] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx.** [(1)] **UCS** (MPa)
12-01	Run 2	6.3	A	25.18	47.22	38.91	7,080	6.71	4.434	-	3.960	55
12-01	Run 2	7.0	D	106.31	39.75	-	11,760	11.15	-	7.056	6.364	89
12-01	Run 3	8.2	A	21.68	47.16	36.08	6,420	6.09	4.675	-	4.037	57
12-02A	Run 3	6.1	A	24.74	47.13	38.53	7,080	6.71	4.521	-	4.021	56
12-02A	Run 3	7.1	D	100.58	39.87	-	11,960	11.34	-	7.133	6.442	90
12-02A	Run 4	8.4	A	25.29	47.20	38.99	12,420	11.77	7.747	-	6.926	97
12-04	Run 2	4.5	D	92.88	42.44	-	11,960	11.34	-	6.295	5.847	82
12-04	Run 3	5.2	A	22.68	46.91	36.81	8,940	8.48	6.257	-	5.451	76
12-04	Run 4	7.1	A	24.45	46.93	38.22	9,760	9.25	6.333	-	5.612	79
13-02	Run 1	11.8	A	23.84	60.83	42.97	15,500	14.69	7.958	-	7.434	104
13-02	Run 2	13.1	A	29.51	60.78	47.79	14,900	14.13	6.185	-	6.061	85
13-02	Run 3	15.1	A	28.11	60.83	46.66	6,760	6.41	2.944	-	2.853	40
13-03	Run 2	12.9	A	27.23	60.84	45.93	11,160	10.58	5.016	-	4.828	68
13-03	Run 2	13.8	A	25.58	60.85	44.52	11,520	10.92	5.511	-	5.230	73
13-03	Run 3	14.6	A	27.03	60.91	45.78	7,940	7.53	3.591	-	3.451	48
13-09	Run 3	4.9	A	20.14	47.44	34.88	4,300	4.08	3.351	-	2.850	40
13-09	Run 3	5.5	A	15.86	47.46	30.96	7,100	6.73	7.023	-	5.660	79
13-09	Run 3	5.2	A	21.65	47.28	36.10	4,980	4.72	3.622	-	3.129	44
13-10	Run 1	6.4	A	20.08	47.45	34.83	4,540	4.30	3.548	-	3.015	42
13-10	Run 2	8.2	A	23.05	47.38	37.29	3,900	3.70	2.659	-	2.330	33
13-10	Run 3	9.0	A	23.18	47.40	37.40	9,420	8.93	6.384	-	5.602	78

(1) Is_{50} x C, from ISRM "Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.
C=14, calculated from Is_{50} average (8 tests) equal to 4.4 MPa on axial orientation and UCS average equal to 62.9 MPa (8 tests)

(2) Actual distance between point load cones at time of failure.

Checked By: MWK

APPENDIX C

Record of Boreholes and Drillholes – Stella Terminal

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
FoS	factor of safety

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

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
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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) Non-Cohesive (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.

V. MINOR SOIL CONSTITUENTS

Per cent 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 (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

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

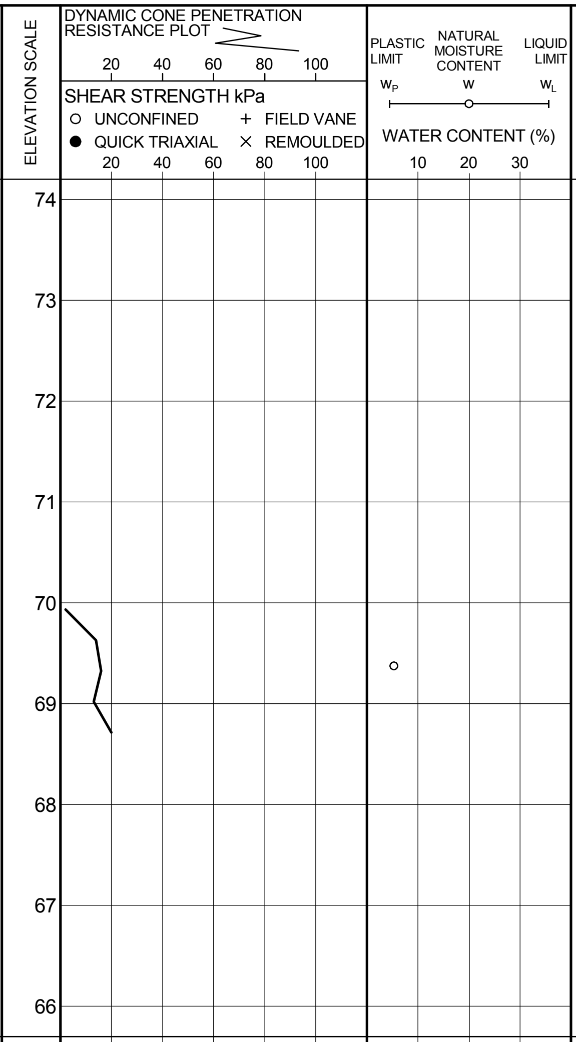
RECORD OF BOREHOLE No 12-07

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
DIST ____ HWY 33
DATUM Geodetic
LOCATION N 4892481.7 ;E 288555.8
BOREHOLE TYPE CME-55 Track Mounted
DATE September 14, 2012
ORIGINATED BY MS/DM
COMPILED BY MWK
CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
74.2	WATER SURFACE							
0.0	Water							
69.9 4.3	Sand and gravel to gravel, trace to some sand, trace silt, containing concrete fragments (FILL) Compact Grey		1	SS	21			82 17 (1)
68.3 5.9	LIMESTONE (BEDROCK) Bedrock cored from 5.9 m to 8.5 m depth. Refer to Record of Drillhole 12-07 for bedrock coring details.		1	RC	REC 71%			RQD = 0%
			2	RC	REC 37%			RQD = 7%
65.7			3	RC	REC 13%			RQD = 0%
8.5	END OF BOREHOLE							



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

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 12-07

SHEET 1 OF 1

LOCATION: N 4892481.7 ;E 288555.8

INCLINATION: -90° AZIMUTH: ---

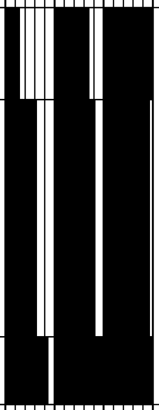
DRILLING DATE:
DRILL RIG: CME-55
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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.	PENETRATION RATE min/(m)	FLUSH COLOUR % RETURN	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/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		68.33																	
6	NQ Rock Core NW Casing	LIMESTONE, thinly bedded, fine grained, laminated Slightly to moderately weathered Grey Weak to medium strong		5.87	1		100														
		LIMESTONE, thinly bedded, fine grained, laminated, highly fractured Slightly to highly weathered Grey Strong		67.72 6.48	2		0												●		(Axial)
7																					
8																			●		(Axial)
					3		0														
		END OF DRILLHOLE		65.70 8.50																	
9																					
10																					
11																					
12																					
13																					
14																					
15																					



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MS/DM
CHECKED: JPD

RECORD OF BOREHOLE No 12-08

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
LOCATION N 4892485.0 ;E 288581.7
ORIGINATED BY MS/DM
DIST
HWY 33
BOREHOLE TYPE CME-55 Barge Mounted, NW Casing
COMPILED BY MWK
DATUM Geodetic
DATE September 14, 2012
CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
74.4	WATER SURFACE										
0.0	Water						74 73 72				
71.6											
71.3	Sand and Gravel, some silt, trace clay, containing organics and shell fragments Compact Grey Wet		1	SS	16				○		46 36 16 2
3.1	SHALEY LIMESTONE (BEDROCK) Bedrock cored from 3.1 m to 7.7 m depth. Refer to Record of Drillhole 12-08 for bedrock coring details.		1	NQ	REC 100%		71 70				RQD = 0%
			2	NQ	REC 100%		69				RQD = 29%
			3	NQ	REC 98%		68 67				RQD = 32%
66.7 7.7	END OF BOREHOLE										

GTA-MTO 001 11-1111-0115.GPJ GAL-GTA.GDT 1/10/14

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

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 12-08

SHEET 1 OF 1

LOCATION: N 4892485.0 ;E 288581.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

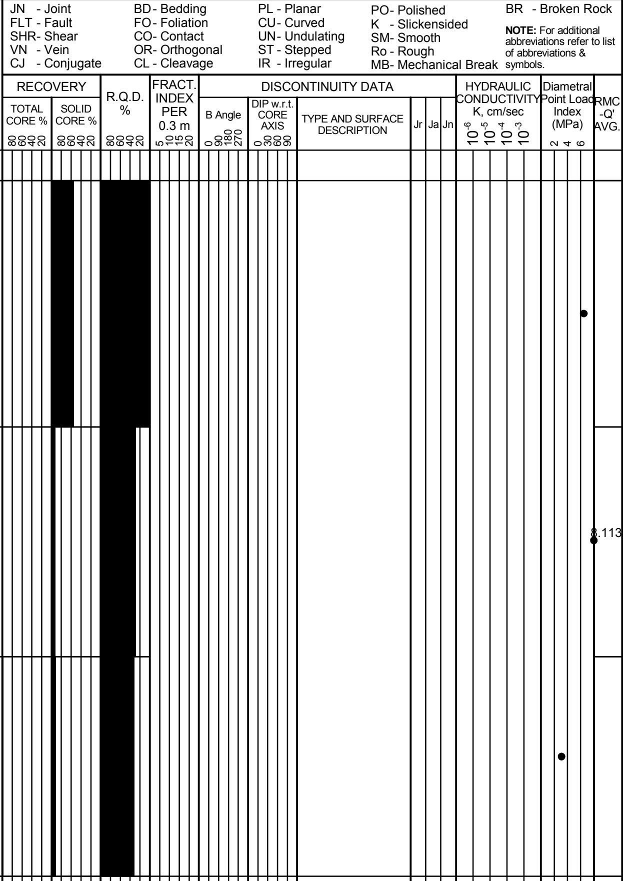
DRILL RIG: CME-55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	COLOUR % RETURN	FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE							
4 5 6 7	NQ Rock Core NW Casing	SHALEY LIMESTONE, fine grained, laminated, slightly porous Slightly weathered to fresh Grey Medium strong to strong		71.28 3.12 66.68	1 2 3				(Axial) UCS = 44.5 MPa
8 9 10 11 12 13		END OF DRILLHOLE		7.72					



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MS/DM

CHECKED: JPD

RECORD OF BOREHOLE No 13-05

SHEET 1 OF 2

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
LOCATION N 4892500.9 ;E 288505.2
ORIGINATED BY PH
DIST
HWY 33
BOREHOLE TYPE CME-75 Barge Mounted, NW Casing
COMPILED BY MWK
DATUM Geodetic
DATE August 14, 2013
CHECKED BY JPD

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
75.0	WATER SURFACE										
0.0	Water										
							74				
							73				
							72				
							71				
							70				
							69				
							68				
							67				
							66				
							65				
							64				
							63				
							62				
							61				

Continued Next Page

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

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

RECORD OF BOREHOLE No 13-05

SHEET 2 OF 2

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
DIST
HWY 33
DATUM Geodetic
LOCATION N 4892500.9 ;E 288505.2
BOREHOLE TYPE CME-75 Barge Mounted, NW Casing
DATE August 14, 2013
ORIGINATED BY PH
COMPILED BY MWK
CHECKED BY JPD

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Water								
59.3 15.7	CLAYEY ORGANIC SILT, gravelly, trace to some sand, containing shells Very soft	1	SS	WR		59	○ 172.8		23 10 31 36
						58			
57.6 17.5	Sandy gravel to SAND and GRAVEL, some silt Compact Grey	2	SS	28			○		64 25 9 2
						57			
56.4 18.6	LIMESTONE (BEDROCK) Bedrock cored from 18.6 m to 22.5 m depth. Refer to Record of Drillhole 13-05A for bedrock coring details.	1	HQ	REC 85%		56			RQD = 0%
		2	HQ	REC 75%		55			RQD = 25%
		3	HQ	REC 62%		54			RQD = 21%
		4	HQ	REC 92%		53			RQD = 29%
52.5 22.5	END OF BOREHOLE								

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

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-05A

SHEET 1 OF 1

LOCATION: N 4892500.9 ;E 288505.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: CME-75

DRILLING CONTRACTOR: Canadian Soil Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

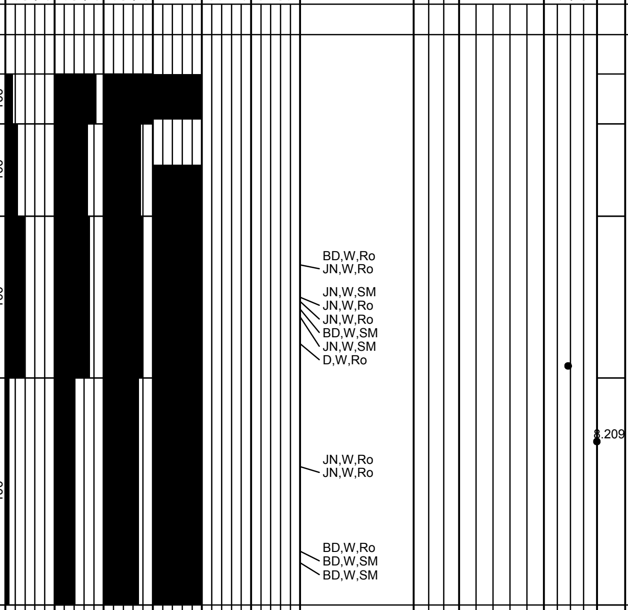
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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.	PENETRATION RATE min/(m)	COLOUR % RETURN FLUSH	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		56.29					
	HQ Rock Core HW Casing	LIMESTONE, with thin interbeds of weak to very black shale Fresh Very thinly bedded to laminated Dark grey Fine to medium grained Weak to strong Moderately fossiliferous		18.71					
19					1		100		
					2		100		
20					3		100	BD,W,Ro JN,W,Ro JN,W,SM JN,W,Ro JN,W,Ro BD,W,SM JN,W,SM D,W,Ro	(Axial)
21					4		100	JN,W,Ro JN,W,Ro	(Axial)
22				52.52				BD,W,Ro BD,W,SM BD,W,SM	UCS=22.8 MPa
		END OF DRILLHOLE		22.48					

NOTE:

Drillhole 13-05A advanced immediately adjacent to borehole 13-05 due to difficulties in advancing drillhole 13-05

23

24

25

26

27

28

DEPTH SCALE

1 : 50

LOGGED: PH

CHECKED: JPD

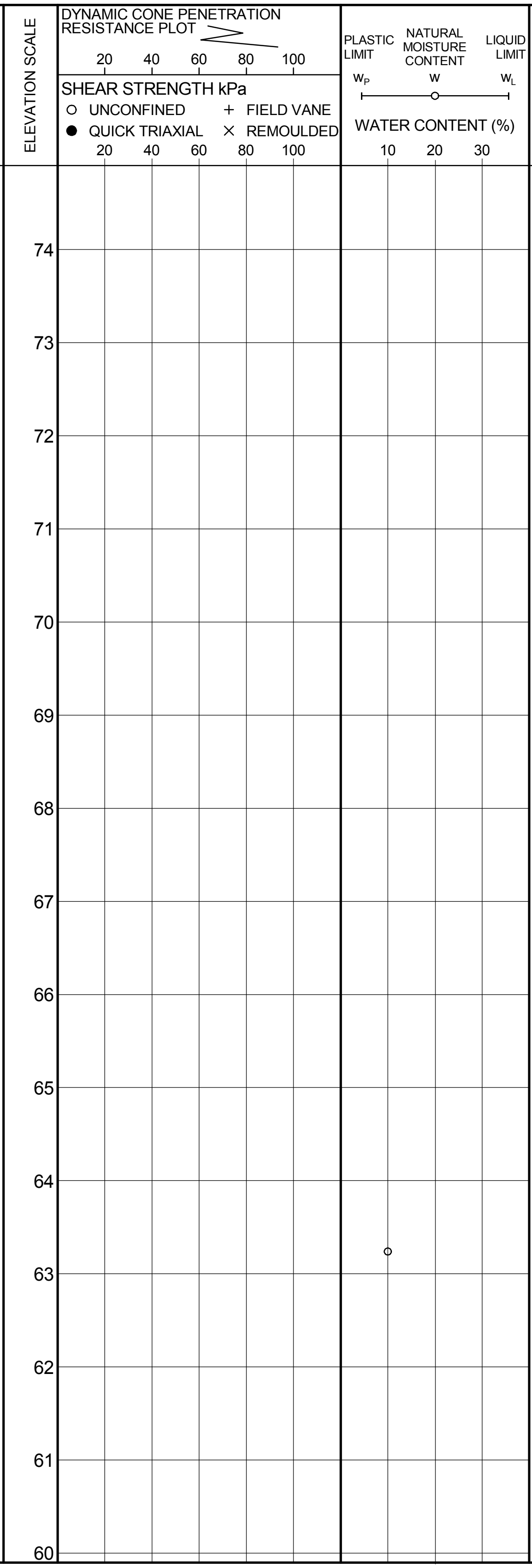
RECORD OF BOREHOLE No 13-06

SHEET 1 OF 2

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
LOCATION N 4892502.8 ;E 288538.4
ORIGINATED BY PH
DIST
HWY 33
BOREHOLE TYPE CME-75 Barge Mounted, NW Casing
COMPILED BY MWK
DATUM Geodetic
DATE August 15, 2013
CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
74.9	WATER SURFACE									
0.0	Water									
63.8										
63.5 11.4	CLAYEY ORGANIC SILT, containing shells									
	Sandy gravel, trace to some silt, trace clay Dense Grey Wet	1	SS	32			67	23	8	2
62.6 12.3	LIMESTONE (BEDROCK) Bedrock cored from 12.5 m to 16.3 m depth. Refer to Record of Drillhole 13-06 for bedrock coring details.	1	HQ	REC 100%			RQD = 88%			
		2	HQ	REC 97%			RQD = 54%			
			RC							



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Golder Associates

PROJECT 11-1111-0115

RECORD OF BOREHOLE No 13-06

SHEET 2 OF 2

METRIC

G.W.P. 4067-09-00

LOCATION N 4892502.8 ;E 288538.4

ORIGINATED BY PH

DIST

HWY 33

BOREHOLE TYPE CME-75 Barge Mounted, NW Casing

COMPILED BY MWK

DATUM Geodetic

DATE August 15, 2013

CHECKED BY JPD

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30		REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE			UNIT WEIGHT γ kN/m³	GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
58.6	LIMESTONE (BEDROCK) Bedrock cored from 12.5 m to 16.3 m depth. Refer to Record of Drillhole 13-06 for bedrock coring details.	▨	3	HQ RC	REC 98% REC %		59				RQD = 86%
16.3	END OF BOREHOLE										

GTA-MTO 001 11-1111-0115.GPJ GAL-GTA.GDT 1/10/14

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-06

SHEET 1 OF 1

LOCATION: N 4892502.8 ;E 288538.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: CME-75

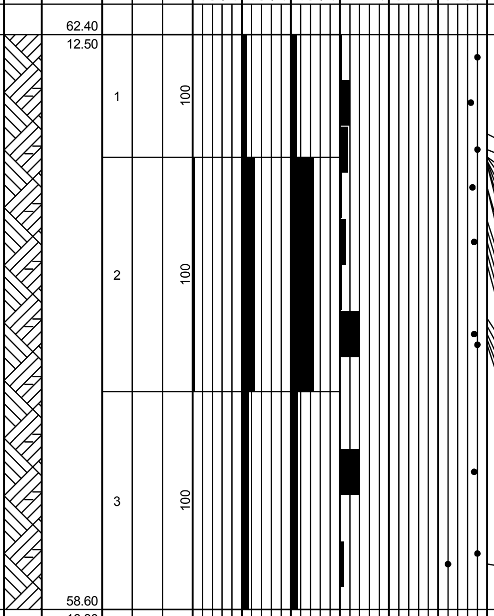
DRILLING CONTRACTOR: Canadian Soil Drilling

DATUM: Geodetic

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	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN
		BEDROCK SURFACE	62.40		
13 14 15 16	HQ Rock Core HW Casing	LIMESTONE with laminated to very thinly bedded black shale Fresh Thin to medium beds Grey Fine to medium grain Strong Moderately fossiliferous	12.50	1	100
				2	100
			58.60 16.30	3	100
17 18 19 20 21 22		END OF DRILLHOLE	16.30		

DISCONTINUITY DATA — TYPE AND SURFACE DESCRIPTION (Jr, Ja, Jn; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.)

JN,PL,SM
BR,,
JN,W,Ro
BD,PL,SM
JN,W,Ro
,,
BD,W,Ro
BD,PL,SM
,,
BD,PL,SM
BD,PL,SM
,,
JN,W,Ro
,,
JN,W,Ro
JN,W,SM
JN,W,Ro
JN,W,Ro
,,
,,
BR,,
JN,,
,,
,W,SM
,,
,W,SM
,W,SM
,,

NOTES WATER LEVELS INSTRUMENTATION

(Axial)

UCS=98.1 MPa

(Axial)

UCS=72.5 MPa

(Axial)

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PH

CHECKED: JPD

RECORD OF BOREHOLE No 13-11

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115

G.W.P. 4067-09-00

LOCATION N 4892471.8 ;E 288553.2

ORIGINATED BY TWB

DIST HWY 33

BOREHOLE TYPE CME-55 Track Mounted

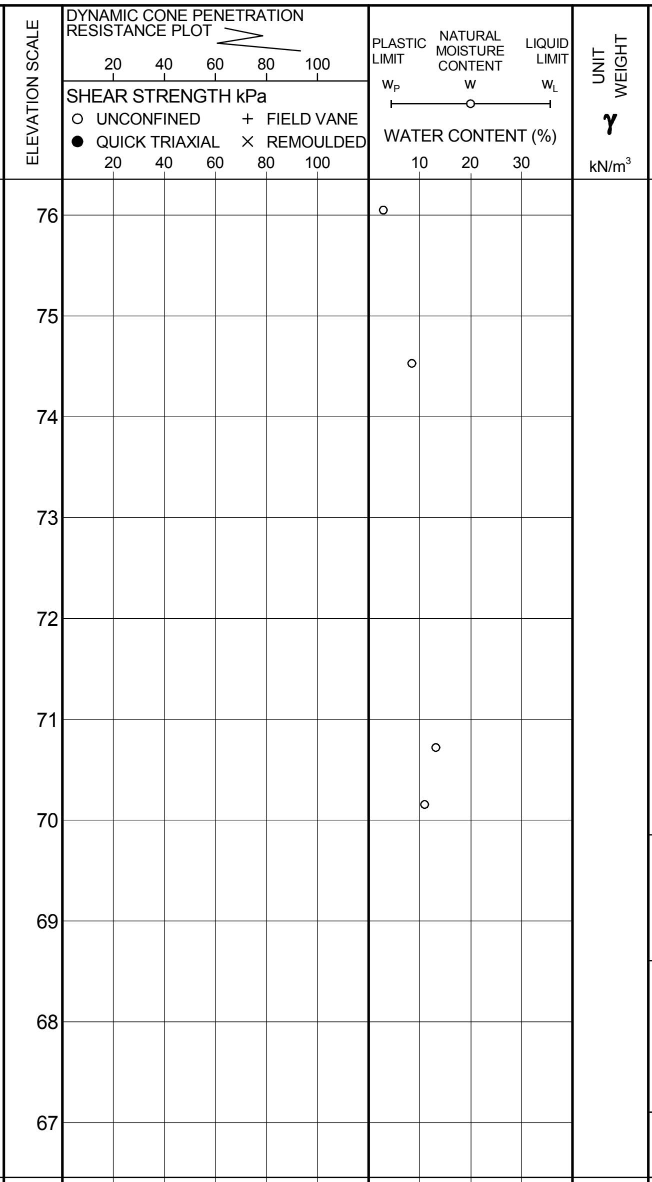
COMPILED BY MWK

DATUM Geodetic

DATE October 23, 2013

CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.4	GROUND SURFACE					
0.0	Sand and gravel, trace to some sand, trace Clay, containing pockets of clayey silt (FILL) Compact Brown Moist	1	SS	25		
		2	SS	16		57 31 8 4
74.7 1.7	Sandy gravel, trace silt, trace clay (FILL) Loose Brown Wet	3	SS	7	▽	
		4A 4B	SS	4		69 28 1 2
73.7 2.7	WOOD fragments (EXISTING CRIBWORK)	5	SS	15		
		6	SS	55		
		7	SS	5		
71.2 5.2	Sandy gravel, trace to some silt, trace clay, trace organics Compact Dark grey Wet	8	SS	22		65 25 7 3
70.5 5.9	GRAVEL, some sand, some silt, trace clay, containing rock fragments Very dense Dark grey Wet	9	SS	50/ .08		
69.9 6.5	LIMESTONE (BEDROCK) Bedrock cored from 6.5 m to 9.9 m depth. Refer to Record of Drillhole 13-11 for bedrock coring details.	1	RC	REC 100%		RQD = 0%
		2	RC	REC 100%		RQD = 31%
66.5		3	RC	REC 100%		RQD = 18%
9.9	END OF BOREHOLE NOTE: 1. Water encountered during drilling at a depth of 1.7 m (Elev. 74.7 m) below ground surface.					



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

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-11

SHEET 1 OF 1

LOCATION: N 4892471.8 ;E 288553.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 23, 2013

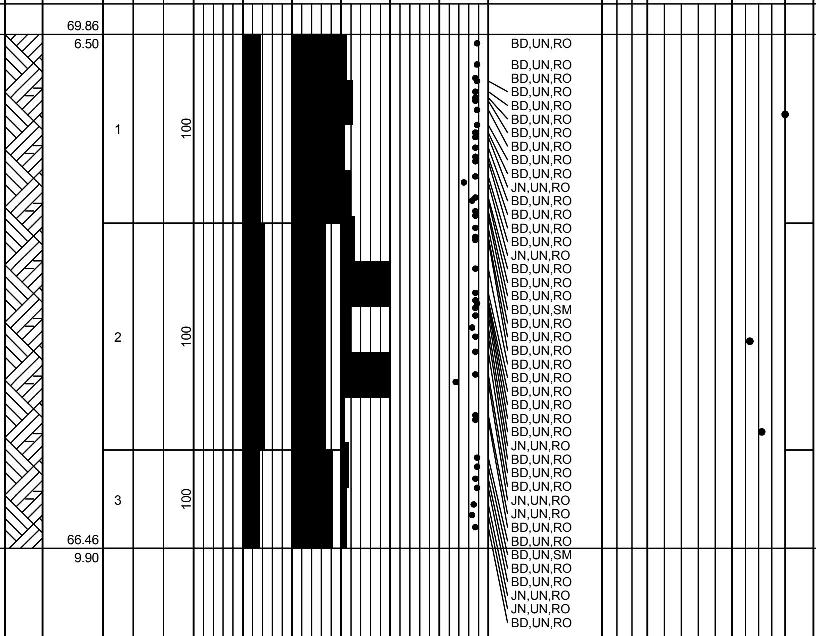
DRILL RIG: CME-55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE	69.86								
	NQ ROCK CORING October 23, 2013	LIMESTONE with shale interbeds Slightly weathered to fresh Laminated Grey Fine grained, slightly porous Medium strong to strong	6.50								
7				1		100	BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO JN,UN,RO BD,UN,RO BD,UN,RO				8.142(axial)
8				2		100	BD,UN,RO BD,UN,RO JN,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,SM BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO BD,UN,RO JN,UN,RO				(axial) (axial)
9				3		100	BD,UN,RO BD,UN,RO BD,UN,RO JN,UN,RO JN,UN,RO BD,UN,RO BD,UN,RO BD,UN,SM BD,UN,RO BD,UN,RO JN,UN,RO JN,UN,RO BD,UN,RO				
			66.46								
10		END OF BOREHOLE	9.90								
11											
12											
13											
14											
15											
16											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TWB

CHECKED: JPD

RECORD OF BOREHOLE No 13-12

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0115
G.W.P. 4067-09-00
DIST HWY 33
DATUM Geodetic
LOCATION N 4892412.9 ;E 288585.6
BOREHOLE TYPE CME-55 Track Mounted
DATE October 23, 2013
ORIGINATED BY TWB
COMPILED BY MWK
CHECKED BY JPD

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
76.4	GROUND SURFACE										
0.0	TOPSOIL										
0.2	Clayey silt with sand and gravel, trace roots and rootlets (FILL) Very stiff Brown Moist		1	SS	20		76		○		40 31 19 10
75.5			2	SS	50/ .08						
75.2 1.2	Gravel, some sand to sandy, some silt, trace clay, trace rootlets (FILL) Brown Moist		1	RC	REC 100%						RQD = 0%
	LIMESTONE (BEDROCK) Bedrock cored from 1.2 m to 4.4 m depth. Refer to Record of Drillhole 13-12 for bedrock coring details.		2	RC	REC 100%		75 74				RQD = 12%
			3	RC	REC 100%		73				RQD = 13%
72.0 4.4	END OF BOREHOLE										

NOTES:

1. Borehole dry upon completion of overburden drilling.

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

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

PROJECT: 11-1111-0115

RECORD OF DRILLHOLE: 13-12

SHEET 1 OF 1

LOCATION: N 4892412.9 ;E 288585.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 24, 2013

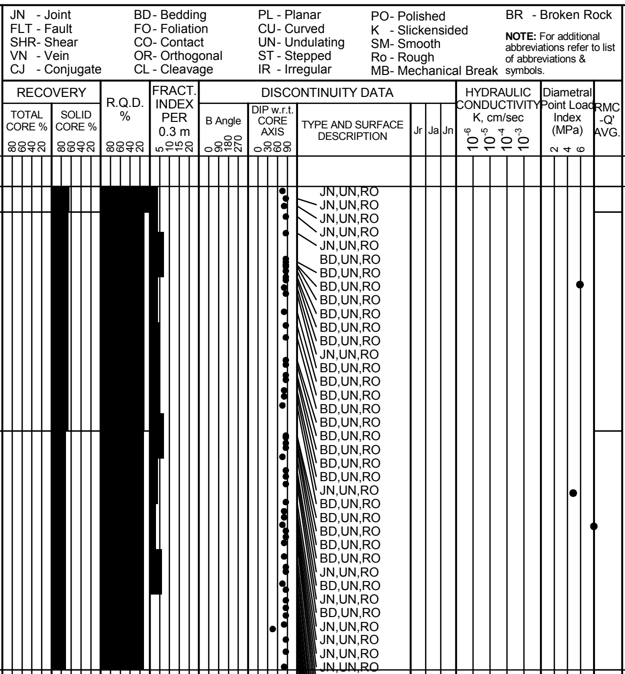
DRILL RIG: CME-55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	COLOUR % RETURN FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		75.24				
	NQ ROCK CORING October 24, 2013	LIMESTONE with shale interbeds Slightly weathered to fresh Laminated Grey Fine grained, slightly porous Strong to very strong		1.20	1		100	
2					2		100	UCS=73.8MPa (axial)
		Highly weathered shale seam at 2.3 m depth						
3					3		100	(axial) 9.512(axial)
4				72.04 4.40				
		END OF BOREHOLE						
5								
6								
7								
8								
9								
10								
11								



TYPE AND SURFACE DESCRIPTION

JN,UN,RO
JN,UN,RO
JN,UN,RO
JN,UN,RO
JN,UN,RO
BD,UN,RO
BD,UN,RO
BD,UN,RO
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BD,UN,RO
JN,UN,VRO
BD,UN,RO
JN,UN,RO
JN,UN,RO
BD,UN,RO
JN,UN,RO
BD,UN,RO

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TWB

CHECKED: JPD

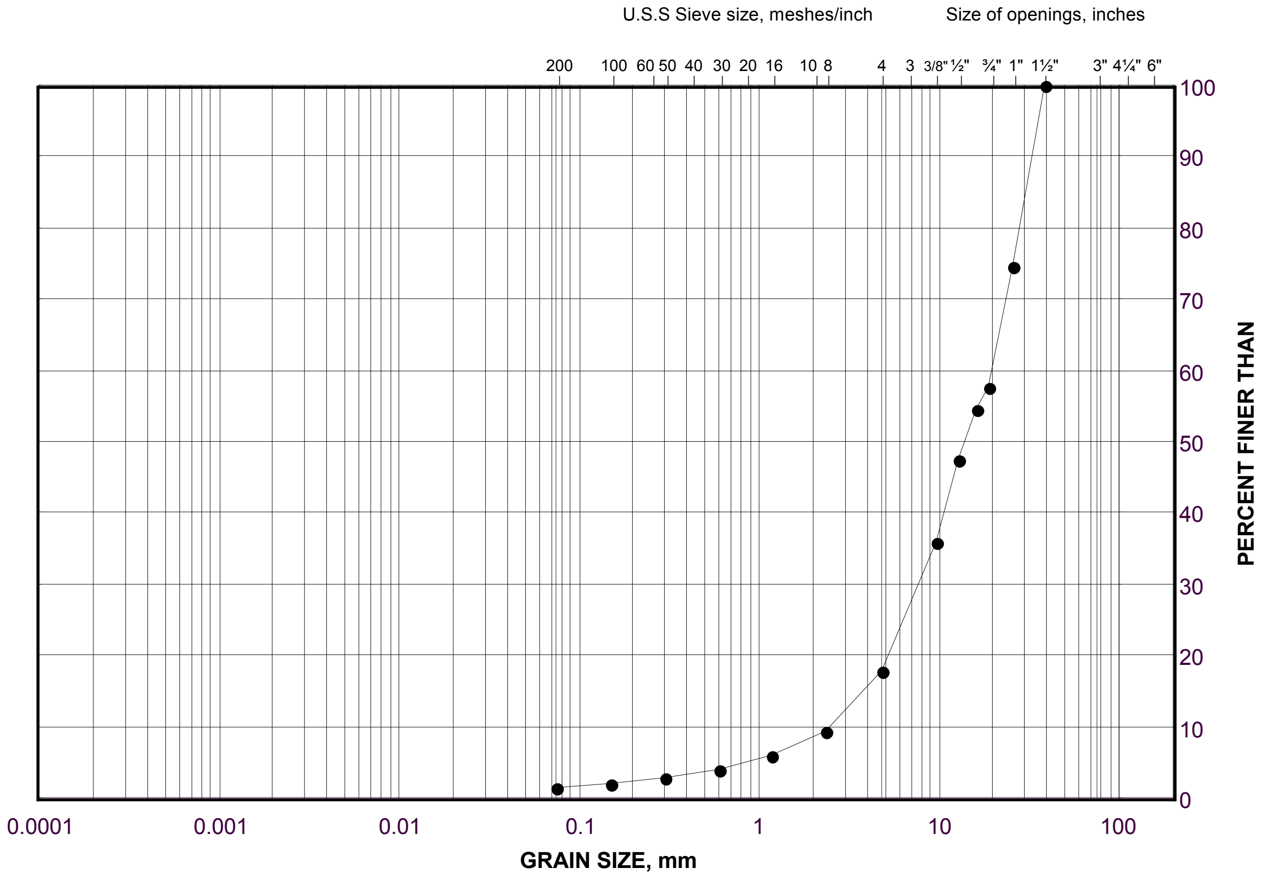
APPENDIX D

Laboratory Test Results – Stella Terminal

GRAIN SIZE DISTRIBUTION

Gravel (FILL)

FIGURE D1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-07	1	69.4

Project Number: 11-1111-0115

Checked By: MWK

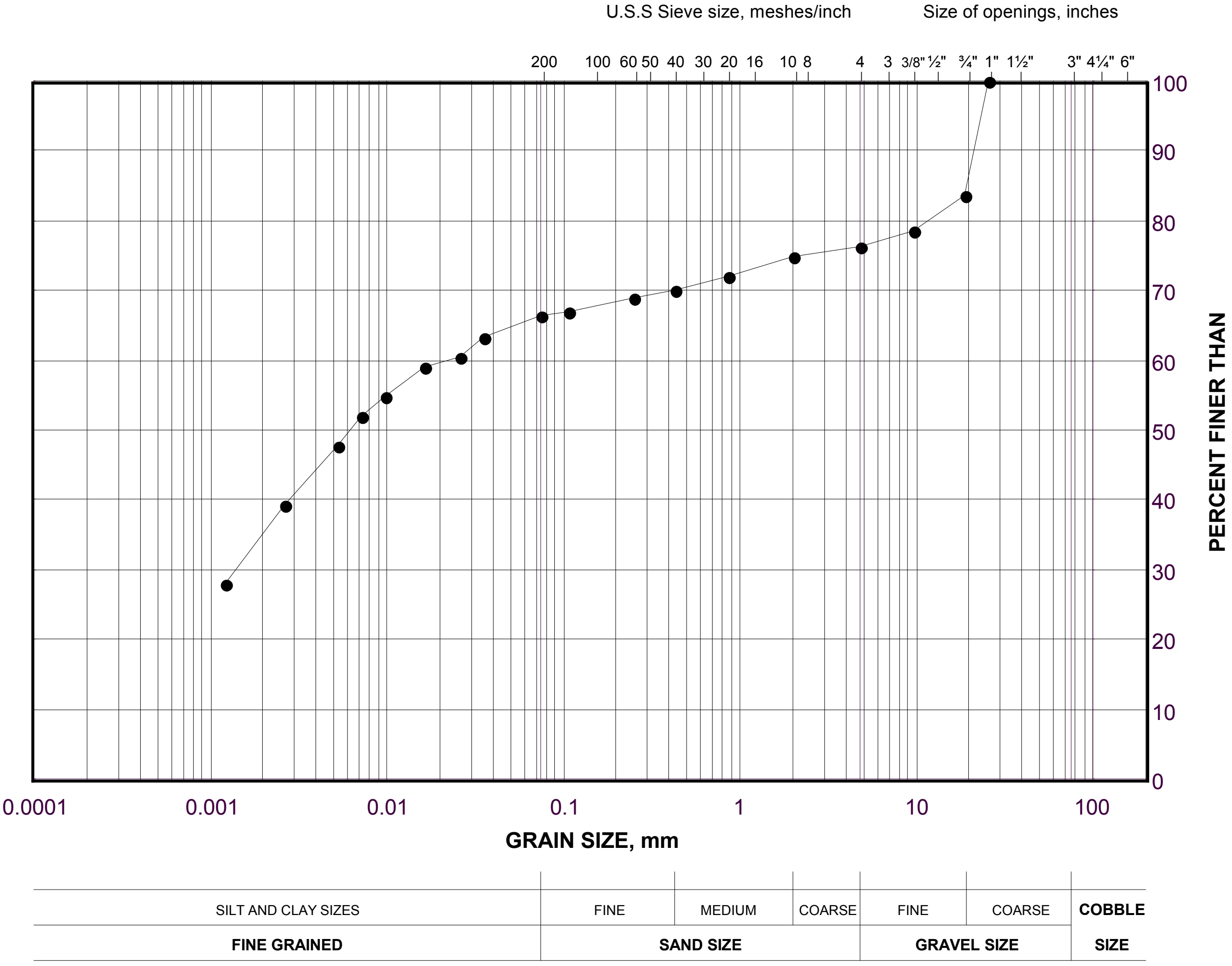
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Clayey Organic Silt

FIGURE D2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-05	1	58.9

Project Number: 11-1111-0115

Checked By: MWK

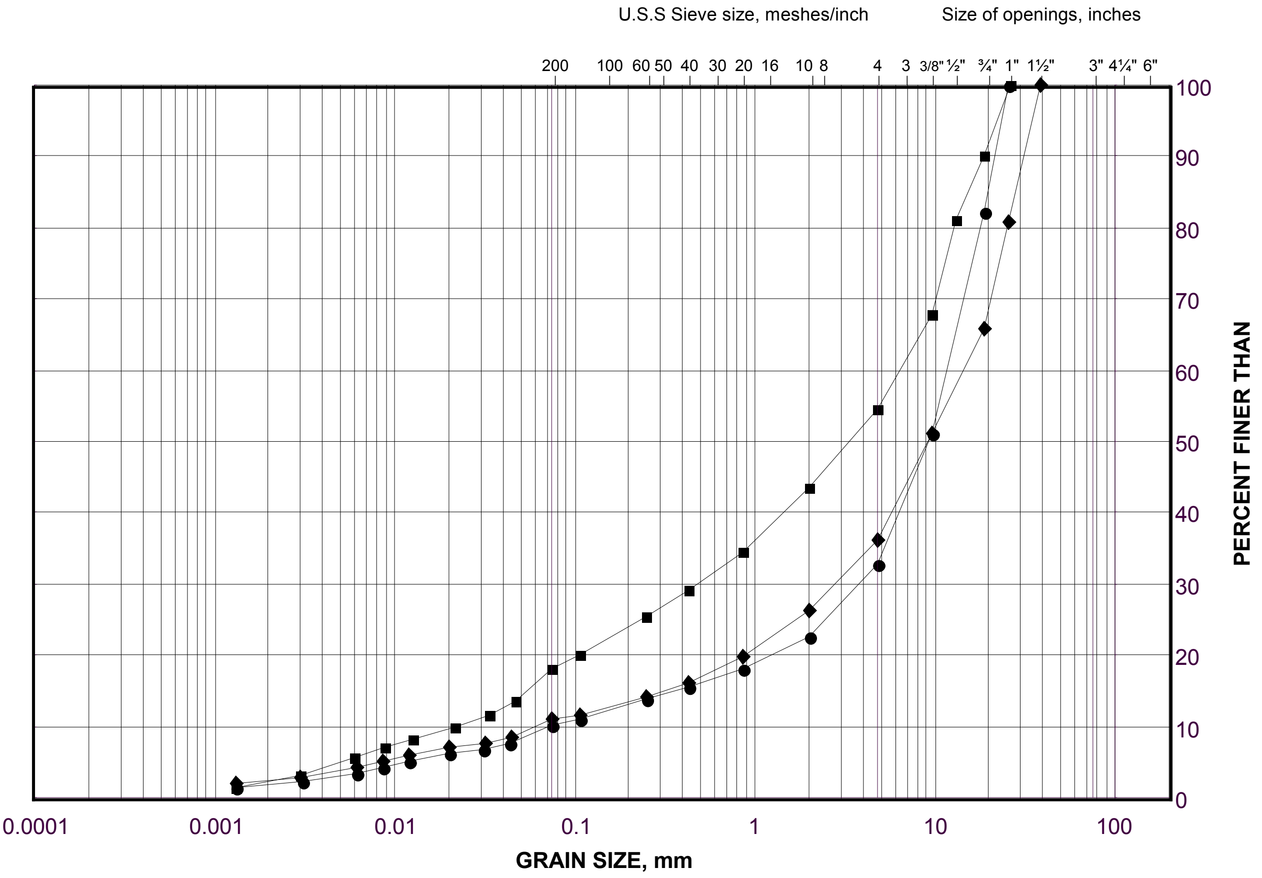
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Sand and Gravel to Sandy Gravel

FIGURE D3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-06	1	63.3
■	12-08	1	71.4
◆	13-05	2	57.2

Project Number: 11-1111-0115

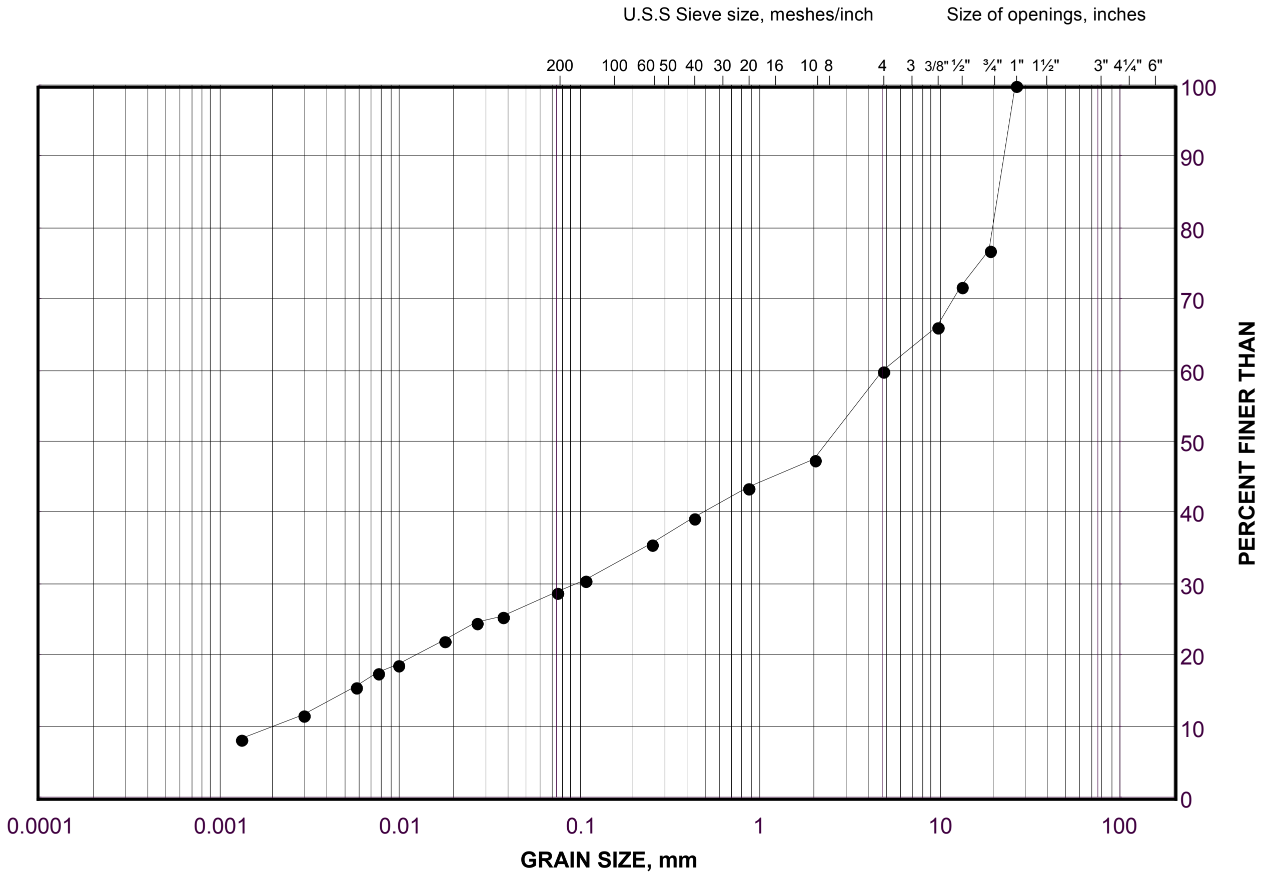
Checked By: MWK

Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION
Clayey Silt (FILL)

FIGURE D4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-12	1	76.1

Project Number: 11-1111-0115

Checked By: MWK

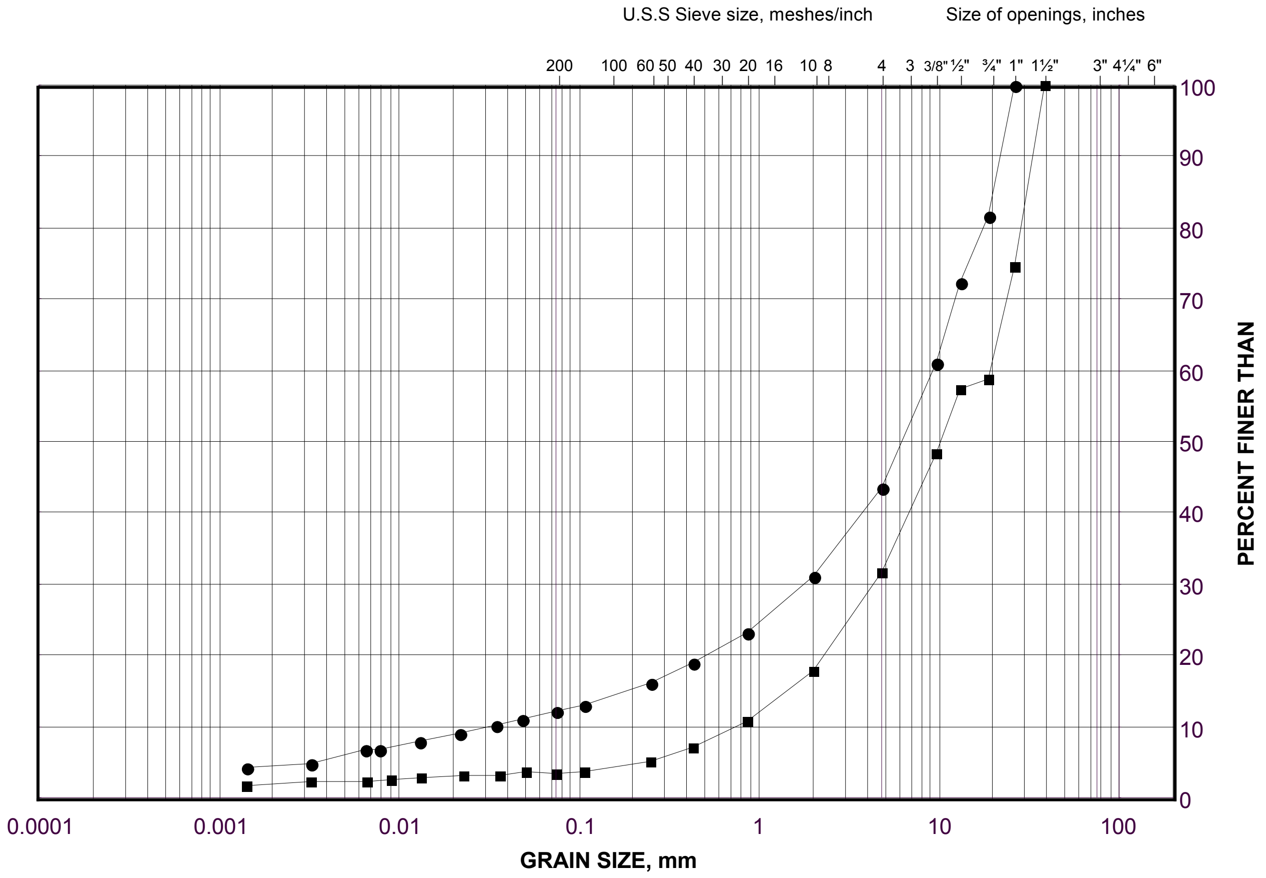
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Sand and Gravel to Sandy Gravel (FILL)

FIGURE D5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-11	2	75.3
■	13-11	4A	73.9

Project Number: 11-1111-0115

Checked By: MWK

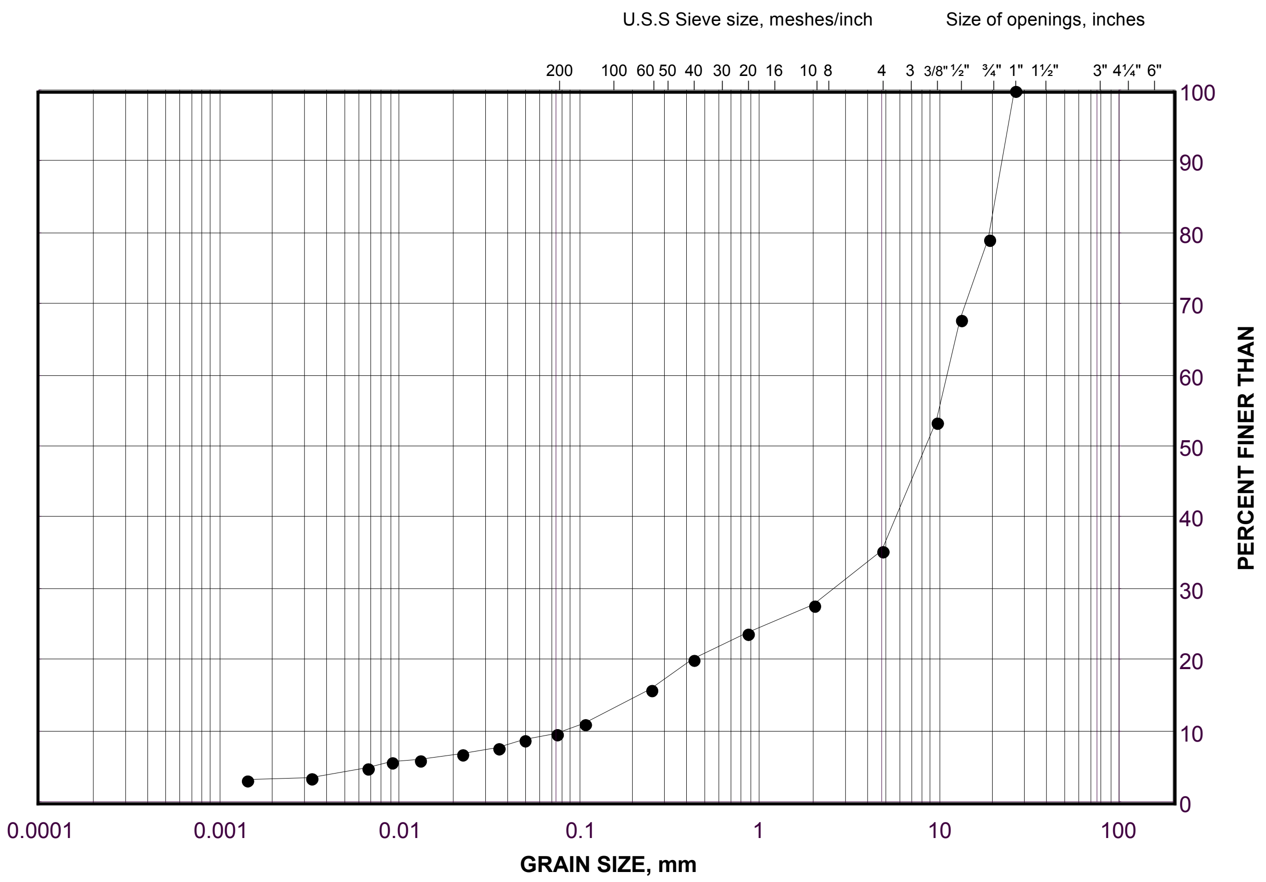
Golder Associates

Date: 19-Nov-13

GRAIN SIZE DISTRIBUTION

Sandy Gravel

FIGURE D6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-11	8	70.8

Project Number: 11-1111-0115

Checked By: MWK

Golder Associates

Date: 19-Nov-13

TABLE D1 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	12-08	SAMPLE DEPTH, m	7.5

TEST CONDITIONS

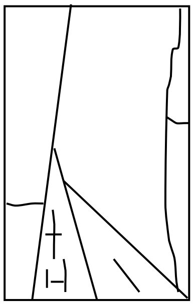
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.59	WATER CONTENT, (specimen) %	0.20
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m^3	25.91
SAMPLE AREA, cm^2	17.50	DRY UNIT WT., kN/m^3	25.86
SAMPLE VOLUME, cm^3	185.33	SPECIFIC GRAVITY	-
WET WEIGHT, g	489.89	VOID RATIO	-
DRY WEIGHT, g	488.91		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	44.5

REMARKS:		DATE:	11/12/2012

Checked By: MWK

Golder Associates

TABLE D2 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 4
BOREHOLE NUMBER	13-05A	SAMPLE DEPTH, m	22.2-22.4

TEST CONDITIONS

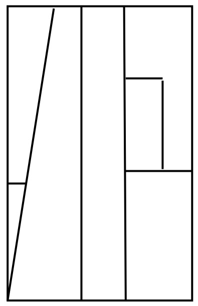
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.13

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.43	WATER CONTENT, (specimen) %	0.33
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m^3	26.09
SAMPLE AREA, cm^2	31.27	DRY UNIT WT., kN/m^3	26.01
SAMPLE VOLUME, cm^3	419.85	SPECIFIC GRAVITY	-
WET WEIGHT, g	1117.60	VOID RATIO	-
DRY WEIGHT, g	1113.92		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	22.8

REMARKS:		DATE:	9/26/2013

Checked By: MWK

Golder Associates

TABLE D3 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 2
BOREHOLE NUMBER	13-06	SAMPLE DEPTH, m	14.1-14.3

TEST CONDITIONS

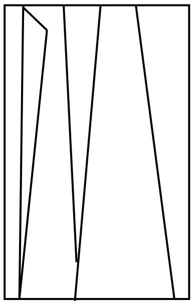
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	1.71

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.81	WATER CONTENT, (specimen) %	0.14
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m^3	26.47
SAMPLE AREA, cm^2	31.29	DRY UNIT WT., kN/m^3	26.43
SAMPLE VOLUME, cm^3	338.10	SPECIFIC GRAVITY	-
WET WEIGHT, g	912.90	VOID RATIO	-
DRY WEIGHT, g	911.62		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	98.1

REMARKS:	L/D Ratio not in accordance with ASTM Standa	DATE:	9/26/2013

Checked By: MWK

Golder Associates

TABLE D4 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	13-06	SAMPLE DEPTH, m	15.3-15.6

TEST CONDITIONS

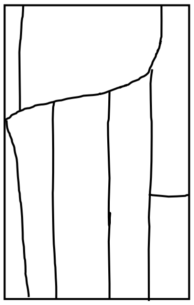
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.16

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.63	WATER CONTENT, (specimen) %	0.05
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m^3	26.47
SAMPLE AREA, cm^2	31.27	DRY UNIT WT., kN/m^3	26.45
SAMPLE VOLUME, cm^3	426.14	SPECIFIC GRAVITY	-
WET WEIGHT, g	1150.50	VOID RATIO	-
DRY WEIGHT, g	1149.93		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	72.5

REMARKS:		DATE:	9/26/2013

Checked By: MWK

Golder Associates

TABLE D5 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0115	SAMPLE NUMBER	Run 2
BOREHOLE NUMBER	13-12	SAMPLE DEPTH, m	1.51-1.87

TEST CONDITIONS

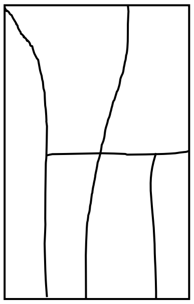
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.53	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m^3	26.38
SAMPLE AREA, cm^2	17.48	DRY UNIT WT., kN/m^3	26.35
SAMPLE VOLUME, cm^3	184.13	SPECIFIC GRAVITY	-
WET WEIGHT, g	495.50	VOID RATIO	-
DRY WEIGHT, g	495.00		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	73.8

REMARKS:		DATE:	10/28/2013

Checked By: MWK

Golder Associates

TABLE D6
Field Estimation of Rock Hardness
(Representation of Intact Rock Strength)

Grade	Description	Field Identification	Approx. Range of UCS (MPa)
R0	Extremely weak rock	Indented by thumbnail.	0.25 – 1
R1	Very weak rock	Material can be shaped with a pocket knife or can be peeled by a pocket knife. Crumbles under firm blows of pick (or point) of geological hammer.	1.0 – 5.0
R2	Weak rock	Knife cuts material but too hard to shape into triaxial specimens or material can be peeled by a pocket knife with difficulty. Shallow indentations (< 5 mm) made by firm blow with pick (or point) of geological hammer.	5.0 – 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife. Hand held specimens can be fractured with *single* firm blow of geological hammer.	25 – 50
R4	Strong rock	Hand held a specimen requires *more than one* blow of geological hammer to fracture it.	50 – 100
R5	Very strong rock	Specimen requires many blows of geological hammer to break intact rock specimens (or to fracture it).	100 – 250
R6	Extremely strong rock	Specimen can only be chipped under repeated hammer blows, rings when hit.	> 250

NOTES:

1. Hand held specimens should have height ≅ 2 times the diameter.
2. Materials having a uniaxial compressive strength (UCS) of less than about 0.5 MPa and cohesionless materials should be classified using soil classification systems.
3. Rocks with a uniaxial compressive strength below 25 MPa (i.e., below R2) are likely to yield highly ambiguous results under point load testing.

REFERENCES:

1. Brown (1981). “Suggested Methods for Rock Characterization Testing and Monitoring”, International Society for Rock Mechanics.

2. Hoek, E., Kaiser, P.K., Bawden, W.F. (1995). “Support of Underground Excavations in Hard Rock”, Balkema, Rotterdam.

TABEL D7 - POINT LOAD TEST RESULTS ON ROCK SAMPLES

PROJECT NO. 11-1111-0115

TITLE URS / Ferry Terminals / Amhearst Island

DATE November, 2012

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core (2) Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx. (1) UCS** (MPa)
12-07	Run 2	7.8	A	23.09	46.95	37.15	10,500	9.95	7.212	-	6.310	76
12-07	Run 2	6.8	A	31.63	47.10	43.55	13,060	12.38	6.527	-	6.134	74
12-08	Run 1	4.0	A	36.36	46.97	46.63	15,320	14.52	6.679	-	6.473	78
12-08	Run 2	5.5	D	100.87	38.10	-	14,040	13.31	-	9.169	8.113	97
12-08	Run 3	7.0	D	60.01	40.68	-	9,780	9.27	-	5.603	5.106	61
13-05A	Run 3	20.9	A	27.72	63.07	47.18	8,800	8.34	3.748	-	3.651	44
13-05A	Run 4	21.4	A	25.57	63.05	45.31	18,580	17.61	8.581	-	8.209	99
13-06	Run 2	13.6	A	25.25	63.19	45.07	17,560	16.65	8.195	-	7.821	94
13-06	Run 2	14.6	A	26.76	63.17	46.39	12,420	11.77	5.471	-	5.289	63
13-06	Run 3	15.8	A	24.32	63.13	44.21	11,400	10.81	5.529	-	5.231	63
13-11	Run 1	7.1	A	19.46	47.42	34.28	11,960	11.34	9.650	-	8.142	98
13-11	Run 2	8.6	A	19.47	47.35	34.26	3,900	3.70	3.150	-	2.657	32
13-11	Run 2	9.2	A	19.63	47.41	34.42	6,600	6.26	5.280	-	4.464	54
13-12	Run 2	1.9	A	19.94	47.44	34.70	8,820	8.36	6.942	-	5.890	71
13-12	Run 3	3.3	A	22.09	47.41	36.52	7,880	7.47	5.602	-	4.864	58
13-12	Run 3	3.5	A	23.05	47.38	37.29	15,920	15.09	10.854	-	9.512	114

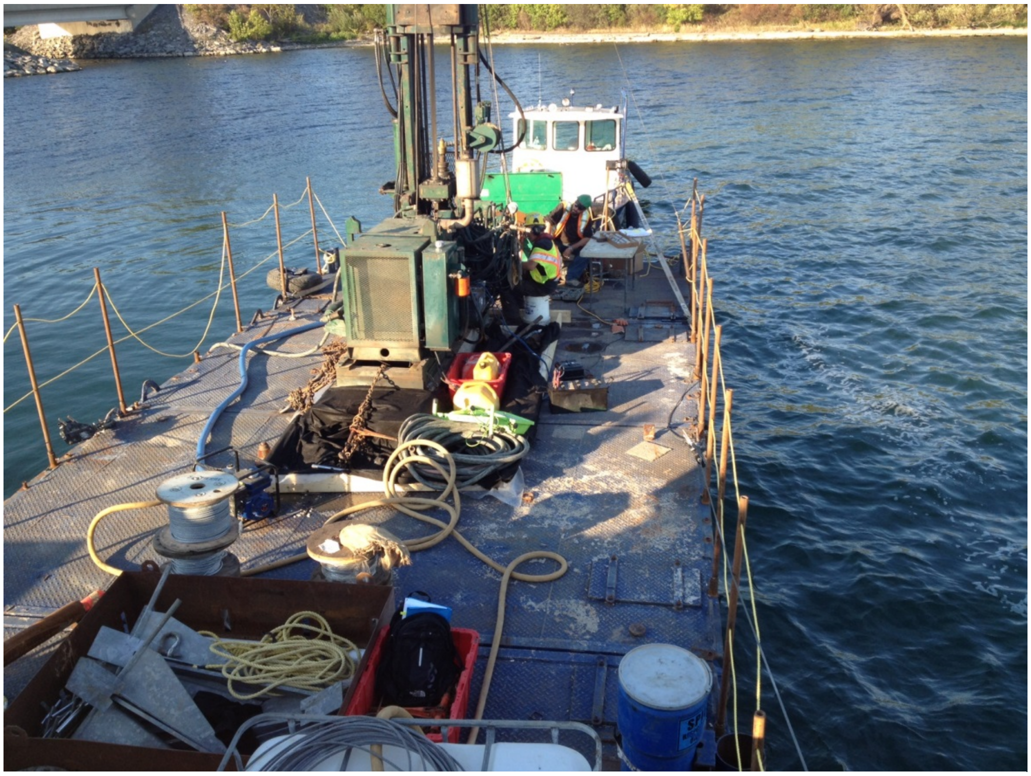
(1) Is_{50} x C, from ISRM "Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.
C=12, calculated from Is_{50} average (4 tests) equal to 6.0 MPa on axial orientation and UCS average equal to 72.2 MPa (4 tests)

(2) Actual distance between point load cones at time of failure.

Checked By: MWK

APPENDIX E

Foundation Investigation Photographs



Modular Barge at Borehole 12-01 (09/12/2012)

Modular barge tug boat at Borehole 12-01 (09/12/2012)

Modular barge and tug boat between Millhaven and Stella (09/13/2012)

Jack-Up Barge at Borehole 13-03 (08/14/2013)

Jack-Up Barge and tug boat at Borehole 13-03 (08/14/2013)

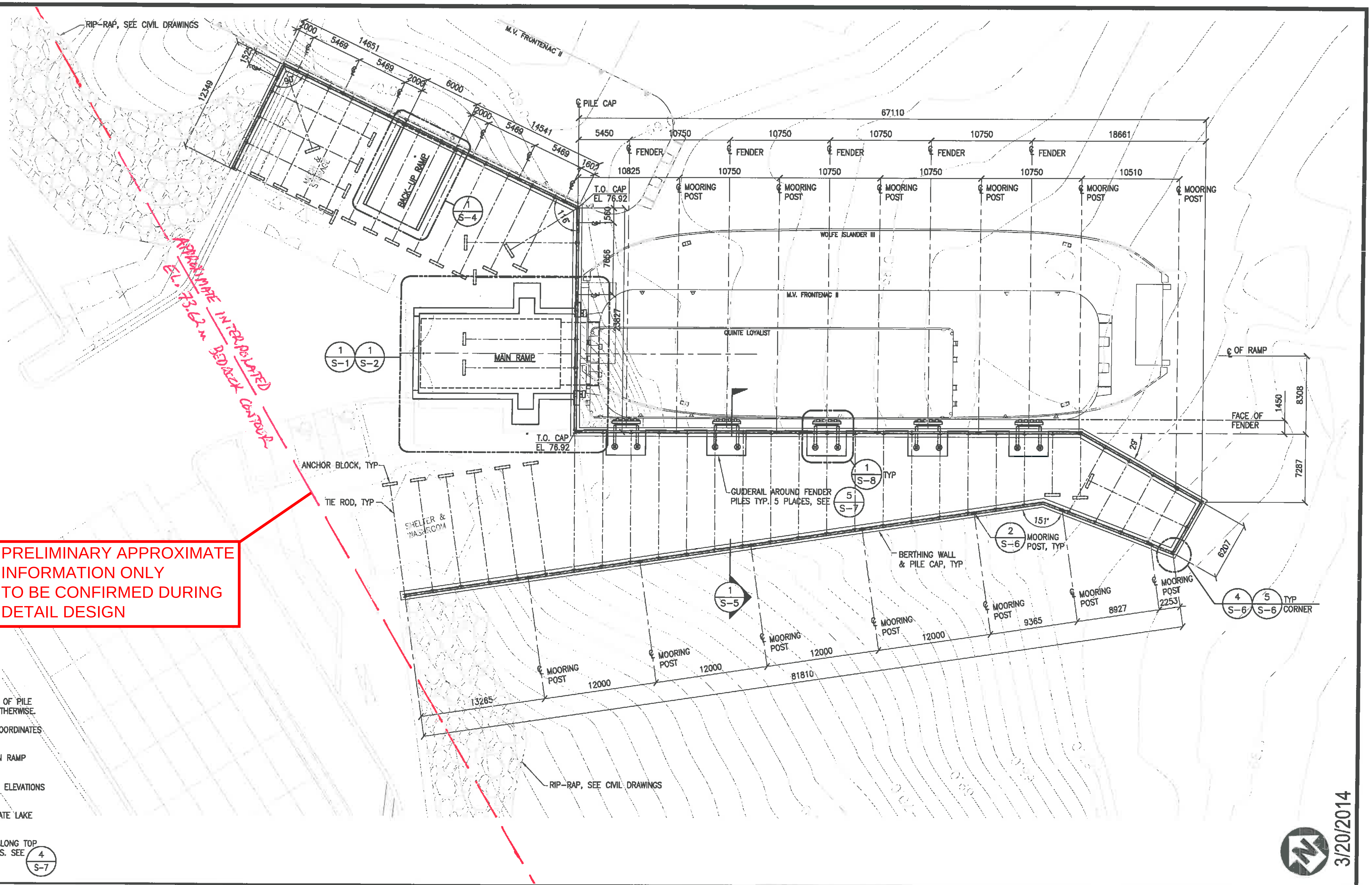
Tug boat docked at Stella dock from Borehole 13-05 (08/14/2013)

Jack-Up Barge at Borehole 13-06 (08/15/2013)

Jack-Up Barge at Borehole 13-06 (08/15/2013)

APPENDIX F

Preliminary Sketch of Bedrock at Bottom of Anchor Blocks



NOTES:

1. ALL DIMENSIONS TO CENTRE LINE OF PILE CAP AND PILES UNLESS NOTED OTHERWISE.
2. REFER TO CIVIL DRAWINGS FOR COORDINATES OF NEW WALL.
3. REFER TO SHEET S-2.1 FOR MAIN RAMP VESSEL PLANS.
4. FOR TOP OF PILE CAP (T.O. CAP) ELEVATIONS NOT SHOWN SEE CIVIL DRAWINGS.
5. CONTOURS SHOWN ARE APPROXIMATE LAKE BED ELEVATIONS.
6. PROVIDE CONTINUOUS GUIDERAIL ALONG TOP OF NEW TERMINAL BERTHING WALLS. SEE 4 S-7

3/20/2014

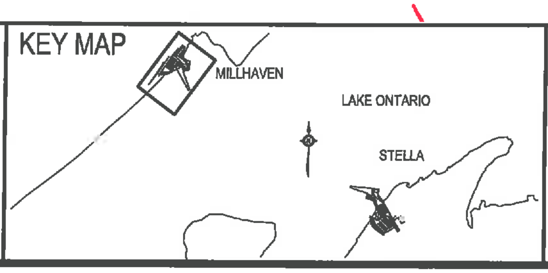
FILENAME: G:\Projects\Rhm\30990286 Amherst Ferry\500 Drawings\510 URS-HB\514 Terminal Concepts\0286_GA1.dwg
PLOTDATE: Mar 27, 2014 - 1:00pm

Ontario
Ministry of Transportation

URS

AMHERST ISLAND FERRY DOCKS
CONVERSION STUDY
G.W.P. 4067-09-00

KEY MAP

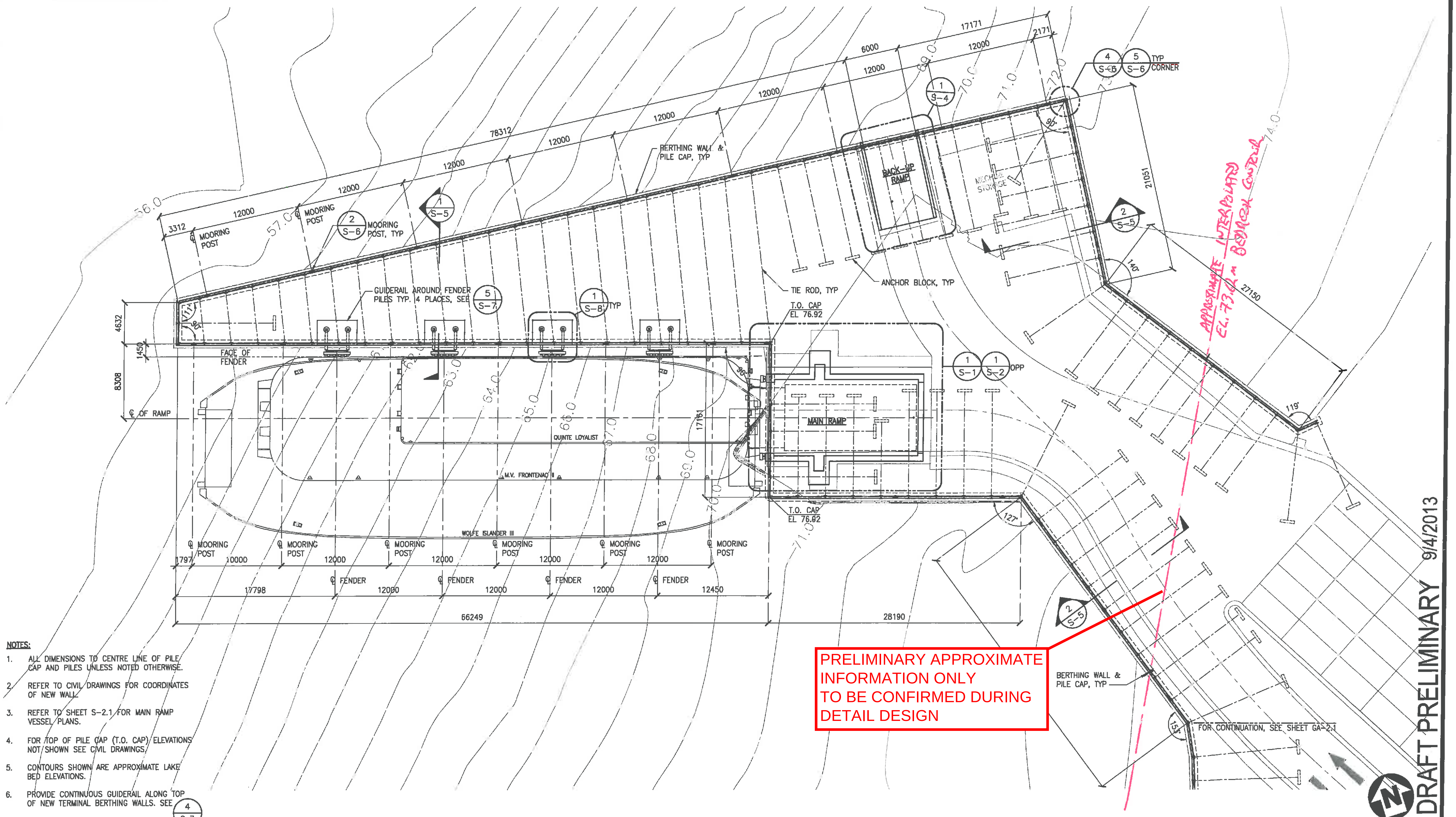


TERMINAL BERTHING WALLS
GENERAL ARRANGEMENT PLAN
MILLHAVEN SITE

0 2m 4m 8m 12m 16m 20m

PLATE

GA-1



PRELIMINARY APPROXIMATE
INFORMATION ONLY
TO BE CONFIRMED DURING
DETAIL DESIGN

DRAFT PRELIMINARY 9/4/2013

NOTES:

1. ALL DIMENSIONS TO CENTRE LINE OF PILE CAP AND PILES UNLESS NOTED OTHERWISE.
2. REFER TO CIVIL DRAWINGS FOR COORDINATES OF NEW WALL.
3. REFER TO SHEET S-2.1 FOR MAIN RAMP VESSEL PLANS.
4. FOR TOP OF PILE CAP (T.O. CAP) ELEVATIONS NOT SHOWN SEE CIVIL DRAWINGS.
5. CONTOURS SHOWN ARE APPROXIMATE LAKE BED ELEVATIONS.
6. PROVIDE CONTINUOUS GUIDERAIL ALONG TOP OF NEW TERMINAL BERTHING WALLS. SEE 4/S-7

REDUCED SET - HALF SIZE

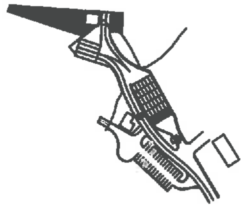
FILENAME: G:\Projects\Rhm\30990286 Amherst Ferry\500 Drawings\510 URS-HB\514 Terminal Concepts\0286_GA2.dwg
PLOTDATE: Sep 03, 2013 - 4:04pm

Ontario
Ministry of Transportation

URS

AMHERST ISLAND FERRY DOCKS
CONVERSION STUDY
G.W.P. 4067-09-00

KEY MAP



TERMINAL BERTHING WALLS
GENERAL ARRANGEMENT PLAN
STELLA SITE

0 2m 4m 8m 12m 16m 20m

PLATE

GA-2

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.

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