November 20, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 400/89 UNDERPASS REPLACEMENT
STRUCTURE SITE NO. 30-256
RECONSTRUCTION OF HIGHWAY 400/89 INTERCHANGE
TOWN OF INNISFIL, SIMCOE COUNTY
MTO G.W.P. 2438-13-00

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REPORT

Golder Associates

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PART A

FOUNDATION INVESTIGATION REPORT
HIGHWAY 400 / 89 UNDERPASS REPLACEMENT
STRUCTURE SITE NO. 30-256
TOWN OF INNISFIL, SIMCOE COUNTY
MTO, G.W.P. 2438-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed reconstruction of the Highway 400 / Highway 89 Interchange and Bridge Replacement project. The proposed works will include replacement of the existing Highway 89 underpass structure with a new underpass structure, in the Town of Innisfil, Simcoe County, Ontario.

The purpose of this investigation is to establish the subsurface soil and groundwater conditions at the proposed structure, including the associated approach embankments, by borehole drilling, in situ testing and geotechnical/analytical laboratory testing on selected soil samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated November 2, 2016, which forms part of the Consultant's Assignment Number 2015-E-0038 for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated April 25, 2017.

2.0 SITE DESCRIPTION

The Highway 400 / Highway 89 interchange is located about 20 km south of the City of Barrie in the Town of Innisfil, Ontario, as shown on the Key Plan on Drawing 1. Highway 400 consists of the three lanes of traffic in each northbound and southbound directions. Highway 89 is oriented in an east-west direction and consists of one lane of traffic in each direction, with turning lanes located near the Highway 400 on-ramps.

The northwest quadrant of the interchange consists of agricultural lands, and a closed highway service centre. All infrastructure, excluding lamp standards, has been removed from the footprint of the closed service centre. The northeast quadrant of the interchange consists of an open field containing a small area vegetated with trees and an industrial facility and yard. The southeast quadrant is occupied by a commuter parking lot and an outlet mall. The southwest quadrant is occupied by an area densely populated with trees and agricultural lands. Overhead power lines extend along the south side of Highway 89.

The pavement surface of Highway 400 varies from about Elevation 229 m to 229.5 m within the limits of the project, and the existing Highway 89 grade at Highway 400 is at about Elevation 235.4 m.

The existing Highway 89 approach embankments have side slopes that are inclined at approximately 2 horizontal to 1 vertical (2H:1V) or flatter. Based on observation of the approach embankments at the time of the borehole investigation, the side slopes appear to be performing adequately with no visual evidence of surficial sloughing or slope instability.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation

A preliminary foundation investigation for the Highway 400 / Highway 89 underpass was carried out by Golder in 2002, during which time a total of two boreholes, designated as Boreholes B1-1 and B1-2, were advanced near the east and west abutments of the underpass. The boreholes were advanced to depths of 28 m and 37 m, respectively, below ground surface and geotechnical laboratory testing was carried out on selected soil samples. The results of this investigation are contained in a report titled, "Preliminary Foundation Investigation and Design

Report, Highway 89 Underpass Structure Site 30-256, Highway 400 Widening from 1 km South of Highway 89 to Highway 11, G.W.P. 30-95-00", dated January 2002 (GEOCRES No. 31D-465).

The locations of the boreholes advanced during the 2002 investigation are shown on Drawing 1, and the borehole records, including a summary of the laboratory testing results from this investigation, are presented in Appendix A. The northing and easting coordinates relative to the MTM NAD 83 (Zone 10) coordinate system, the ground surface elevations referenced to Geodetic datum, and the drilled depths are presented below and on the borehole records in Appendix A.

Borehole No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
B1-1	4,895,635.8 (44.200617)	292,452.1 (-79.654489)	228.9	28.0
B1-2	4,895,623.5 (44.200506)	292,394.6 (-79.655208)	228.4	37.0

3.2 Current Investigation

Field work for the current foundation investigation was carried out between June 11 and August 15, 2017 during which time a total of nine boreholes, designated as Boreholes 89UP-01 to 89UP-08 and HF-02, were advanced near the location of the structure foundation footprints and high fill approach embankments as follows:

Foundation Element	Nearest Relevant Boreholes
West Approach Embankment	89UP-01
West Abutment	89UP-02 and 89UP-03
Center Pier	89UP-04 and 89UP-05
East Abutment	89UP-06 and 89UP-07
East Approach Embankment	89UP-08 and HF02

The locations of the boreholes are shown on Drawing 1 and the borehole records are provided in Appendix B. Lists of abbreviations and symbols are also provided in Appendix B to assist in the interpretation of the borehole and records.

Field work was carried out using a D-50 track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 203 mm outer diameter hollow stem augers and wash boring methods using 'NW' casing and a tricone. Soil samples were obtained at 0.75 m, 1.5 m and 3 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedures outlined in ASTM D1586. In situ field vane shear testing, using MTO standard "N"-sized vanes, was carried out to measure the undrained shear strength of cohesive soils (ASTM D2573, Standard Test Method for Field Vane Shear Test). Samples of the cohesive soils were obtained at selected locations using 76 mm outer diameter thin-walled Shelby tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples.

Groundwater conditions and water levels in the open boreholes/drill casing were observed during and immediately following drilling operations. A standpipe piezometer was installed in each of Boreholes 89UP-03 and 89UP-07 to permit monitoring of groundwater level at the borehole locations. The standpipe piezometers consists of a 50 mm diameter PVC pipe with a slotted screen sealed at a depth within the boreholes. Details of standpipe piezometer installations and water level readings are presented on the borehole records in Appendix B.

In addition to the field investigation program described above, Golder also carried out in-situ Pressuremeter testing (PMT) and Vertical Seismic Profiling (VSP) (in Boreholes PMT-01 and PMT-02) within the project limits. Details pertaining to this supplementary investigation and the results of the testing are presented in the report titled:

- "Foundation Investigation Report, High Fill Embankment, Highway 400/89 Interchange Reconstruction, Town of Innisfil, Simcoe County, Ministry of Transportation, Ontario, G.W.P. 2483-13-00" dated November 5, 2018, GEOCRES No. 31D-703, Report No 1668512-2.

Field work was observed by a member of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and geotechnical laboratory testing. All of the geotechnical laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate. Classification testing (i.e., water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. In addition, three, one-dimensional consolidation (oedometer) tests were carried out on selected samples of the clayey silt to silty clay deposit. The results of the geotechnical laboratory testing for the current investigation are presented in Appendix C.

Three selected soil samples were submitted, under chain-of-custody procedures, to Maxxam Analytics of Mississauga, Ontario (a Standards Council of Canada (SCC) accredited laboratory) for corrosivity testing. The soil samples were analyzed for a suite of parameters, including conductivity, resistivity, soluble chloride concentration, soluble sulphate concentration and pH. The results of the analytical analyses are presented in Appendix D.

Borehole locations and ground surface elevations were obtained using a GPS unit (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The locations provided on the borehole records and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) coordinate system and the ground surface elevations referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	**Location (MTM NAD 83 Zone 10)**		**Ground Surface Elevation (m)**	**Borehole Depth (m)**
	Northing (m) (Latitude, °)	**Easting (m) (Longitude, °)**		
HF-02	4,895,665.0 (44.200881)	292,504.8 (-79.653830)	227.5	35.7
89UP-01	4,895,618.4 (44.200459)	292,361.9 (-79.655616)	227.8	8.2
89UP-02	4,895,597.2 (44.200269)	292,389.9 (-79.655266)	235.4	50.8

Borehole No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
89UP-03	4,895,628.3 (44.200549)	292,375.2 (-79.655451)	227.4	49.2
89UP-04	4,895,619.3 (44.200469)	292,430.3 (-79.654761)	229.3	50.5
89UP-05	4,895,649.6 (44.200750)	292,418.6 (-79.654912)	229.2	50.4
89UP-06	4,895,621.9 (44.200493)	292,469.4 (-79.654271)	235.4	52.4
89UP-07	4,895,660.9 (44.200843)	292,451.0 (-79.654503)	227.2	50.6
89UP-08	4,895,655.5 (44.200795)	292,478.1 (-79.654165)	227.6	11.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This project area is located within the Peterborough Drumlin Field physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1894)[1]. The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins. Bedrock of Lindsay and Verulam Formations which underlies the Peterborough Drumlin Field consists mainly of fossiliferous limestone.

4.2 Subsurface Conditions

Detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the investigation and the results of the laboratory tests carried out on selected soil samples are presented on the borehole records provided in Appendix B. The results of the in situ field tests (i.e. SPT "N"-values and field vane) as presented on the Record of Borehole sheets and in sub-sections of Section 4.2 are uncorrected. The geotechnical laboratory testing plots are contained in Appendix C.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile and cross-sections on Drawings 1 and 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and in situ field vane tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the borehole records governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions.

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[1] Chapman, L.J. and Putman, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

In general, the subsurface conditions consist of a layer of topsoil or pavement structure underlain by granular fill, in turn underlain by an upper granular deposit of silt to silt and sand to silty sand. The upper granular deposit is underlain by an upper cohesive deposit composed of varved clayey silt to clay, underlain in places by a lower non-cohesive deposit and/or layers of silt to sandy silt to silty sand. The lower granular deposit is underlain by a lower cohesive clayey silt deposit and a deposit of glacial till that varies in composition from silt and sand to sandy clayey silt. A more detailed description of the subsurface conditions encountered in the boreholes during the previous and current field investigations is provided in the following sections.

4.2.1 Topsoil

A 200 mm to 460 mm thick layer of topsoil was encountered from ground surface in Boreholes 89UP-01, 89UP-03, 89UP-07, 89UP-08 and HF-02. The topsoil was classified based on visual and textural observations; organic content testing was not carried out.

4.2.2 Pavement Structure

Boreholes 89UP-02 and 89UP-06 advanced through the existing pavement structure on the westbound lane of Highway 89 and Boreholes 89UP-04 and 89UP-05 advanced on the northbound lanes of Highway 400 penetrated the pavement structure which comprised of asphaltic concrete ranging in thickness from approximately 165 mm to 250 mm. The asphaltic concrete is underlain by a layer of granular road base material consisting of sand and gravel and ranging in thickness from approximately 250 mm to 600 mm.

4.2.3 Fill

Granular fill was encountered underlying the pavement structure and topsoil at all borehole locations advanced for the proposed underpass structure and approach embankments, except at Borehole HF-02. The fill is variable in composition and generally consists of layers of gravelly sand, silt and sand, and silty sand. Organic odour was noted in Borehole 89UP-05 throughout the fill and clayey silt pockets were encountered below a depth of 7.2 m Borehole 89UP-06. The surface of the fill was encountered at Elevations 234.6 m and 234.8 m in Boreholes 89UP-02 and 89UP-06, respectively, both of which were advanced on Highway 89, and between Elevations 228.8 m and 225.7 m at the other borehole locations. In Boreholes 89UP-02 and 89UP-06, the fill extends to depths of 9.1 m and 10.2 m below ground surface (Elevations 225.3 m and 226.2 m), while the fill extends to depths between about 0.7 m and 3.0 m below ground surface (between Elevations 226.7 m and 224.6 m) at the other borehole locations.

The SPT “N”-values measured within the granular fill range from 3 blows to 56 blows per 0.3 m of penetration, indicating a very loose to very dense relative density.

The results of grain size distribution tests completed on seven samples of the granular fill are presented on Figure C-1 in Appendix C.

The water content measured in the fill deposit ranges from 10 per cent to 23 per cent, and field observations indicate moist to wet conditions.

4.2.4 Silt to Silty Sand (Upper Granular Deposit)

Underlying the topsoil and/or fill a non-cohesive deposit consisting of silt to sandy silt to silt and sand to silty sand was encountered in all boreholes between about Elevations 227.3 m and 224.6 m. The thickness of the deposit ranges from about 17.8 m to 21.3 m and the deposit extends to depths between about 20.9 m and 29.3 m (between

Elevations 209.1 m and 205.3 m) below ground surface. Boreholes 89UP-01 and 89UP-08 terminated within this deposit at a depth of 8.2 m (Elevation 219.6 m) and 11.3 m (Elevation 216.3 m) below ground surface, respectively.

The SPT "N"-values recorded within the non-cohesive deposit range from 6 blows to 80 blows per 0.3 m of penetration, indicating a loose to very dense compactness condition. Between about Elevations 228 m and 211 m, the SPT "N"-values generally range from about 6 blows to 72 blows and below about Elevation 211 m, the SPT "N"-values generally range from about 30 blows to 80 blows.

The results of grain size distribution testing completed on twenty-seven samples is shown on Figures C-2A, C-2B, C-3A to C-3B in Appendix C. The deposit generally contains trace to some gravel and trace clay and the silt portion contains trace to some sand. At some locations, the deposit contains clayey silt to silty clay interlayers which are between about 0.1 m to 2.3 m thick as described further in Section 4.2.5.

Atterberg limits testing carried out on five samples of the non-cohesive deposit measured liquid limits ranging from about 14 per cent to about 17 per cent, plastic limits ranging from about 12 per cent to about 15 per cent, and plasticity indices ranging from about 1 per cent to about 3 per cent, indicating that the fines portion of the silt to sandy silt layers of the deposit has slight plasticity as presented on the plasticity chart on Figure C-4.

Natural water content measured on samples of the silt to silty sand ranges from about 10 per cent to 24 per cent.

4.2.5 Clayey Silt to Silty Clay (Interlayers)

Boreholes 89UP-03, 89UP-07 and 89UP-08 penetrated approximately 0.3 m to 2.3 m thick layers of clayey silt to silty clay within the upper granular deposit. The grey clayey silt to silty clay is generally varved and contains trace to some sand. The surface of the clayey silt to silty clay layers were encountered between about Elevations 226.0 m and 209.6 m.

The SPT "N"-values measured within the cohesive layer in Boreholes 89UP-07 are 8 blows and 23 blows (measured at the interface of the silty clay layer and the underlying silt deposit) per 0.3 m of penetration, suggesting a stiff to very stiff consistency. A SPT "N"-value measured at the interface of the silty clay layer and underlying silt and sand deposit in Borehole 89UP-03 is 44 blows per 0.3 m of penetration, suggesting a hard consistency. A SPT "N"-value measured at the interface of the silty clay layer and overlying silt and sand deposit in Borehole 89UP08 is 15 blows per 0.3 m of penetration, suggesting a stiff consistency.

The natural water content measured on four samples of the clayey silt to silty clay deposit ranges from about 21 per cent to 25 per cent.

4.2.6 Clayey Silt with Sand to Clay (Upper Cohesive Deposit)

A varved cohesive deposit comprised of clayey silt with sand to clayey silt to silty clay to clay was encountered underlying the upper granular deposit in Boreholes B1-1, B1-2, 89UP-02 to 89UP-07 and HF-02. The cohesive deposit is generally varved (typically comprised of silty clay with thin clayey silt and silt laminae), but also includes homogenous zones of silty clay. The surface of the cohesive deposit was encountered at depths between about 20.9 m and 29.3 m (between Elevations 209.1 m and 205.3 m) below ground surface. The thickness of the cohesive deposit varies from about 9.6 m to 12.4 m and the deposit extended to between about Elevations 195.7 m and 194.0 m. Borehole B1-1 was terminated within this deposit at a depth of 28.0 m (Elevation 200.9 m) below ground surface.

The SPT “N”-values recorded within this deposit ranges from 0 blows (weight of hammer) to 40 blows per 0.3 m of penetration. In situ vane tests carried out within this deposit measured undrained shear strength ranging from about 11 kPa to greater than 96 kPa, but typically greater than 40 kPa. The sensitivity ranges from about 1 to 4, with the exception of Borehole 89UP-06 in which the sensitivity value was measured at about 7 at Elevation 200.8 m. The in situ field vane tests results together with the SPT “N”-values indicate that the clayey silt to silty clay deposit predominately has a firm to very stiff consistency, with the exception of the upper zone of the silty clay deposit encountered in Borehole B1-1 which has a hard consistency based on an SPT “N”-value of 40 blows per 0.3 m of penetration recorded at about Elevation 207.0 m.

The results of grain size distribution tests completed on three samples of the cohesive deposit are shown on Figure C-5 in Appendix C. The deposit generally contains trace to some gravel and trace sand to a sandy composition. As noted above, the cohesive deposit generally includes clayey silt and silt laminae as well as occasional sand inclusions, as shown on the photographs on Figure C-6.

Atterberg limits tests were carried out on sixteen samples of this cohesive deposit and measured liquid limits ranging from about 15 per cent to about 53 per cent, plastic limits ranging from about 13 per cent to about 21 and plasticity indices ranging from about 5 per cent to about 33 per cent. The results of the Atterberg limits tests are shown on the plasticity charts on Figures C-7A and C-7B in Appendix C indicate that the cohesive deposit can be classified as clayey silt of low plasticity to silty clay of intermediate plasticity to clay of high plasticity. The liquidity index of the varved cohesive deposit ranges from about 0.6 to 1.3.

Laboratory consolidation tests were carried out on three samples of the cohesive deposit obtained from Shelby tubes in Boreholes 89UP-03, 89UP-06 and 89UP-07. A preconsolidation stress ranging between about 335 kPa and 555 kPa was estimated from the void ratio versus logarithmic pressure plots and from the total work versus pressure plots. A bulk unit weight ranging between about 17.4 kN/m^3 and 18.8 kN/m^3 and a specific gravity between about 2.71 and 2.75 was measured on the consolidation test samples. The overconsolidation ratio (OCR) ranges from 1.0 to 2.2. The OCR value of 1.0 is estimated on a tested specimen recovered from Borehole 89UP-06 (Sample 24), which was advanced through the existing Highway 89 embankment where the effective vertical stress is higher in comparison to that for the other two samples tested, which are located outside the existing embankment footprint. Details of the test results are shown on Figures C-8A to C-8D, C-9A to C-9D and C-10A to C-10D in Appendix C, and the test results are summarized below.

Borehole and Sample No.	**Sample Depth / Elevation**	**σ_{vo}' (kPa)**	**σ_p' (kPa)**	**$\sigma_p' - \sigma_{vo}'$ (kPa)**	**OCR**	**C_c**	**C_r**	**e_o**	**c_v [1] (cm^2/s)**
89UP-03 Sample 20	24.7 m / 202.7 m	235	335	100	1.4	0.53	0.024	0.96	9.3×10^{-3} 1.7×10^{-3}
89UP-06 Sample 24	36.9 m / 198.5 m	410	410	~0	1.0	0.70	0.058	1.19	2.6×10^{-3} 1.5×10^{-4}
89UP-07 Sample 21	26.2 m / 201.0 m	250	555	305	2.2	0.52	0.022	0.91	6.0×10^{-3} 1.9×10^{-3}

Note:

1. Two coefficients of consolidation (c_v) have been presented for each sample. The first value (top line) is based on a stress range below the effective overburden stress (i.e., within the overconsolidated stress range). The second value (bottom line) is based on a stress range between the effective overburden stress and the final stress due to a 7.5 m high west approach embankment (applicable to Borehole 89UP03) and a 9.5 m high east approach embankment (applicable to Boreholes 89UP06 and 89UP07) (i.e., normally consolidated stress range).

where:
σ_{vo}' is the in situ vertical effective overburden stress in kPa
σ_p' is the preconsolidation stress in kPa
OCR is the overconsolidation ratio
e_o is the initial void ratio
C_c is the compression index
C_r is the recompression index
c_v is the coefficient of consolidation in cm^2/s

The natural water content measured on thirty-one samples of this deposit ranges from about 20 per cent to 44 per cent.

4.2.7 Silt to Silty Sand (Lower Granular Deposit)

Underlying the upper cohesive deposit in Boreholes B1-2, 89UP-02 to 89UP-07 and HF-02, a deposit consisting of silt to silt and sand to silty sand was encountered between Elevations 195.7 m and 194.0 m. The thickness of the deposit ranges from about 3.0 m to 6.5 m and the deposit extended to between about Elevations 190.9 m and 189.0 m. Boreholes HF-02 and B1-2 terminated within this deposit at depths of between 35.7 m and 37.0 m (at Elevations 191.8 m and 191.4 m) below ground surface, respectively.

The SPT "N"-values recorded in the lower granular deposit range from 27 blows to 100 blows per 0.3 m of penetration, indicating a compact to very dense compactness condition.

The results of grain size distribution tests completed on six samples of this deposit are shown on Figure C-11. The silt layers contain trace to some clay, trace to some sand, and the silt and sand layers contain trace clay. The deposit occasionally contains clayey silt inclusions and trace gravel.

The natural water content measured on fifteen samples of this deposit ranges from about 12 per cent to 24 per cent.

4.2.8 Sandy Clayey Silt to Clayey Silt (Lower Cohesive Deposit)

A lower cohesive deposit was encountered underlying the lower granular deposit in Boreholes 89UP-02 to 89UP-07. The surface of the deposit was encountered between about Elevations 190.9 m to 189.0 m, the thickness of the deposit ranges from between about 3.1 m to greater than 7.9 m and the deposit extended to between about Elevations 186.3 m and 182.9 m. Boreholes 89UP-02 and 89UP-06 were terminated within this deposit at a depth of 50.8 m (Elevation 184.6 m) and 52.4 m (Elevation 183.0 m), respectively.

The SPT "N"-values recorded within this deposit generally range from 15 blows to 77 blows per 0.3 m of penetration, with one value of 100 blows per 0.15 m of penetration. A SPT "N"-value measured in the upper portion of the clayey silt deposit in Borehole 89UP-06 at about Elevation 189.4 m is 0 blows per 0.3 m of penetration (weight of hammer) but two in situ vane tests carried out within this deposit between about Elevations 189 m and 188 m measured undrained shear strengths greater than 96 kPa. The two field vane tests results together with the SPT "N"-values suggest that the sandy clayey silt to clayey silt deposit has a very stiff to hard consistency.

The results of grain size distribution tests completed on five samples of this deposit are shown on Figure C-12. The clayey silt deposit contains trace sand to sandy and trace gravel.

Atterberg limits tests were carried out on seven samples of this deposit and measured liquid limits ranging from about 18 per cent to 24 per cent, plastic limits ranging from about 11 per cent to 16 per cent and a plasticity indices ranging from about 4 per cent to 8 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C-13 in Appendix C and indicate that the cohesive deposit can be classified as silt of slight plasticity to clayey silt of low plasticity.

The natural water content measured on ten samples of the deposit ranges from about 12 per cent to 22 per cent.

4.2.9 Clayey Silt with Sand to Clayey Silt Till / Silt and Sand to Silty Sand Till

A glacial till deposit was encountered underlying the lower cohesive deposit at Boreholes 89UP-03, 89UP-04, 89UP-05 and 89UP-07. The till deposit varies in composition from silt and sand to silty sand (i.e. granular till) to clayey silt with sand to clayey silt (i.e. cohesive till). The surface of the till was encountered between Elevations 186.2 m and 182.9 m and the boreholes were terminated within the till deposit at depths between about 49.2 m and 50.6 m (between Elevations 178.8 m and 176.6 m) below ground surface.

The SPT “N”-values measured within the granular till deposit range from 101 blows per 0.3 m of penetration to 100 blows per 0.08 m of penetration, indicating a very dense compactness condition. The SPT “N”-values measured within the cohesive till deposit range from 35 blows per 0.3 m of penetration to 100 blows per 0.05 m of penetration, suggesting a hard consistency.

Grain size distribution testing carried out on two samples of the granular till deposit are shown on Figure C-14 in Appendix C. The granular till consists of grey silt and sand to silty sand trace to some gravel, and trace clay. Grain size distribution of the three samples of the cohesive till deposit are shown on Figure C-15 in Appendix C. The cohesive till consists of grey clayey silt with sand to clayey silt, trace to some gravel. Inferred cobbles and boulders were encountered between about Elevations 182.9 m and 182.3 m in Borehole 89UP-05.

Atterberg limit tests were carried out on three samples of the cohesive till deposit and measured liquid limits ranging from about 15 per cent to 23 per cent, plastic limits ranging from about 10 per cent to 11 per cent, and a plasticity indices ranging from about 4 per cent to 12 per cent. The results of the Atterberg limits tests shown on the plasticity chart on Figure C-16 indicate that the cohesive till deposit can be classified as a clayey silt of low plasticity.

The natural water content measured in samples from eight samples of this till deposit range from about 7 per cent to 19 per cent.

4.2.10 Groundwater Conditions

The overburden samples obtained from the boreholes advanced during the previous and current investigations were generally moist to wet. The groundwater levels in the open boreholes or inside the drill casing were measured upon completion of drilling operations; however, the water levels in the drill casing does not necessarily reflect groundwater conditions on completion of drilling as water with drilling mud was used to advance Boreholes 89UP-02 to 89UP-07 and HF-02. Standpipe piezometers were installed in Boreholes 89UP-03 and 89UP-07 to permit monitoring of groundwater level at this site. The piezometers in Boreholes 89UP-03 and B1-2 are screened within the upper granular deposit and the piezometer in Borehole 89UP-07, is screened in the lower granular deposit. The water level in the standpipe piezometer installed in Borehole 89UP-07 was measured at a depth above the top of the lower granular deposit and, therefore, the groundwater level in this deposit is indicative of artesian condition, but not flowing above ground surface. Details of the piezometer installations and measured groundwater levels are shown on the borehole records in Appendix B. The groundwater levels recorded are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)	Comments
89UP-01	227.8	2.2	225.6	15/08/2017	Open borehole
89UP-03	227.4	3.3	224.1	21/07/2017	Open Borehole
		1.0	226.4	03/08/2017	Measured in standpipe piezometer
		1.0	226.4	10/08/2017	
		1.2	226.2	15/08/2017	
		1.3	226.1	19/09/2017	
		0.7	226.7	05/03/2018	
		0.5	226.9	16/05/2018	
89UP-07	227.2	7.9	219.3	02/08/2017	Open borehole
		0.7	226.5	10/08/2017	Measured in standpipe piezometer
		0.7	226.5	15/08/2017	
		0.9	226.3	19/09/2017	
		1.0	226.2	05/03/2018	
89UP-08	227.6	1.1	226.5	09/08/2017	Open borehole
HF-02	227.5	2.7	224.8	09/08/2017	Open borehole
B1-1	228.9	2.7	226.2	18/12/2000	Open borehole
B1-2	228.4	2.3	226.1	18/12/2000	Open borehole
		1.8	226.6	19/01/2001	Standpipe piezometer
		1.3	227.1	15/03/2001	

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.2.11 Analytical Laboratory Testing Results

Analytical testing was carried out on soil samples recovered from Boreholes 89UP-02, 89UP-05 and 89UP-06. The soil samples were submitted to Maxxam Analytics of Mississauga, Ontario for corrosivity testing. Detailed analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix D, and summarized below.

Borehole No.	Sample No.	Depth (m)	Parameters				
			Resistivity (ohm-cm)	Electrical Conductivity (µmho-cm)	Soluble Sulphate (SO_4) Content (µg-g)	Chloride (Cl) Content (µg-g)	pH
89UP-02	SS 15[1]	19.8 – 20.4	1,200	820	35	480	8.10
89UP-05	SS 1[1]	1.5 – 2.0	330	3,060	<20[2]	1700	7.54
89UP-06	SS 23[1]	33.5 – 34.1	2,600	387	220	<20[2]	8.12

Notes:

1. "SS" refers to a split-spoon sampler used to carry out the soil sampling in the boreholes.
2. The sulphate concentration (Borehole 89UP05 SS1) and chloride concentration (Borehole 89UP06 SS23) are below the reportable detection limit of 20 µg/g.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Katelyn Nero, and was reviewed Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng, a MTO Foundations Designated Contact and Senior Consultant with Golder conducted a technical review of the report and Ms. Lisa Coyne, P.Eng., Golder MTO Foundation Designated Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 400 / 89 UNDERPASS REPLACEMENT
STRUCTURE SITE NO. 30-256
TOWN OF INNISFIL, SIMCOE COUNTY
MTO, G.W.P. 2438-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

The section of the report provides foundation design recommendations for the construction of a new Highway 89 underpass (Structure Site No. 30-256). These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the field investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and carry out the design of the bridge foundations. The Foundation Investigation Report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Contractors must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

It is understood that a component of the Highway 400 / 89 interchange reconstruction / improvement involves the construction of a new underpass between Stations 9+961 and 10+039 of Highway 89, along a new alignment that is shifted approximately 30 m north of the existing Highway 89 alignment (centreline to centreline) to facilitate construction staging as well as to accommodate the future widening of Highway 400 and additional lanes along Highway 89.

Based on the General Arrangement drawing provided by Morrison Hershfield (MH) to Golder on September 19, 2017, the proposed Highway 89 underpass will consist of a two-span structure with a total span length of 78 m and a width of approximately 33.5 m, carrying three lanes of traffic in each direction. The proposed bridge will include integral abutments and wingwalls adjacent to the abutments.

It is further understood that grade raises of up to about 7.5 m and 9.5 m (at the centreline of the realigned Highway 89) are proposed at the west and east approach embankments, respectively. The pavement grade on the proposed underpass is at approximately Elevation 237.3 m.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* (CHBDC, 2014) and its *Commentary*, the proposed bridge and its foundation system are expected to carry medium to high traffic volumes and its performance will have potential impacts on other transportation corridors; hence, the structure is classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the typical project specific foundation investigation carried out at this site (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of *CHBDC* (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the *CHBDC* have been used for design.

6.3 Seismic Design

6.3.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations and vertical seismic profile (VSP) testing. Based on the VSP testing, the site may be classified as Site Class D.

6.3.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the *CHBDC* (2014), the peak ground acceleration (PGA) values, peak ground velocity (PGV) and design spectral acceleration (Sa) values for Site Class D are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475year return period)
PGA (g)	0.036	0.055	0.089
PGV (m/s)	0.038	0.060	0.096
Sa (0.2) (g)	0.062	0.093	0.143
Sa (0.5) (g)	0.053	0.076	0.116
Sa (1.0) (g)	0.033	0.048	0.073
Sa (2.0) (g)	0.016	0.025	0.039
Sa (5.0) (g)	0.0035	0.0058	0.0095
Sa (10.0) (g)	0.0015	0.0024	0.0040

6.4 Foundation Options

Based on the proposed structure configuration and the subsurface conditions encountered at this site, shallow foundation, “intermediate” foundation (defined below) and deep foundation options have been considered for support of the new abutments and centre pier.

- <u>Shallow Foundations</u>
  - **Spread/strip footings:** Shallow foundations comprised of spread or strip footings, “perched” within the bridge approaches and supported on a compacted Granular ‘A’ pad, are considered feasible for support of the new abutments, although this foundation type will preclude the use of integral abutments. However, due to the large loads imposed by the structure and the presence of generally compact and silty foundation soils, the geotechnical resistances may be marginal for the support of a conventional bridge deck. Spread/strip footings founded on the generally compact silt and sand fill (or on compacted granular fill following subexcavation of this material), which overlies generally compact non-cohesive native soils are also feasible for support of the new centre pier; however, this option would require groundwater control prior to foundation excavations and the geotechnical resistances will likely be insufficient for a conventional bridge deck design, unless ground improvement measures are implemented. Therefore, the shallow foundation option is not considered to be the preferred alternative at this site, particularly at the location of the new centre pier.

- **“Intermediate” Foundations**
  - **Helical piles, micropiles, continuous flight auger (CFA) piles and drilled displacement (DD) piles:** The wide variety of piles listed in this category is typically included in the subset of deep foundations. However, for categorization purposes at this site, deep foundations have been classified as the more “conventional” friction piles or end-bearing foundations (i.e., driven steel H-piles/tube piles and drilled shafts) that would be on the order of 30 m to 50 m long and extend beyond the extensive cohesive deposit. The “intermediate” foundations have been classified as foundations that have not been as widely utilized on MTO bridge projects but that may represent a shorter and more cost-effective alternative. For this site, all of the intermediate foundation options would be approximately 15 m long, to be founded within the generally dense to very dense portion of the upper silty/sandy deposit (i.e., above the extensive, varved cohesive deposit). Consideration could be given to installing longer piles (i.e., more than 20 m long) that would be founded within the varved cohesive deposit, although this option could result in excessive settlement of the piles. These foundations are considered feasible for support of the new abutments and centre pier. However, based on the bridge loading and anticipated requirements for pile spacing, the pile groups are anticipated to experience settlements that may exceed the structure’s serviceability limits (addressed in more detail in the following subsections). As such, this foundation option is not considered to be the preferred alternative at this site.

- **Deep Foundations**
  - **Driven steel H-piles/tube piles:** Steel H-piles or tube piles driven into the lower very dense non-cohesive deposit/lower very stiff to hard cohesive deposit or the till deposit, resulting in approximately 40 m and 50 m long piles, respectively, are considered feasible for support of the new abutments and centre pier. Driven piles would permit design of conventional and semi-integral abutments (for H-piles and tube piles) or integral abutments (generally for H-piles). However, it will be challenging to install battered piles at the location of the centre pier while maintaining traffic flow along Highway 400 due to the constrained working space along the centre median. The structural design could consider the use of vertical piles to address lateral loads at the centre pier, to mitigate the construction challenges associated with battered piles adjacent to live traffic lanes.

  - **Drilled shafts (caissons):** Drilled shafts (caissons) founded within the hard till deposit, resulting in approximately 45 m long drilled shafts, are also considered feasible for the support of the new abutments and centre pier; however, this option would also preclude integral abutment design. Drilled shafts can offer a narrower footprint for construction in constrained working areas, as compared with shallow foundations and driven/battered piles; and can be affixed directly to the underside of the superstructure, eliminating the need for foundation excavations and pile caps at the pier. If drilled shafts are adopted for support of the abutments and/or centre pier, permanent (rather than temporary) liners are likely to be required, and these will need to be advanced with water/bentonite/polymer drilling slurry inside the liners. Drilled shafts would be more expensive than shallow foundations and driven pile foundations at this site, particularly given the likely permanent liner requirements, although fewer drilled shaft elements would be required in comparison to the number of driven steel piles.

A more comprehensive summary of the advantages, disadvantages and risks for each foundation option, from a geotechnical/foundations perspective, is presented in Table 1 following the text of this report. The key challenges and considerations for the various foundation options are also discussed in greater detail within Sections 6.5 to 6.7.

Based on the above considerations, steel H-pile/pipe pile and drilled shaft foundations are considered the most feasible and practicable, from a geotechnical/foundations perspective, for support of the new abutments and centre pier; however, as mentioned above, steel H-piles (and potentially steel pipe piles) would permit integral abutment design, and are therefore considered advantageous from this perspective. Furthermore, steel H-piles are expected to be more economical and to be subject to fewer construction and constructability challenges than drilled shafts.

Given the estimated settlement associated with the extensive non-cohesive deposit and the underlying cohesive deposit encountered at the site, it is expected that the underpass structure may undergo serviceability and performance issues if a shallow foundation option is selected and the abutment footings are constructed prior to the approach fill placement. Installation of deep foundation elements at the abutments is similarly expected to be impacted by downdrag and drag loads if the approach fill placement occurs after the deep foundations are installed. In order to minimize the settlement below spread/strip footings, or to eliminate downdrag and drag loads exerted on the deep foundations, consideration should be given to implementing one of the downdrag mitigation measures (as described in Section 6.8) prior to installation of the piles for the abutment foundations units. Various options for accelerating or minimizing the settlement and drag loads are presented and should be considered in the context of the overall construction schedule.

6.5 Shallow Foundations

6.5.1 Spread / Strip Footings

Strip footings for the abutments may be constructed within the bridge approach embankments, founded on a compacted Granular ‘A’ pad having a minimum thickness of 2 m. At the centre pier, strip footings may be founded on the generally compact silt and sand fill, which extends to a depth of about 3 m below the pavement surface.

The factored ultimate and serviceability geotechnical resistances that may be used for the design of 3 m and 4 m wide strip footings at the abutments and centre pier are provided below.

Foundation Element	Founding Elevation	Founding Stratum	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)	Dewatering Required (Yes or No)
East and West Abutments	229.5 m (2 m above existing ground surface)	Compacted Granular ‘A’ pad (at least 2 m thick) over compact to dense silt to silty sand to silt and sand to silt	3 m	650 kPa	280 kPa	No
			4 m	725 kPa	240 kPa	
Centre Pier	227.7 m	Compact silt and sand fill or possible fill (1) over compact to dense silt to sandy silt to silt and sand to silty sand	3 m	590 kPa	215 kPa	Yes
			4 m	650 kPa	190 kPa	

Note:

1. It is noted that very loose to loose silt and sand fill was encountered in Borehole 89UP-05 (advanced near the northern limit of the proposed centre pier) based on the measured SPT ‘N’-values; however, the SPT ‘N’-values- are considered unrepresentative since they were measured at/near the interface of the groundwater level in silty soils, and likely experienced disturbance during sampling. This material has been interpreted as a fill to a depth of approximately 1.5 m below the highway surface, and as a possible fill to a depth of approximately 3 m below the highway surface, although these materials do appear consistent with the composition of the native silty soils.

The factored serviceability geotechnical resistances for the footings were assessed based on the load imposed by the footings within the upper portion of the non-cohesive deposit (i.e., within the zone of influence based on the vertical stress distribution with depth) and the corresponding immediate settlement resulting from the elastic compression of the non-cohesive soils within the footprint of the footings.

As discussed in Section 6.12, construction of the underpass will require placement of up to about 7.5 m and 9.5 m of fill within the limits of the west and east approach embankments, respectively. Consequently, shallow foundations at the abutments would have to be constructed after the preferred settlement mitigation option has been implemented within the footprint of the approach embankments/abutments in order to ensure that the immediate settlement and the majority of the primary consolidation settlement of the deep cohesive deposit has occurred to avoid excessive post-construction settlements below the abutment footings. The post-construction settlement at the centre pier will not be impacted by the fill placement at the abutments. The differential post-construction settlement between the abutments and the centre pier must be considered in the structural design if shallow foundations are adopted for one relative to a deep foundation-supported option for the other.

Resistance to lateral forces/sliding resistance between the cast-in-place concrete strip footings and the Granular ‘A’ pad at the abutments and the silt and sand fill at the centre pier should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The unfactored coefficient of friction (tan φ’) between the cast-in-place concrete strip footing and the subgrade may be taken as follows:

Foundation Element	Subgrade Material	tan φ’
Abutments	Compacted Granular A pad	0.70
Centre Pier	Compact silt and sand fill	0.58

Key Challenges and Considerations

- Due to the limited working space at the proposed centre pier, temporary protection systems will be required along the perimeter of the footing excavation. Temporary protection systems could consist of driven steel sheetpiles (which would serve to cut-off groundwater inflow from the sides of the excavation, if not the base), or soldier piles and timber lagging (a system that is not watertight, and which would require greater groundwater control measures to minimize the potential for loss of fine-grained soil particles through the lagging boards). Variations on protection systems, such as a slide rail shoring system, may also be feasible at this site.
- The groundwater level was measured at a depth as high as approximately 0.5 m below ground surface. Considering that foundation excavations for the strip footings are required to be founded at a minimum depth of 1.5 m below the ground surface (unless perched within the bridge approaches) for frost protection, dewatering measures will be required to construct the footings in dry conditions (refer to Section 6.16.7 for more details). The groundwater is required to be lowered 1 m below the founding elevation of the strip footing. The proposed groundwater drawdown will result in an increase in the vertical effective stress of the soils along the highway (and within the zone of depression) corresponding to about 15 kPa. Consequently, it is anticipated that the highway may experience between about 5 mm and 10 mm of settlement within the zone of influence of the dewatering (i.e., within the cone of depression). Alternatively, the footing excavation could be completed within a shored, water-filled excavation to balance the groundwater pressure at the excavation, with a concrete base / “plug” placed via tremie methods, to minimize or eliminate the requirement for

dewatering. Where dewatering measures are adopted, an appropriate strategy will be required to deal with the pumped water (i.e., on-site storage and disposal) at the location of the centre pier considering the constrained working space within the highway median.

- If a concrete tremie “plug” is not adopted at the base of the excavation and an effective dewatering operation is adopted, the founding silt and sand fill will be susceptible to loosening and disturbance due to water seepage, ponded water and construction traffic. It is recommended that a 100 mm thick concrete working slab/mud mat be placed on the subgrade within three hours to protect the integrity of the bearing stratum. An NSSP for working slab is included in Appendix E for inclusion in the Contract Documents.

6.6 “Intermediate” Foundations

Brief descriptions, including advantages and disadvantages, of the various types of intermediate foundations are provided in the following subsections. Based on the estimated pile group settlement for these intermediate foundation options (as presented in Section 6.6.5), and further to discussions with the MH and MTO team during the design evolution, these “intermediate” foundation options are not considered to be the preferred alternative at this site. Given that some of the foundation systems listed below involve a proprietary design and given that some of these foundation types have never been employed on MTO projects and would require a detailed assessment beyond the scope of work for this project, specific foundation recommendations (e.g., founding elevations, geotechnical resistances, etc.) have not been provided herein.

If one of the intermediate foundation options is pursued further, detailed analysis of the pile group using a specialty pile group software or a numerical modelling software utilizing explicit finite difference of finite element formulation will be required to analyze the pile soil interaction and reassess the estimated magnitudes of settlement. Input will be required from MH’s structural team to carry out the advanced modelling.

6.6.1 Helical Piles

A helical pile (or anchor) is a foundation system designed as an end-bearing pile, which is comprised of three main components as follows:

i) Bearing plate/helix – at least one plate/helix is required to transfer the load to the soil;

ii) Central shaft – a small diameter round or square shaft is required to transfer the axial load to the helical plates; and

iii) Termination – a termination (i.e., a bracket or attachment designed by the structural engineer) connects the structure/foundation unit and the top of the helical pile in order to transfer the load to the shaft/helical plates.

Helical piles are considered a proprietary foundation system due to variability in the use of pile materials and installation methods. Therefore, the design and verification of capacity of helical piles is the pile supplier’s responsibility.

Helical piles can generally be installed using small equipment, which may be advantageous in the relatively restricted working space at the proposed centre pier, and the installation method does not produce spoil. However, helical piles can be difficult to install in soils with SPT “N”-values greater than 50 blows per 0.3 m of penetration; difficult installation conditions may potentially be encountered at this site near the bottom portion of the upper non-cohesive deposit, where the soils were noted to be generally dense to very dense. It is also noted that the settlement and resistance of helical piles is largely installation-dependent. Furthermore, because helical piles have slender shafts, they can be subject to buckling in loose or soft to firm soils, although this challenge would likely not be applicable for the conditions at this site.

6.6.2 Micropiles

A micropile is defined as “a small diameter (typically less than 300 mm), drilled and grouted non-displacement pile that is typically reinforced” (FHWA, 2005). There are two types of micropiles:

i) Conventional micropile system; and

ii) Hollow bar micropile system.

The conventional micropile system is expected to be adequate at this site; however, a complete assessment of each type of micropile system would be required during detail micropile design stage, if this alternative were to be selected. The conventional micropile system involves the advancement of a borehole into the overburden using steel casing, and upon completion of drilling, a solid steel reinforcing bar is lowered to the bottom of the borehole and grouted in place for the length required to achieve the design axial capacity.

The construction equipment required to install micropiles occupies a much smaller footprint compared to equipment required to install driven steel piles or drilled shafts, which may be advantageous in the relatively restricted working space at the proposed centre pier. However, micropiles generally yield lower lateral resistances, and their feasibility would need to be confirmed by the structural designer. Further, the overall costs are generally higher as compared to other deep foundation installations.

6.6.3 Continuous Flight Auger (CFA) Piles

CFA piles are drilled foundation units that are constructed in three phases as follows:

i) Drilling phase – a continuous flight auger system is used to drill a hole to a design depth;

ii) Extraction phase – once the design depth has been reached, the auger is withdrawn from the hole, and at the same time, concrete or a grout mixture is pumped through a hollow opening at the bottom of the auger; and

iii) Reinforcement phase – steel reinforcement/cage is placed into the hole filled with concrete/grout (FHWA, 2007).

CFA pile diameters typically range between 0.4 m and 0.75 m and the pile lengths generally do not exceed 25 m. However, specialized equipment can be utilized to install CFA piles up to about 1.2 m diameter with lengths reaching 50 m; CFA piles can be installed in cohesive and non-cohesive soils below the groundwater level (however, soft clayey soils and loose sandy soils below the groundwater level can pose many difficulties. One of the main advantages of this type of foundation is that the piles can be advanced relatively quickly without the use of liners/sleeves. Additionally, this method of installation does not produce vibration/loud noise. However, soil cuttings generated during the extraction phase need to be managed.

6.6.4 Drilled Displacement (DD) Piles

DD piles are "rotary displacement piles that are installed by inserting a specially designed helical auger segment into the ground with both vertical force and torque" (Basu, P. et al., 2010). As the soil is displaced laterally, the void is filled with a concrete/grout mixture. DD piles offer similar advantages, disadvantages and limitations as CFA piles; however, some of the main differences are as follows:

i) DD piles perform better in loose sandy soils – the loose soils are densified during the installation phase;

ii) DD piles generate very small amounts of excess soil cuttings, which can be advantageous when installing piles at contaminated sites;

iii) DD piles would have a limited depth of installation in dense/hard soils due to high downward/retraction force and torque requirements; and,

iv) DD pile installation could affect adjacent utilities or sensitive structures and should be assessed in detail where such facilities are present.

DD pile diameters generally range between 0.3 m and 0.8 m and depend on the selection of the proprietary drilling tool used for installation of the piles. The pile lengths generally do not exceed 20 m and are largely dependent on the subsurface conditions due to drill rigs' force and torque limitations.

6.6.5 Settlement of Pile Groups

The various pile types listed above can generally be designed to withstand high axial loads and yield high ultimate geotechnical resistances, and the displacement a single pile may satisfy the serviceability component of design. However, the settlement of a pile group is likely much greater than the displacement of an individual pile and must be considered in foundation design. Based on preliminary pile group settlement analysis (i.e., utilizing empirical methods formulated by Vesic and Meyerhof (as outlined in CFEM, 2006) as well as the equivalent footing method for piles supported by shaft resistance in sand underlain by clay (Cheney and Chassie, 2000)) for a pile group at the proposed centre pier, with individual piles extending to approximately Elevation 214 m (i.e., approximately 14 m long piles under a pile cap embedded 1.5 m below the ground surface), the estimated magnitudes of settlement for intermediate pile groups are as follows:

Foundation Element	Estimated Magnitude of Settlement (Unfactored)				Estimated Magnitude of Settlement (Factored)			
	$\delta_{immediate}$	$\delta_{primary}$	$\delta_{secondary}$	δ_{total}[2]	$\delta_{immediate}$	$\delta_{primary}$	$\delta_{secondary}$	δ_{total}[2]
Centre Pier[1]	25 mm	10 mm	0 mm	35 mm	~30 mm	~15 mm	~0 mm	~45 mm

Notes:

1. Assuming 4 m wide by 33.5 m long pile group.
2. The total settlement (δ_{total}) is defined as the sum of the immediate settlement ($\delta_{immediate}$) due to elastic compression of the non-cohesive deposits as well as primary ($\delta_{primary}$) and secondary ($\delta_{secondary}$) settlements due to time-dependent consolidation of the cohesive deposits.

It is noted that the estimated magnitude of settlement does not include the elastic shortening of the piles and is comprised predominantly of compression of the upper non-cohesive deposit. As such, the majority of the settlement is expected to occur during and shortly after construction. It is further noted that some of the settlement is expected to occur during construction of the pile cap, column and potentially other parts of the superstructure which may tolerate some of the estimated magnitude of settlement. If this option were to be adopted, input would be required from MH's structural team to assess tolerable magnitudes of settlement during various stages of bridge construction.

6.7 Deep Foundations

6.7.1 Driven Steel H-Piles / Tube Piles

Driven steel H-piles and closed end, concrete filled steel tube piles founded below the extensive, upper cohesive deposit within the lower silt/sand/clayey silt deposit or the till deposit are considered feasible for support of the abutments and centre pier.

The factored ultimate and serviceability geotechnical resistances that may be used for the design of steel HP 310x110 piles, HP 310x132 piles and closed end, concrete filled 324 mm (12 ¾ inch) diameter steel tube piles having a minimum wall thickness of 9.5 mm (3/8 inch) are presented below. Two pile tip elevations have been provided per foundation element to allow the structural designer to optimize the foundation design from a structural and economical perspective.

Foundation Element	Approximate Pile Length [1]	Estimated Pile Tip Elevation	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)
West Abutment	39.5 m	192.5 m	Very dense silt and sand to silt (lower deposit)	1,250 kN	-- [2]
	51.0 m	181.0 m	"100-blow" clayey silt till / silt and sand till	1,600 kN	-- [2]
Centre Pier	35.0 m	192.5 m	Dense to very dense silt (lower deposit)	1,250 kN	-- [2]
	45.5 m	182.0 m	Hard clayey silt with sand till / very dense silty sand till	1,600 kN	-- [2]
East Abutment	39.5 m	192.5 m	Very stiff to hard sandy clayey silt / clayey silt (lower deposit)	1,250 kN	-- [2]
	51.0 m	181.0 m	"100-blow" clayey silt with sand till / silt and sand till	1,600 kN	-- [2]

Notes:

1. Estimated pile lengths based on pile cap elevations shown on MH's 60% Submission General Arrangement drawing dated July 2017
2. The factored geotechnical serviceability resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and, as such, the SLS condition does not apply.

The estimated factored ultimate geotechnical resistances provided above are based on a combination of shaft and tip resistance. As discussed in Section 6.4 drag loads will be imposed on the steel H-piles if they are driven prior to construction of the approach embankments. One of the downdrag mitigation measures discussed in Section 6.8 must be implemented at the abutments to eliminate drag loads being imposed on the piles. All pile installation/driving should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*).

Key Challenges and Considerations

- Driven piles will be very long at this site (i.e., on the order of 40 m to 50 m, depending on the selected pile tip elevation, unless the shorter "friction" pile is adopted), and it is expected that two splices per pile will be required for these longer lengths. Additional time will be required to weld the pile segments and carry out quality assurance. The effect of this operation on the overall construction schedule should be taken into account.

- Driving battered piles at the centre pier while maintaining traffic flow along Highway 400 near the work zone will be challenging due to the constrained working space along the Highway 400 median. Consideration should be given to resisting the lateral loads with vertical, not battered, piles.
- The lower cohesive deposit (i.e., clayey silt to silty clay and silt and sand till/clayey silt with sand till) contains gravelly/cobbley layers and possible boulders. These obstructions may affect the installation of deep foundations, and appropriate measures will need to be implemented. Where steel H-piles are adopted, the pile tips should be reinforced with driving shoes (refer to Section 6.16.9 for more details).
- Tube piles are generally not considered sufficiently flexible to be used in an integral abutment configuration, but this should be confirmed by the structural engineer.
- Consideration should be given to carrying out a full-scale pile load test at the site. The pile load test would provide an indication of the ultimate geotechnical resistance of the pile and would allow for the use of a higher geotechnical resistance factor, ϕ_{gu} (i.e., 0.6 instead of 0.4). Through discussions between MTO, MH and Golder during the detail design process, it is understood that the preference is to carry out the pile load test prior to construction; however, it is understood that this will not permit sufficient time to adjust the structural design of the pile layout. Conducting the pile load test would also allow for correlation with the results of the Hiley and PDA testing, which would support a more rapid decision process if the required resistances are not being achieved, which will lead to reduced delays and costs by the Contractor.

Piles that terminate at about Elevation 192.5 m will derive the majority of the resistance from the shaft and although cobbles and boulders are not anticipated in the upper granular deposit, the upper varved cohesive deposit or the underlying compact to dense silt to silt and sand deposit, it is recommended that the pile tips be equipped with a driving shoe/flange reinforcement or bearing points. It is also recommended that pile tip reinforcement be incorporated for piles driven to found within the “100-blow” soil to reduce the potential for damage to the pile during driving. In this regard, for piles that are driven to be founded within the “100-blow” soil, pile driving shoes (such as Titus standard “H” points or equivalent) are recommended over flange reinforcement.

Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile. The set criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should be verified in the field by the use of both the Hiley formula (MTO Standard Drawing SS103-11) and pile dynamic analyzer (PDA) testing during the final stages of driving to achieve an ultimate capacity. It is understood that MTO will include the PDA testing in the Contract Administrator (CA) assignment; however, the contractor should be made aware that PDA testing will be required. Special Provision SSP 903S06 *(High Strain Dynamic Testing, Deep Foundations – Amendment to OPSS 903)* should be included in the Contract Documents to address the requirement for PDA testing (see Appendix E). Based on MTO experience with the Hiley formula in Southern Ontario, a resistance factor equal to 0.5 may be used on the ultimate resistance to verify the factored ULS design values.

For piles driven to Elevation 192.5 m, the following note from MTO's Structural Manual should be shown on the Contract Drawing, based on the application of a resistance factor of 0.5 to the use of the Hiley formula (per MTO experience in Southern Ontario) and to the ultimate capacity as assessed by PDA testing:

- *Piles to be driven in accordance with Standard SS103-11 plus PDA testing using an ultimate geotechnical resistance of 2,500 kN per pile at the abutments and pier, but should be driven to no higher than 1.5 m above the design pile tip Elevation of 192.5 m.*

Alternatively, for longer end bearing piles driven to between Elevation 182 m and 181 m, the following note from MTO's Structural Manual should be shown on the Contract Drawing, based on a resistance factor of 0.5:

- *Piles to be driven in accordance with Standard SS103-11 plus PDA testing using an ultimate geotechnical resistance of 3,200 kN per pile at the abutments and pier, but should be driven to no higher than 1.5 m above the design pile tip of Elevation 181 m at the east and west abutment and Elevation 182 m at the pier.*

Assessment of ultimate geotechnical resistance by the Hiley formula and PDA testing should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula and/or PDA testing is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula and PDA testing should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48-hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation. An NSSP has been developed to amend OPSS.PROV 903 *(Deep Foundations)* to address the 48-hour wait period between initial driving and re-tapping, for inclusion in the Contract Documents (see Appendix E).

6.7.2 Drilled Shafts (Caissons)

The new abutments and centre pier for the proposed underpass structure may also be supported on drilled shafts (caissons) founded a minimum of 2 m within the "100-blow" till deposit. The factored ultimate and serviceability geotechnical resistances that may be used for the design are presented below.

Foundation Element	Approximate Pile Length [1]	Estimated Base Elevation	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)
West Abutment	51.0 m	181.0 m	Very dense silt and sand till / Hard clayey silt	6,000 kN	-- [2]
Centre Pier	46.5 m	181.0 m	Hard clayey silt with sand till / very dense silty sand till	6,000 kN	-- [2]
East Abutment	51.0 m	181.0 m	Very dense silt and sand till	6,000 kN	-- [2]

Notes:

1. Estimated pile lengths based on pile cap elevations shown on MH's 60% Submission GA drawing dated July 2017
2. The factored geotechnical serviceability resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and, as such, the SLS condition does not apply.

If drilled shafts are adopted, a permanent liner will be required to support the overburden soils from collapsing/sloughing during or after drilling operations (refer to Key Challenges and Considerations subsection below for a discussion regarding the requirement for a permanent liner).

Given that the above drilled shaft capacities have a significant end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the base of the drilled shaft. As such, the base of each drilled shaft excavation must be cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the very dense/hard till deposit or hard clayey silt deposit. A qualified geotechnical engineer should be retained during construction to inspect the drilled shafts to check that the conditions encountered are consistent with the information obtained from the boreholes and to confirm the base elevation of the drilled shaft and cleanliness. To allow for visual remote inspection of the base of the drilled shafts, which can be accomplished by means of a shaft inspection device (SID) such as a video camera, the drilled shaft excavations should be lined through the overburden. The liner must be maintained tight to the sides of the soil. The Contract Documents should include provisions for groundwater control to allow for inspection of the drilled caissons. Should the camera inspection indicate that loosened material is present at the base of the caissons, the base would need to be re-cleaned and re-inspected-.

Key Challenges and Considerations

- Considering the length of the drilled shafts (i.e., extending about 47 m below the Highway 400 grade) and high groundwater level, there is potential for the overburden soils to “set-up” following installation which will likely result in difficulties with the extraction of temporary liners. In addition, as a result of the high groundwater pressures within the granular deposits, there is the potential risk for “necking” to occur during the placement of the concrete using tremie methods and simultaneous extraction of temporary liners. The resulting reduced cross-sectional area of the drilled shaft would adversely impact the structural integrity of the cast-in-place reinforced concrete elements. Therefore, although it may be possible to use polymer slurries and/or temporary liners, it is recommended that the drilled shafts be constructed using permanent liners to minimize these risks.
- Placement of concrete using tremie methods will be required to construct the caissons.
- It is noted that the geotechnical resistances provided above are based on undisturbed conditions at the base of the drilled shafts. Given that the lower non-cohesive deposit is under high positive water pressure (i.e., water level measured at approximately 30.9 m above the base of the cohesive deposit in a standpipe piezometer with the screen sealed below the cohesive deposit), the permanent liners will need to be advanced with water/bentonite drilling slurry inside the liners to counterbalance the groundwater pressures and minimize disturbance at the founding level of the drilled shaft.

6.8 Downdrag and Drag Loads

As a result of the loading from the new approach embankments, long-term consolidation settlement of the underlying cohesive deposit will occur. Using the method of stress distribution as presented in Westergaard (1938) it is estimated that the immediate settlement of the upper granular deposit under the full height embankment loading at the abutment will be about 70 mm and the consolidation settlement will be between about 9 mm and 13 mm and that it will take about two months for about 80 per cent of the consolidation settlement to occur. With this amount of settlement downdrag loads are expected to be imposed on the piles. The difference in the vertical movement between the thick overburden (i.e., from the consolidation settlement and creep of the cohesive deposits) and the long piles (i.e., from the elastic deformation of the piles under the load from the bridge structure and from the punching of the piles into the soil deposit below the pile tip) will result in the development of negative skin friction and downdrag on the piles. Based on discussion by Fellenius and Broms (1969) and Poulos and Davis (1980) if there is between 2 mm to 3 mm of settlement this is sufficient movement between the soil and the pile to impose drag loads onto the piles.

Analyses to estimate drag loads and geotechnical resistances for the recommended pile foundation option at the abutments was carried out in accordance with Section 6.11.4.10 of *CHBDC and its Commentary* using the method proposed by Briaud and Tucker (1996). It is noted that the method used to assess the deformation of the pile and the associated drag load is dependent on a number of factors including the pile length, foundation conditions at the pile tip, the unfactored dead load on the pile and the anticipated post-construction settlement profile of the foundation soils. If any of these factors and/or the recommended embankment settlement mitigation option is different from those assumed in the analysis, the estimated drag loads and pile capacity need to be reassessed.

The position of the neutral plane is estimated to be just below the top of the upper varved cohesive deposit (i.e., at about Elevation 205.0 m) for the “shorter” piles that are driven to Elevation 192.5 m, and at about Elevation 192.0 m for the “longer” piles that are driven to Elevation 181 m. For design, an unfactored drag load of 1,500 kN may be used for the steel H-piles that terminate at Elevation 192.5 m and a unfactored drag load of 2,000 kN may be used for piles that terminate at Elevation 181 m.

If the piles for the abutments are installed prior to the construction of the approach embankments then in accordance with the requirements of the Canadian Foundation Engineering Manual (2006), an assessment is required to be carried out to estimate if the structural capacity of the steel H-pile would be exceeded when taking into account the factored dead load combined with the factored drag load. MH provided the structural capacity of various steel H-piles, the dead load acting on a single pile and the required factors that are applied to the dead load based on the CHBDC (2016). For the steel HP310x110 pile driven to Elevation 192.5 m, the factored dead load plus the factored drag load exceed the structural capacity of the pile.

Several measures to mitigate against downdrag were considered such as preloading, use of a pile section with a higher structural capacity, use of light-weight fill and high-grade steel piles. Preloading requires the full height embankment which extends a minimum distance of 20 m from the abutment be in place for a period of two months so that the remaining consolidation settlement is less than 2 mm to 3 mm. Further details on preloading are provided in Section 6.16.3. The following mitigation measures were considered to mitigate against downdrag:

- Preload for a period of two months and use HP 310x110 piles;
- Preload for a period of two months and use of a stiffer pile section such as a HP 310x132;
- No preload period required and use of HP 310x174 piles; however, they are required to be driven to Elevation 181 m to meet the SLS requirement for pile group settlement;
- Constructing the embankment using light-weight material such as cellular concrete to about 6 m above the grade of Highway 400 and then allowing a one month preload period, prior to driving steel HP 310x132.; and,
- No preload period required and use of HP 310x132 high grade steel pile however they are required to be driven to Elevation 181 m to meet the SLS requirement for pile group settlement.

A more comprehensive summary of the advantages, disadvantages relative costs and risks/consequences from a geotechnical/foundations perspective, is presented in Table 3, following the text of this report. In addition, input was provided by MH regarding the advantages and disadvantages from a structural perspective and this information is presented in Table 3 as well.

6.9 Frost Protection

Spread/strip footings and pile caps for all intermediate and deep foundation elements (i.e., H-piles, tube piles, drilled shafts, helical piles, micropiles, CFA piles and DD piles) should be provided with a minimum 1.5 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the footing or pile cap.

If adequate soil cover cannot be provided for the footing or pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration. However, if the depth of embedment for the footings is less than 1.5 m, the factored ultimate and serviceability geotechnical resistances would have to be re-evaluated.

6.10 Resistance to Lateral Loads

The design of piles and drilled shafts subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile/drilled shaft to the surrounding soil, the fixity condition at the head of the pile/drilled shaft (i.e., at the pile cap level), the structural capacity of the pile/drilled shaft to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile/drilled shaft and group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles, where possible.

The resistance to lateral loading in front of a single pile/drilled shaft may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equations below (*CFEM*, 1992 as referenced in the *Commentary of the CHBDC, 2014*). Additional assessment of the deformation response may be developed and provided in the form of p-y curves if the structural engineers' initial analyses suggest that the values derived from subgrade reaction theory do not adequately characterize the response for this structure site.

For non-cohesive soils:

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

where:
n_h = coefficient related to soil density (kPa/m)
z = except for the loose sand within the CSP where z is the depth (m) below the top of the CSP; and,
B = pile/drilled shaft diameter or width (m)

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where:
s_u = undrained shear strength of the soil (kPa)
B = pile/drilled shaft diameter or width (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native overburden, to be used for the structural analysis of the piles or drilled shafts at this site are summarized below.

Foundation Elements	Soil Unit	Elevation	n_h	s_u
Abutments and Centre Pier	Loose sand inside CSP at the abutments (H-Pile option only)	231.8 m to 228.8 m	3,000 kPa/m	--
	Very loose to compact silt and sand fill at centre pier (above groundwater level)	228.8 m to 226.4 m	5,000 kPa/m	--
	Loose to compact silt to silty sand at abutments (above groundwater level)	228.8 m to 226.4 m	10,000 kPa/m	--
	Generally compact silt to silt and sand to silty sand	226.4 m to 211.0 m	15,000 kPa/m	--
	Generally dense to very dense silt to silt and sand to silty sand	211.0 m to 206.5m	25,000 kPa/m	--
	Generally firm to very stiff clayey silt to silty clay to clay	206.5 m to 195.0 m	--	75 kPa to 90 kPa (Refer to Figure 2)
	Compact to very dense silt to silt and sand to silty sand	195.0 m to 189.8 m	20,000 kPa/m	--
	Very stiff to hard sandy clayey silt to clayey silt	189.8 m to 184.1 m	--	150 kPa
	Hard clayey silt with sand to clayey silt till / Very dense silt and sand to silty sand till	184.1 m to 176.7 m	32,500 kPa/m	--

For a single H-pile, tube pile filled with concrete, and drilled shaft, the estimated factored ultimate geotechnical resistance and factored serviceability geotechnical resistances for 10 mm of factored horizontal deflection at the pile caps are presented below. These values are based on analyses carried out using the commercially available program LPILE Plus (Version 5.0), developed by Ensoft Inc.

Foundation Element	Deep Foundation Unit	Axial Load Applied at the Top of Pile	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 10 mm of Deflection
Abutments [1, 3]	HP 310 x 110 pile	650 kN	165 kN	17 kN
	HP 310x132 pile	780 kN	185 kN	18 kN
	324 mm dia. tube pile	650 kN	70 kN	45 kN

Foundation Element	Deep Foundation Unit	Axial Load Applied at the Top of Pile	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 10 mm of Deflection
	1.2 m dia. drilled shaft	2,500 kN	500 kN	360 kN
Centre Pier [1, 2]	HP 310 x 110 pile	650 kN	300 kN	65 kN
	HP 310x132 pile	780 kN	320 kN	70 kN
	324 mm dia. tube pile	650 kN	75 kN	45 kN
	1.2 m dia. drilled shaft	2,500 kN	520 kN	400 kN

Notes:

1. Analysis assume that the steel H-piles at the abutments are oriented for weak axis bending.
2. Analysis assume that the steel H-piles at the centre pier are oriented for strong axis bending.
3. Analyses assume a pinned head condition at the abutments and centre pier.

Based on the above, both the structural and geotechnical resistances of the piles/drilled shafts should be evaluated to establish the governing case at ultimate limit state (ULS). At serviceability limit state (SLS), the horizontal resistance of the piles/drilled shafts will be controlled by deflections, and the horizontal resistance of the piles/drilled shafts should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should correspond to a factored horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments and centre pier (see Section C6.11.2.2.2 of the *Commentary to the CHBDC, 2014*).

The upper zone of the soil (down to a depth below the pile cap equal to about 1.5$\cdot B$ (after Broms, 1964, where B is the pile/drilled shaft diameter) should be neglected in the calculation of lateral resistance of the pile/drilled shaft to account for disturbance effects during installation.

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2014).

6.11 Lateral Earth Pressures for Design of Abutments and Wingwalls

The lateral earth pressures acting on the abutment walls and wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of 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 the design of the abutment and wingwalls.

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular ‘A’ or Granular ‘B’ Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls*, *Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall in accordance with Figure C6.20(a) of the *Commentary to the CHBDC* (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.20(b) of the *Commentary to the CHBDC* (2014).

6.11.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- For a restrained wall, the pressures are based on the proposed new embankment fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material for the general embankment fill:

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At Rest, K_o	Active, K_a
Earth Fill / Select Subgrade Material	20 kN/m^3	0.47	0.31

- For an unrestrained wall, the pressures are based on the granular fill in the backfill zone, and the following parameters (unfactored) may be used:

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At Rest, K_o	Active, K_a
Granular 'A'	22 kN/m^3	0.43	0.27
Granular 'B' Type II	21 kN/m^3	0.43	0.27

- If lightweight fill (EPS) is installed behind the abutment wall, the pressure acting over the depth of the EPS may be calculated as follows:

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At Rest, K_o	Active, K_a
Lightweight Fill (EPS)	0.5 kN/m^3	0.11	0.11

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary to the CHBDC*, 2014.

- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.11.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining and wingwalls in accordance with Section 4.6.5 of the *CHBDC* (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the *CHBDC* (2014) and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Wall Type	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, K_{AE}		
			Granular 'A'	Granular 'B' Type II	SSM
Yielding Wall	475-Year	0.036g	0.26	0.26	0.29
	975-Year	0.055g	0.26	0.26	0.29
	2,475 Year	0.089g	0.27	0.27	0.30
Non-Yielding Wall	475-Year	0.036g	0.27	0.27	0.30
	975-Year	0.055g	0.28	0.28	0.31
	2,475-Year	0.089g	0.30	0.30	0.33

- The K_{AE} value for a yielding wall is applicable provided that the wall can move up to $250 \cdot k_h$ (measured in mm), where k_h is the site specific PGA as given in the table above. This corresponds to displacements of 9 mm, 14 mm, and 22 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined per Section C4.6.5 of the *Commentary* to *CHBDC* (2014).

6.12 Approach Embankment Design

As outlined in Section 6.1, the proposed Highway 89 grade at the new underpass will be at approximately Elevation 237.3 m, requiring placement of up to about 7.5 m and 9.5 m of fill to construct the west and east approach embankments, respectively.

6.12.1 Global Stability

The following subsections outline the method used to evaluate static global stability of the proposed approach embankments. The geotechnical soil parameters used in the analyses are also presented. The results of the stability analyses are presented in Section 6.12.3 where they are discussed together with the results of the settlement analyses and recommendations regarding possible design and construction alternatives to mitigate post-construction settlement.

6.12.1.1 Method of Analysis

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. Morgenstern-Price is a general method of slices which is based on equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_{gu}. (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, minimum Factors of Safety of 1.3 and 1.5 have been used for the design of the embankment slopes for the short-term/temporary and long-term/permanent conditions, respectively, as per Table 6.2 of *CHBDC* (2014).

6.12.1.2 Parameter Selection

The simplified stratigraphy together with the foundation engineering parameters for the different soil types encountered at the site are summarized in Table 2. The following is a summary of the embankment slope inclination, unit weight and effective friction angle for new earth fill and new granular fill modelled in the slope stability analyses.

Fill Type	**Recommended Slope Inclination**	**Unit Weight, γ**	**Effective Friction Angle, ϕ’**	**Cohesion, c'**
Earth Fill ^{1}	2H:1V	20 kN/m^3	32°	0 kPa
Granular Fill ^{1}	2H:1V	21 kN/m^3	45°	0 kPa

Note:

1. The overall strength of the earth fill is lower compared to the granular fill. As such, approach embankments constructed predominantly with earth fill represent the worst case scenario in terms of global slope stability of the embankments. All slope stability figures presented in this report illustrate embankments using earth fill, where feasible.

For the non-cohesive soils present at this site, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). A plot of SPT “N”-values (corrected to N_{60} based on automatic hammer energy) measured within each deposit at the site is summarized in Figure 2. The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions. As shown on Figure 1 and in Table 2, the upper granular deposit has been sub-divided into two layers as follows:

i) from the top of the granular deposit to about Elevation 214 m the corrected SPT “N”-values range from about 6 blows to 75 blows per 0.3 m of penetration and on average the “N”-value is 25 blows, and;

ii) below Elevation 214 m the range of the corrected SPT “N”-values is narrower, as the corrected SPT “N”-values range from about 20 blows to 75 blows per 0.3 m of penetration and on average the “N”-value is 45 blows.

For cohesive deposits, total stress parameters were employed in the analyses assuming short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in situ field vane shear tests, inferred from the laboratory consolidation test results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content, liquid limit, etc.), where appropriate. A plot of the undrained shear strength versus elevation is shown on Figure 2.

Effective stress parameters were also employed to evaluate the stability of the embankments based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle (ϕ’) and effective cohesion (c’)) for the cohesive deposits were estimated from empirical correlations based on the plasticity index. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 226.4 m, which is based on the highest piezometric groundwater level measured in Borehole 89UP-03.

6.12.2 Settlement

The following subsections outline the methods used to carry out the settlement analyses at the proposed approach embankments for the realigned highway. The results of the analyses are presented in Section 6.12.3 where they are discussed together with the results of the stability analyses and recommendations regarding potential design and construction alternatives to mitigate stability issues and/or post-construction settlement, where applicable.

6.12.2.1 Method of Analysis

To estimate the magnitude of expected settlement, analyses were carried out at the west and east approach embankments. Settlement analyses were carried out using the commercially available program Settle^{3D} (Version 4.0), developed by Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on Westergaard's (1938) solution.

The sources of settlement are considered to include the following:

- Immediate settlement of the granular soils (short-term);
- Primary time dependent consolidation of the cohesive deposits (using Terzaghi’s one-dimensional consolidation theory long-term); and,
- Secondary time dependent (creep) consolidation of the cohesive deposits (long-term).

The thickness of the compressible foundation soils and the height of the approach embankments vary along the proposed Highway 89 alignment, and as such the settlements along the length of a highway alignment will similarly vary; however, the settlements estimated from the settlement analysis represent the maximum anticipated value along a given section of the Highway 89 alignment. Where the embankment height decreases the amount of settlement will be less than that estimated based on the critical embankment height which is at the approach abutments.

6.12.2.2 Parameter Selection

The simplified stratigraphy together with the deformation and time-rate consolidation parameters, where applicable, employed for the different soil types encountered at the site are summarized in Table 2. The parameters associated with the extensive cohesive deposit encountered at the site are presented on Figure 2 and are based on the graphical presentation of this data the upper varved cohesive deposit was sub-divided into multiple layers based on the preconsolidation stress (function of OCR), void ratio, compression index/recompression index.

The immediate compression of the non-cohesive deposits (i.e., silt, sandy silt, silt and sand to silty sand) were modelled by estimating an elastic modulus of deformation based on the SPT “N”-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were also compared with the typical range of expected values for similar soil types, as outlined in Section C6.9.3.6 of the *Commentary to the CHBDC* (2014) and adjusted, if necessary. However, the results of in situ Pressuremeter (PMT) and Vertical Seismic Profile (VSP) testing carried out within the project limits were used in order to refine the deformation parameters (i.e., modulus of elasticity or Young’s modulus, E’) of the upper granular deposit, which has a significant impact on the estimated magnitude of settlement below the high fill approach embankments.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation test, where appropriate, and in situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the laboratory index tests were employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Azzouz et al. (1976), Koppula (1986), Kulhawy and Mayne (1990), Nishida (1956) and Terzaghi and Peck (1967).

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests and/or estimated from the U.S. Navy (1986) correlation with liquid limit assuming normally consolidated or over-consolidated soils, as applicable.

In addition to primary consolidation within the cohesive deposits, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress.

For the purpose of the settlement analyses, the groundwater level was assumed to be at Elevation 226.4 m, which is based on the highest piezometric groundwater level measured in Borehole 89UP-03.

6.12.2.3 Settlement Performance

The settlement performance criterion for design of approach embankments is in accordance with MTO’s “Embankment Settlement Criteria for Design”, dated July 2, 2010. In general, embankments approaching structural elements such as a bridge abutment are to be designed as follows:

- Total settlements and differential settlement rates are not to exceed 25 mm, over a 15-year period following completion of construction for a secondary highway.

6.12.3 Results of Analyses

6.12.3.1 East Approach Embankment

The stability analyses for the east approach embankment indicate that after completion of construction, the embankment will have a Factor of Safety of 1.2 during the long-term/permanent condition for deep-seated, global failure surfaces of the front slope (2H:1V) / abutment configuration that would impact the operation of the highway (see Figure 3A). In order to achieve a Factor of Safety equal to or greater than 1.5, granular backfill must extend at least 6 m behind the abutment stem wall and from highway grade down to the toe of embankment. The granular backfill can be placed as a block extending perpendicular to the back of the abutment stem wall (see Figure 3B) or a wedge extending upwards from the bottom of the embankment to the highway grade at an inclination of 1.5H:1V (see Figure 3C).

Where RSS walls are present along the side slopes of the approach embankment, in order to achieve a Factor of Safety equal to or greater than 1.5, the length of the reinforcing strips associated with the proposed RSS walls must be at least 1.2 times the height of the respective RSS walls (see Figure 3D). These analyses and factors of safety are based on an approximately 9.5 m high embankment comprised predominantly of earth fill (excluding the aforementioned granular backfill zones, where adopted) constructed following subexcavation of topsoil/existing fill and any surficial organic/deleterious material and replacement with SSM, earth fill or granular fill.

Based on the results of the settlement analysis (with the topsoil, existing fill and any organic/deleterious materials subexcavated and replaced with SSM, earth fill or granular fill), the factored settlement of the foundation soils under the loading imposed by a 9.5 m high embankment is estimated to be about 140 mm. The estimated total factored settlement is comprised of about 115 mm of immediate factored settlement due to compression of the non-cohesive deposits, and about 25 mm of factored primary consolidation settlement of the approximately 10 m thick clayey silt to silty clay deposit.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is estimated to be about 5 mm over a 15-year period following completion of construction.

The majority of the total settlement (i.e., immediate settlement of the non-cohesive soil deposits) is expected to occur during or shortly after completion of construction and the total factored post-construction settlement is estimated to be about 25 mm. However, considering that preloading for a period of 60 days is required along the east approach embankment to mitigate against drag loads imposed on the steel H-piles at the east abutment (as described in Section 6.16.3), the total factored post-construction settlement is estimated to be about 10 mm.

6.12.3.2 West Approach Embankment

The stability analyses for the west approach embankment indicate that after completion of construction, the embankment will have a Factor of Safety less than 1.5 during the long-term/permanent condition for deep-seated, global failure surfaces of the front slope that would impact the operation of the highway (see Figure 4A). In order to achieve a Factor of Safety equal to or greater than 1.5, granular backfill must extend at least 5 m behind the abutment stem wall and from highway grade down to the base of the abutment wall. The granular backfill can be placed as a block extending perpendicular to the back of the abutment stem wall (see Figure 4B) or a wedge extending upwards from the bottom of the abutment wall to the highway grade at an inclination of 1.5H:1V (see Figure 4C). Furthermore, in order to achieve a Factor of Safety equal to or greater than 1.5 for the side slope of the approach embankment, the length of the reinforcing strips associated with the proposed Retained Soil System (RSS) walls must be at least 1.2 times the height of the respective RSS walls (see Figure 4D). These analyses and factors of safety are based on an approximately 7.5 m high embankment comprised predominantly of earth fill (excluding the aforementioned granular backfill zones) constructed following subexcavation of topsoil/existing fill and any surficial organic/deleterious material and replacement with SSM, earth fill or granular fill.

Based on the results of the settlement analysis (with the topsoil, existing fill and any organic/deleterious materials subexcavated and replaced with SSM, earth fill or granular fill), the factored settlement of the foundation soils under the loading imposed by a 7.5 m high embankment is estimated to be about 105 mm. The estimated total factored settlement is comprised of about 80 mm of immediate factored settlement due to compression of the non-cohesive deposits and about 25 mm of factored primary consolidation of the cohesive deposit.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is estimated to be negligible over a 15-year period following completion of construction.

The majority of the total settlement (i.e., immediate settlement of the non-cohesive soil deposits) is expected to occur during or shortly after completion of construction and the total factored post-construction settlement is estimated to be about 25 mm. However, considering that preloading for a period of 60 days is required along the east approach embankment to mitigate against drag loads imposed on the steel H-piles at the east abutment (as described in Section 6.16.3), the total factored post-construction settlement is estimated to be about 15 mm.

6.13 Liquefaction Potential Below Embankments

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e. leading to potentially large surface deformations) and under undrained conditions generate excess pore water pressures. The excess pore water pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e. analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with that presented in the *Commentary to the CHBDC, 2014*. It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using in situ testing data collected at the borehole locations. The design groundwater level was determined based on the highest measured groundwater level in the standpipe piezometer installed in Borehole 89UP-03 at about Elevation 226.4 m (measured on August 3 and 10, 2017). The CRR with depth was calculated at each borehole location using the parameter, $(N_1)_{60cs}$, that is based on the SPT “N”-value obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment indicate that the silts and sands at the site are not considered liquefiable during the 2,475-year design earthquake.

6.14 Retained Soil System (RSS) Walls

6.14.1 Founding Elevations

Based on the provided General Arrangement drawing, the Highway 89 underpass will involve the integration of wingwalls and retaining walls on both sides of each abutment, and retaining walls in front of the integral abutment stem walls. Each wingwall will consist of a 5.8 m wide cast-in-place concrete wall cantilevered off the back face of the abutment stem wall. This structural element will incorporate the coat of arms of Ontario imprinted on the face of the wall. It is understood that the bottom portion of this section of the wing wall will be embedded 1.5 m below ground surface for frost protection. In addition to the cast-in-place concrete wingwalls, RSS retaining walls will be required; these RSS walls will step upward into the embankment beyond the wingwalls. It is also understood that approximately 2 m high RSS walls will be utilized to construct partially false abutments. The RSS walls will be constructed below the 3.3 m high integral abutment stem walls that are supported on a single row of steel H-piles. The RSS walls constructed behind the wingwalls will be about 2.8 m long, and the RSS walls in front of the abutment walls will be about 33.5 m long. The RSS wall details are summarized below.

Retaining Wall Segment	Approximate Length of Wall	Approximate Height of Wall	Approximate Founding Elevation	Founding Soil
West Abutment				
Front of abutment stem wall	33.5 m	2 m	229.8 m	New embankment fill (earth fill, SSM or granular fill)
North and south side of abutment stem wall	2.8 m	2 m	229.8 m	
Back end of cast-in-place wing wall (first step)	2.2 m	3.3 m	233.8 m	
Western end of wing wall (second step)	2.2 m	2.3 m	234.9 m	
East Abutment				
Front of abutment	33.5 m	2 m	229.8 m	New embankment fill (earth fill, SSM or granular fill)
North and south side of abutment	2.8 m	2 m	229.8 m	
Back end of cast-in-place wing wall (first step)	2.6 m	3.8 m	233.4 m	
Eastern end of wing wall (second step)	2.6 m	2.5 m	234.7 m	

The front facing panels should be supported on a footing constructed on a granular pad. The granular pad should consist of a minimum thickness of 0.3 m of compacted Granular ‘A’ material, which should extend at least 1 m beyond the outside edge of both sides of the facing footing, then outward/downward at an inclination of 1H:1V.

6.14.2 Geotechnical Resistances

For the RSS facing panels supported on a 0.3 m wide footing constructed on a compacted granular pad as described in Section 6.14.1, the following geotechnical resistances may be used for design:

Foundation Unit	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
Footing on a 0.3 m thick compacted Granular 'A' pad	250 kPa	N/A [1]

Note:

1. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and as such, the serviceability condition does not apply.

Assuming that the RSS walls act as a unit and utilize the full width of the reinforced soil mass, which is assumed to be 1.4 times the height of the wall for RSS walls in front and at the sides of the abutment stem walls (i.e., 2.8 m long) and 1.2 times the height of the wall for RSS walls at the end of the wing walls – the recommended ratios of the length of the reinforcing strips to the height of the walls are relatively large in order to ensure stability of the approach embankments as described in Section 6.14.5 and are founded on the new embankment fill, the following geotechnical resistances may be used for design:

Retaining Wall Segment	Minimum Reinforcing Strip Ratio	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
RSS walls in front and at the sides of abutment stem walls	1.4H	425 kPa	175 kPa
RSS walls at the end of the wing walls	1.2H	450 kPa	150 kPa

6.14.3 Frost Protection

Based on MTO's *RSS Design Guidelines* (2008), it is understood that the minimum soil cover to the underside of the levelling pad supporting the footings for the RSS facing panels should be at least 0.8 m or 40 per cent of the frost depth (i.e., 1.5 m at this site), whichever is greater; and the minimum soil cover to the top of the levelling pad should be at least 0.5 m. It is further understood that the levelling pad at the last step of the RSS wall should be provided with 1 m of soil cover or 50 per cent of the frost depth, whichever is greater.

However, if annual wall movements resulting from freeze thaw cycles must be minimized, consideration should be given to providing the footings with a minimum 1.5 m of soil cover for frost protection as per OPSD 3090.101 *(Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

6.14.4 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall mass and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The coefficient of friction (tan φ'), between the compacted granular fill of the RSS wall and the new embankment fill may be taken as 0.60 (if the fill is expected to be comprised of earth fill or SSM) and 0.70 (if the embankment fill is expected to be comprised of granular fill).

6.14.5 Global Stability

As discussed in Section 6.12.1.1, a target minimum Factor of Safety of 1.5 is considered appropriate for design of the RSS walls for global stability. The results of the stability analyses, which are summarized in greater detail in

Section 6.12.3, indicate that a Factor of Safety for the RSS walls equal to or greater than 1.5 is achieved (see Figures 3B, 3C, 3D, 4B, 4C and 4D). However, a Factor of Safety of 1.5 will only be achieved if the length of the reinforcing strips are as follows:

- RSS walls in front and at the sides of the abutment stem walls: 1.4 times the wall height; and,
- Two-step RSS walls at the end of the cast-in-place concrete wing walls: 1.2 times the wall height.

The strip lengths must be specified on the Contract Drawings. The internal stability of a reinforced earth structure is to be designed and assessed by the proprietary product designer/manufacturer to ensure that the internal stability of the walls is acceptable.

6.14.6 Settlement

The estimated factored settlement along the RSS walls is expected to be up to about 25 mm assuming the walls are constructed following completion of preloading of the approach embankments for a period of two months. The estimated magnitude of settlement is also based on the assumption that the RSS walls will be founded within the bridge approach embankments comprised of new well compacted earth fill, SSM or granular fill as outlined in Section 6.16.4. However, if the RSS walls have to be constructed prior to employing the preloading period, consideration should be given to constructing a more flexible, two-stage construction RSS wall which is designed to tolerate higher post-construction settlements.

6.14.7 Performance and Appearance

Given that the RSS walls at the bridge abutments will be located next to a 400 series highway (i.e., Highway 400) and considering that the bridge will depict the coat of arms of Ontario and is classified as a heritage bridge, a high site performance rating and a high appearance rating is to be maintained in accordance with the MTO RSS Design Guidelines (2008).

6.15 Analytical Testing of Construction Materials

The results of analytical tests carried out on three samples of the silt and sand to silty sand deposit are presented in Section 4.2.11 and on the Certificate of Analysis in Appendix D. The analytical test results were compared to CSA A23.1 Table 3 (Additional requirements for concrete subjected to sulphate attack) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured on the soil samples range between less than 0.002 per cent and 0.022 per cent, which is below the moderate degree of exposure (i.e., below the class S3 exposure limits). Therefore, based on the three soil samples tested (only one tested soil sample was recovered from an average depth of about 1.8 m below ground surface and the other two soil samples were recovered from depths of about 20.1 m and 33.8 m below ground surface), when the designer is selecting the exposure class for the concrete structure, the effects of sulphates from within the non-cohesive deposit in contact with the spread footing or pile cap and any portion of the proposed structure constructed below the ground surface may not need to be considered. However, given that the proposed structure will be exposed to de-icing salt/chemicals, consideration should be given by the designer to designing the concrete structure for a "C" type exposure class as defined by CSA A23.1 Table 1.

The analytical test results of the soil samples were also compared to Table 7.1 (Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)), as presented in the Federal Highway Administration/National Highway Institute Publication No. FHWANHI14007 (Federal Highway Administration, 2015), to assess the relative level of corrosion potential on buried steel in contact with soil. The resistivity values measured on the soil samples from Boreholes 89UP-06, 89UP-02 and 89UP-05 are 2,600 ohm-cm, 1,200 ohm-cm and 330 ohm-cm, respectively. These results indicate a “moderately corrosive” to “very corrosive” potential. The very low resistivity (i.e., 330 ohm-cm), indicating a “very corrosive” potential, was measured on a surficial soil sample recovered from a borehole advanced at the location of the proposed centre pier.

It is also noted that the measured pH levels range between about 7.5 and 8.1, suggesting the presence of alkaline soils.

Ultimately, it is the designer’s decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) are satisfied.

6.16 Construction Considerations

The following subsections identify construction considerations that may impact the design and construction of the proposed Highway 89 underpass and its associated approach embankments.

6.16.1 Open-Cut Excavations

The topsoil, existing fill and any organic/deleterious materials encountered within the footprint of the proposed foundation elements and approach embankments should be subexcavated and replaced with earth fill, SSM or granular fill. In addition, construction of new pile caps at the abutments and centre pier will necessitate excavations to a depth of up to about 1.5 m below the existing ground surface.

All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

The soils to be excavated can be classified according to OHSA as follows (assuming the groundwater level is below the foundation subgrade level):

- Existing fill – Type 3; and,
- Generally compact silt to sandy silt to silt and sand to silty sand – Type 3.

Temporary excavations (i.e., those open for a relatively short-time period) should be made with side slopes no steeper than 1H:1V based on the soil profile. However, if water inflow is observed, flatter slopes and dewatering measures may need to be implemented. Temporary excavations should be observed and reviewed during construction to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, a geotechnical engineer should review the excavation plan considering the conditions at that time.

6.16.2 Instrumentation and Monitoring

As discussed in Section 6.8, preloading of the approach embankments for a period of two months prior to driving of the steel H-piles was presented as a mitigation measure against the application drag loads acting on the piles. For this mitigation option it is recommended that settlement monitoring and measurement of the dissipation of pore water pressures in the upper varved cohesive deposit be carried out as the approach embankments are constructed. The monitoring should be carried out for a period of time such that 80 per cent of the consolidation settlement of the upper varved clay deposit is complete and that only 2 mm to 3 mm is remaining based on the predicted consolidation settlement.

A monitoring program has been developed, consisting of the following:

- Settlement plates installed at the base of the approach embankment;
- Vibrating Wire Piezometers (VWP) installed at various depths within the upper varved cohesive deposit; and,
- Vibrating Wire Inline Extensometers (VWIX) installed at various depths within the granular deposit.

Instrumentation and monitoring plans (see Drawings E-1 and E-2, in Appendix E) and an NSSP for settlement monitoring are included in Appendix E, for inclusion in the Contract Documents.

6.16.3 Preloading

As discussed in Sections 6.4 and 6.8, in order to mitigate against drag loads being imposed on the steel H-piles at the east and west abutments for the underpass structure, a mitigation option is to construct the full embankment height with side slopes inclined at two horizontal to one vertical (2H:1V) and preload for a minimum period of about two months (i.e., 60 days). It is estimated that about 75 mm of immediate settlement of the upper granular deposit and between about 9 mm and 13 mm of consolidation settlement of the upper varved cohesive deposit will occur, under the approach embankment loading. The full height embankment that is in place for the preload period should extend a minimum distance of 20 m from the abutment. Prior to constructing the preload embankments, instrumentation as detailed is Section 6.16.2 must be installed. Following the preload period the embankment will need to be partially removed so that the piles can be driven and the RSS walls can be constructed.

An Notice to Contractor has been developed for inclusion in the Contract Documents to address the timing requirements associated with the preloading of the embankment locations and is presented in Appendix E.

6.16.4 Embankment Construction

Placement of Select Subgrade Material (SSM), earth fill meeting the requirements of OPSS.PROV 212 (*Borrow*), or granular fill (satisfying OPSS.PROV 1010 (*Aggregates*) Granular ‘B’ Type I or Type II requirements) above the water table for construction of new embankments (including backfilling operations) should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*). The SSM, earth fill or granular fill should be compacted in accordance with OPSS.PROV 501 (*Compacting*). Inspection and field testing should be carried out by a qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are being achieved. Side slopes for the SSM, earth fill or granular fill roadway embankment should be no steeper than 2H:1V. The embankment side slopes should also include a minimum 2 m wide bench at mid-height for all fill heights greater than 8 m as suggested in OPSD 202.010 (*Slope Flattening*). In addition, benching of the existing Highway 89 side slopes should be carried out to “key in” the new fill materials for the realigned highways in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

6.16.5 Erosion Protection

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments. In the short-term, if placement of cover material cannot be carried out soon after the construction of the embankments, erosion control blankets should be installed to minimize erosion of the embankment slopes. The erosion protection should be in accordance with OPSS.PROV 804 (*Seed and Cover*).

6.16.6 Temporary Protection Systems

Temporary protection systems will be required to facilitate the construction of the centre pier in the highway median, abutment construction relative to the north side slope of the existing embankment, as well as the removal of the existing Highway 89 underpass. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation. It is understood that due to space constraints in the median of Highway 400 that the pile cap will be constructed tight to the protection system and therefore the protection system will be left in place. Consideration could be given to removal of the protection system above the pile cap upon completion of construction or each stage of construction (as required) to mitigate potential impediments to future rehabilitation/reconstruction work at the underpass sites, or to the road structure above. An NSSP is included in Appendix E which addresses the cut-off of the protection system.

The Contractor should also be alerted to the groundwater level at site relative the underside of the pile cap at the pier and that dewatering will be required in order to maintain base stability. A Notice to Contractor is provided in Appendix E for inclusion in the Contract Documents.

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

6.16.7 Control of Groundwater and Surface Water

The highest groundwater level measured in the standpipe piezometer installed at the location of Borehole 89UP-03 is at about Elevation 226.4 m (i.e., about 1 m below the existing ground surface). The groundwater level could be higher during periods of heavy/sustained precipitation or during the wet seasons. Foundation excavations for shallow foundations or pile caps that extend 1.5 m below the ground surface for frost protection are expected to be excavated below the groundwater level. Consequently, dewatering measures will likely be required to ensure that the foundation elements can be constructed in dry conditions. Considering that the subsoils consist of sands and silts, the groundwater level is required to be lowered to 1 m below the founding elevation of the strip footings to mitigate against base instability. It is recommended that a Notice to Contractor be included in the Contract Documents to address this requirement (refer to Appendix E). Pumping from within trenches/ditches with properly filtered sump pumps will likely be insufficient to control the groundwater inflow as a result of the permeable nature of the sandy/silty deposits encountered at the surface. The control of water from dewatering operations should be managed in accordance with FOUN0003 (*Dewatering Structure Excavations*) and SP 517F01 (*Dewatering System; Temporary Flow Passage System*).

At the location of the proposed centre pier (i.e., at the centre median of the existing Highway 400), it will be challenging to implement an effective groundwater control operation due to limited working area and logistical challenges associated with the management and disposal of water generated during the dewatering operation. As such, an alternative approach could be adopted, such as incorporating an approximately 0.5 m thick mass concrete “plug”, placed at a depth of 1.5 m below existing grade using tremie methods. The reinforced concrete footing/pile cap could then be constructed in dry conditions above the tremie “plug”. Any inflow into the excavation above the tremie “plug” can be managed using a properly sized and filtered sump pump.

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

6.16.8 Control of Ground and Groundwater during Drilled Shaft (Caisson) Construction

As discussed in Section 6.7.2, if drilled shaft foundations are adopted, permanent caisson liners with a balancing head of water will be required to support the overburden soils and equalize groundwater pressures during construction. In addition, placement of concrete by tremie methods would be required.

6.16.9 Obstructions

The presence of cobbles/boulders was inferred between depths of about 46.3 m and 46.9 m below ground surface (between approximately Elevations 182.9 m and 182.3 m) in Borehole 89UP-05 which was advanced at the location of the proposed centre pier. If driven piles extend into the hard/very dense till deposit, steel H-piles should be reinforced with driving shoes such as Titus Standard “H” Bearing Pile Point design as per OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*), for protection during driving. In addition, it is recommended that piles driven to Elevation 192.5 m be equipped with driving shoes/flange reinforcement or bearing points. If steel tube piles are selected, driving shoes should be utilized in accordance with OPSD 3001.100 Type II (*Foundation Piles – Steel Tube Pile Driving Shoe*). Pile installation and driving shoes should be in accordance with OPSS.PROV 903 (*Deep Foundations*).

If drilled shaft foundations are selected, the construction equipment should be capable of advancing the permanent liner through such obstructions.

It is recommended that a Notice to Contractor be included in the contract documents to address obstructions (refer to Appendix E).

6.16.10 Vibration Monitoring During Pile Installation or Caisson Construction

If support of the underpass on driven steel H-piles are adopted and if the temporary protection systems are installed using vibratory methods, significant vibrations are not anticipated, given the compact nature of the native soil deposits. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by vibration activities will reach this threshold level and, therefore, vibration monitoring for the existing overpass structure is not expected to be required during construction at this site.

However, there are commercial/industrial buildings in the vicinity of the site, and a lower PPV threshold of 25 mm/s is generally considered applicable for buildings. If deep foundations are adopted, it is recommended that monitoring of vibrations be conducted at the building locations during construction, to defend against potential damage claims. An NSSP has been provided in Appendix E for inclusion in the Contract Documents.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer, and reviewed by Ms. Sandra McGaghran, M.Eng., P.Eng., a geotechnical engineer and Associate with Golder. Ms. Lisa Coyne, P.Eng., a Principal and MTO Foundations Designated Contact for Golder, conducted an independent technical and quality control review of this report.

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https://golderassociates.sharepoint.com/sites/12201g/6 - deliverables/fnds/reports/hwy 400-89 underpass/3 - final/1668512-fidr - highway 89 underpass-2018nov21.docx

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ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and SplitBarrel- Sampling of Soils
ASTM D1587	Standard Practice for ThinWalled Tube Sampling of -FineGrained- Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Saturated FineGrained- Soils

Canadian Standards Association (CSA):

CAN/CSA A23.1-14	Concrete Materials and Methods of Concrete Construction

Commercial Software:

LPILE Plus (Version 5.0) by Ensoft Inc.

Slide (Version 6.0) by Rocscience Inc.

Settle^{3D} (Version 4.0) by Rocscience Inc.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213	Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS):

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 212	Construction Specification for Earth Borrow

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD):

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile- Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Special Provisions:

SP 517F01	Dewatering System; Temporary Flow Passage System
SP 903S06	Deep Foundations
FOUN0003	Dewatering Structure Excavations

Ontario Regulations:

R.R.O 1990, Regulation 903	Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

TABLES

TABLE 1 – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES – HIGHWAY 89 UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread/strip footings founded on existing silt and sand fill or native deposit of sandy silt to silt and sand to silty sand	• Feasible, but very low geotechnical resistances (not recommended from a geotechnical / foundations perspective)	• Conventional construction techniques.	• Does not allow for integral abutment construction. • Requirement for excavations up to about 1.5 m for frost protection and will necessitate protection systems at the centre pier and possibly at the abutments. • Requirement for dewatering measures. • Low geotechnical resistances.	• Additional cost associated with requirement for temporary protection systems and dewatering measures.	• Low geotechnical resistances. • Higher total and differential settlement compared to foundation elements founded on deep foundations. • Footings must be constructed after the preferred settlement mitigation measure has been implemented to avoid post-construction settlements.
Spread/strip footings "perched" within the bridge approach embankments on a Granular 'A' pad ***(Abutments Only)***	• Feasible, but low geotechnical resistances (not recommended from a geotechnical / foundations perspective)	• Conventional construction techniques. • Foundation excavations not required at the abutments. • Higher geotechnical resistances compared to footings founded on native soil deposits.	• Does not allow for integral abutment construction. • May require a longer bridge deck to accommodate larger abutment foreslopes. • Low serviceability geotechnical resistances.	• Lower relative cost than deep foundations. • Additional cost associated with construction of a Granular 'A' pad; however, this may be offset by lesser excavation costs.	• Low geotechnical resistances. • Higher total and differential settlement compared to foundation elements founded on deep foundations. • Footings must be constructed after the preferred settlement mitigation measure has been implemented to avoid post-construction settlements.
Driven Steel H-piles (HP 310x110) or Tube Piles (324 mm diameter)	• Feasible and preferred from a geotechnical/ foundations perspective	• Conventional construction methods for H-pile / tube pile foundations. • Allows for integral abutment design, but tube piles mat not be feasible for integral abutment design. • Pile cap may be "perched" within the bridge	• Very long piles (about 35 m to 51 m) – requirement for two to three splices. • Requires driving shoes due to potential presence of cobbles/boulders within the lower till deposit.	• Higher relative cost than spread/strip footings. • Lower relative cost than drilled shafts (caissons) and potentially some of the intermediate foundation alternatives. • Additional cost for the	• Risk of H-piles hanging up above the design pile tip elevation, or of damage to the pile, due to cobbles and boulders within the till deposit. • Piles must be driven after the preferred settlement

TABLE 1 – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES – HIGHWAY 89 UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
		approaches to reduce the amount of excavation.	• Pile load test recommended to confirm capacity for design.	static pile load test.	mitigation measure has been implemented to reduce/eliminate downdrag and drag loads.
1.2 m Diameter Drilled Shafts (Caissons)	• Feasible, but less economical than driven steel H-piles	• Conventional construction methods for drilled shaft foundations. • Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. • Pile caps may be “perched” within the bridge approaches or affixed to underside of the bridge deck to reduce the amount of excavation and potentially eliminate requirement for protection systems. • Generally offers a narrower footprint for construction in constrained working areas, as compared with spread/strip footings or driven/battered piles.	• Precludes use of integral abutments. • Permanent liners will be required, plus special measures such as use of bentonite slurry to counterbalance groundwater pressures and minimize disturbance at the founding level of the drilled shaft. • Generation of soil cuttings during drilled shaft advancement.	• Higher relative cost than spread/strip footings, driven steel H-piles and intermediate foundation alternatives.	• High risk of caisson refusal on boulders. • May not be possible to inspect the base of the drilled shaft due to length of foundation element and potential need for bentonite slurry inside the liners. • Drilled shafts must be installed after the preferred settlement mitigation measure has been implemented to reduce/eliminate downdrag and drag loads.
Helical Piles (founded within the upper granular deposit)	• Feasible, but low factored serviceability geotechnical resistances due to settlement of pile groups and low lateral resistances (not recommended from a geotechnical / foundations perspective)	• Requires smaller equipment as compared to equipment required to install other deep foundation units. • Relatively quick installation operation. • No generation of excess soil cuttings during installation.	• Generally low lateral resistance due to slender shafts – subject to buckling. • Difficult to install in soils with SPT ‘N’-values greater than 50 blows per 0.3 m of penetration. • Proprietary/detail helical pile design will be required.	• Higher relative cost than spread/strip footings. • Additional cost associated with proprietary helical pile design and verification of pile capacities.	• Fewer contractors have experience with helical piles installations on MTO projects as compared to other deep foundation installations such as, driven piles and drilled shafts. • Settlement and resistance of helical piles is largely installation dependent.

TABLE 1 – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES – HIGHWAY 89 UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
					• High risk of not satisfying lateral load design criteria.
Micropiles (founded within the upper granular deposit)	• Feasible, but low factored serviceability geotechnical resistances due to settlement of pile groups; micropiles would have to extend below the upper cohesive deposit, but would be more expensive than driven steel piles	• Requires smaller drilling equipment as compared to equipment required to install other deep foundation units.	• Allows only for semi-integral abutment design. • Generally low lateral resistances. • Detail micropile design will be required. • Pile load test recommended to confirm capacity for design.	• Higher relative cost than spread/strip footings and driven pile foundation options. • Additional cost associated with detail micropile design. • Additional cost for specialized drilling equipment. • Additional cost for the micropile load test.	• Fewer contractors have experience with micropile installations on MTO projects as compared to other deep foundation installations such as, driven piles and drilled shafts.
Continuous Flight Auger (CFA) Piles (founded within the upper granular deposit)	• Feasible, but low factored serviceability geotechnical resistances due to settlement of pile groups (not recommended from a geotechnical / foundations perspective)	• Relatively quick installation without the need for liners/sleeves. • Consensus	• Typical pile length limited to about 25 m. • Typical pile diameter limited to about 0.75 m. • Specialized equipment required to increase diameter and length • Generation of excess soil cuttings during the extraction phase. • Loose sandy/silty soils below the groundwater level can pose difficulties. • Pile load test recommended to confirm capacity for design.	• Generally higher mobilization costs compared to other deep foundation units, but overall cost may be competitive (project specific). • Additional cost for the pile load test.	• CFA piles have not been utilized on previous MTO projects. • Settlement of pile groups likely not tolerable.

TABLE 1 – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES – HIGHWAY 89 UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Drilled Displacement (DD) Piles (founded within the upper granular deposit)	• Feasible, but low factored serviceability geotechnical resistances due to settlement of pile groups (not recommended from a geotechnical / foundations perspective)	• Perform better in loose sandy/silty soils compared to CFA piles – loose soils are densified during installation. • Generation of very small amount of excess soil cuttings.	• Limited depth of installation in dense/hard soils. • Large equipment required, especially for long piles, due to high downward/retraction force and torque requirements. • Pile load test recommended to confirm capacity for design.	• Similar cost compared to CFA piles. • Additional cost for the pile load test.	• DD piles have not been utilized on previous MTO projects. • Settlement pile groups likely not tolerable.

TABLE 2 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS – EAST AND WEST APPROACH EMBANKMENTS

Foundation Investigation Area (Relevant Boreholes)	Stratigraphic Unit	Top Elevation (m)	Thickness (m)	γ (kN/m^3)	ϕ' (°)	c' (kPa)	s_u (kPa)	σ_p' (kPa)	e_o	C_c	C_r	m_v (kPa^{-1})	E' (MPa)	c_v (cm^2/s)
West Abutment and Approach Embankment (Boreholes 89UP-01 to 89UP-03 and B1-2)	Silty Sand to Gravelly Sand (Fill)	228.4 – 227.2 ~ 235.2 (Hwy 89)	0.5 – 2.3 ~ 10.0 (Hwy 89)	19	32	0	--	--	--	--	--	--	10 – 25	--
	Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand (Upper Granular Deposit)	226.7 – 225.2	18.3 – 20.2	19	32	0	--	--	--	--	--	--	10 – 45	--
	Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand (Upper Granular Deposit)	~214.0	5.6 – 9.0	19	36	0	--	--	--	--	--	--	45 – 55	--
	Varved Clayey Silt to Silty Clay with Silt and Clay Laminae (Upper Cohesive Deposit)	206.9 – 206.5	11.0 – 12.4	18.5	31	0	75 – 90	340 – 410	0.75 – 1.15	0.40 – 0.67	0.02 – 0.046	--	--	4.4 x 10^{-3} (Overconsolidated Range)
	Silt to Sandy Silt to Silty Sand (Lower Granular Deposit)	195.5 – 194.5	5.1 – 6.5	19	36	0	--	--	--	--	--	--	75	--
	Clayey Silt (Lower Cohesive Deposit)	189.4 – 189.0	~6.1	18.5	34	0	150	680	--	--	--	1.3 x 10^{-5}	--	--
	Clayey Silt (Till) / Silt and Sand (Till)	~182.9	~4.7, but not fully penetrated	21	38	0	--	--	--	--	--	--	175	--
East Abutment and Approach Embankment (Boreholes 89UP-06 to 89UP-08, HF-02 and B1-1)	Silt and Sand to Silty Sand (Fill)	228.9 – 226.7 ~235.2 (Hwy 89)	0.2 – 2.7 ~8.9 (Hwy 89)	19	32	0	--	--	--	--	--	--	10 – 30	--
	Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand (Upper Granular Deposit)	227.7 – 224.6	18.6 – 21.3	19	32	0	--	--	--	--	--	--	10 – 45	--
	Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand (Upper Granular Deposit)	~214.0	5.6 – 10.7	19	36	0	--	--	--	--	--	--	45 – 55	
	Varved Clayey Silt to Silty Clay with Silt and Clay Laminae (Upper Cohesive Deposit)	209.1 – 205.3	9.6 – 12.2	18.5	31	0	75 – 90	340 – 410	0.75 – 1.15	0.40 – 0.67	0.02 – 0.046	--	--	4.4 x 10^{-3} (Overconsolidated Range) 1.0 x 10^{-3} (Normally Consolidated Range)
	Silt to Silt and Sand to Silty Sand (Lower Granular Deposit)	195.7 – 194.0	3.0 – 5.3	19	36	0	--	--	--	--	--	--	75	--
	Sandy Clayey Silt to Clayey Silt (Lower Cohesive Deposit)	190.9	6.1 ~7.9, but not fully penetrated	18.5	34	0	100 – 150	450 – 680	--	--	--	1.3 x 10^{-5}	--	--
	Clayey Silt with Sand (Till) / Silt and Sand (Till)	~184.2	~7.6, but not fully penetrated	21	38	0	--	--	--	--	--	--	175	--

Note:

1. Complete plots of the parameters (i.e., undrained shear strength (s_u), preconsolidation stress (σ_p'), void ratio (e_o), compression index (C_c) and recompression index (C_r)) versus elevation for the upper cohesive deposit are presented on Figure 2.

TABLE 3 – COMPARISON OF DRIVEN STEEL H-PILES OPTIONS FOR DRAG LOAD MITIGATION – HIGHWAY 89 UNDERPASS ABUTMENTS

Foundation Option	Design for Drag Load?	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Driven Steel H-piles (HP 310 x 110) **Pile Length = 40 m** ***Drag Load Not Accommodated in Design*** **2-month preload** (full height) or **1-month surcharge** (full height + 2m) **Monitoring Required** Drive piles after settlement of the clay deposit is >95% complete	• **NO.** • Pile does not have structural capacity for dead load + drag load. • Minimum 2-month preload period or 1-month surcharge period required for sufficient settlement of the clay to avoid drag loads on piles and to reduce post-construction settlement of pile group.	Foundations: • Standard integral abutment design. • Standard pile size. • Shorter (40 m long) pile(s) terminating in very dense silt and sand or very stiff to hard sandy clayey silt deposit. Penetration into “100-blow” till deposit containing cobbles and boulders not required, therefore low risk of damaging piles.	Foundations: • Preload or surcharge period required to ensure that settlement of the clay is complete before pile installation in order to avoid drag loads on piles and post-construction pile group settlement exceeding the SLS requirement.	Foundations: • Additional costs for instrumentation and monitoring program. • Additional costs for double handling of fill. • Higher costs for adding and removing surcharge. ***Instrumentation Cost^{6} = approx. $60K***	Foundations: • Risk to schedule if settlement is greater and/or takes longer than expected (see Note 5). • If settlement of the clay takes longer than the anticipated preload / surcharge period, this option has the lowest reserve in the structural pile capacity to accommodate drag loads.
		Structural: • Piles can be sourced locally and don’t require a lead time for ordering.	Structural: • Pile has a factored reserve capacity for drag load of 1,560 kN << 1.25 (1,500 kN^{1A}) = 1,875 kN and has the lowest structural capacity of options considered.	Structural: • Lowest cost option.	Structural: • Pile does not have reserve for combined dead load + drag load, if settlements are remaining after preload or surcharge period.
Driven Steel H-piles (HP 310 x 132) **Pile Length = 40 m** ***Drag Load is Accommodated in Design*** **2-month preload** (full height) or **1-month surcharge** (full height + 2m) **Monitoring Required** Drive piles after settlement of the clay deposit is >95% complete	• **YES.** • Pile does have structural capacity for dead load + drag load, but minimum 2-month preload period or 1-month surcharge period required for sufficient settlement of the clay and to reduce post-construction settlement of pile group.	Foundations: • Heavier pile section, therefore less chance of damaging pile during driving.	Foundations: • Preload or surcharge period required to ensure that settlement of the clay is complete before pile installation in order to avoid post-construction pile group settlement exceeding the SLS requirement.	Foundations: • Additional costs for instrumentation and monitoring program. • Additional costs for double handling of fill. • Higher costs for adding and removing surcharge. ***Instrumentation Cost^{6} = approx. $60K***	Foundations: • Risk to schedule if settlement is greater and/or takes longer than expected (see Note 5); however higher reserve capacity in pile provides flexibility to accommodate the drag loads, if necessary.
		Structural: • Piles can be sourced locally and don’t require a lead time for ordering. • Piles have larger structural reserve compared to lighter pile. • Pile has a factored reserve capacity for drag load of 2,000 kN > 1.25 (1,500 kN^{1A}) = 1,875 kN	Structural: • None noted.	Structural: • Higher costs of piles compared to HP 310x110.	Structural: • None noted.
Driven Steel H-piles (HP 310 x 174) **Pile Length = 50 m** ***Drag Load is Accommodated in Design*** Preload or surcharging period not required, but piles must be driven into “100-blow” till (pile length = 50 m). Monitoring recommended, but not strictly required.	• **YES.** • Pile does have structural capacity for dead load + drag load	Foundations: • No preload or surcharge period required • Monitoring not required; however, it is recommended to improve understanding of settlement and stresses in the piles to inform future design.	Foundations: • Longer (50 m) piles required to be driven into the till deposit to meet the SLS requirement for pile group settlement.	Foundations: • Cost savings realized since no double handling of fill for preload or surcharge. • Monitoring is not strictly required, but is recommended to improve understanding of settlement and stresses in the piles to inform future design. • Longer piles (i.e. 50 m) required to satisfy SLS requirement of 25 mm for the pile group settlement ***Instrumentation Cost^{7} = approx. $80K***	Foundations: • No schedule impacts and uncertainty of preload/surcharge period avoided. • Risk of damage to piles driven into till containing cobbles and boulders.
		Structural: • Piles have largest structural reserve compared to lighter piles. • Pile has a factored reserve capacity for drag load of 2,880 kN > 1.25(2,000 kN^{1B}) = 2,500 kN.	Structural: • Two to three months lead time to order piles. • Stiffer pile section not typical as per Ministry practice.	Structural: • Higher pile costs compared to lighter piles.	Structural: • Stiffer pile section may not be typical as per Ministry practise.
Driven Steel H-piles (HP 310 x 132) **Pile Length = 40 m** ***Drag Load is Accommodated in Design*** **Construct Embankment with Cellular Concrete** (to about 20 m back from the abutment). **1-month preload** period (6 m preload fill height)	• **YES** • Pile does have structural capacity for dead load + drag load, but minimum 1-month preload period required for sufficient settlement of the clay to avoid drag loads on piles and to reduce post-construction settlement of the pile group.	Foundations • Shorter preload period. • Lower preload height.	Foundations • Requires placement of cellular concrete behind abutment and RSS walls to ensure that settlement of the clay is complete before pile installation in order to avoid post-construction pile group settlement exceeding the SLS requirement. • Placement of cellular concrete requires cure time and therefore it needs to be placed in lifts, which may take longer compared to placement of conventional fill.	Foundations • High cost of cellular concrete. • Additional costs for instrumentation and monitoring program. ***Instrumentation Cost^{6} = approx. $60K*** ***Cellular Concrete Cost = approx. $3.4 M***	Foundations • Risk to schedule if settlement is greater and/or takes longer than expected (see Note 5); however, higher reserve capacity in pile provides flexibility to accommodate drag loads, if necessary. • Risk to schedule with additional construction method of cellular concrete placement (ie curing time and potential delays).

TABLE 3 – COMPARISON OF DRIVEN STEEL H-PILES OPTIONS FOR DRAG LOAD MITIGATION – HIGHWAY 89 UNDERPASS ABUTMENTS

Foundation Option	Design for Drag Load?	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Monitoring Required		Structural • Piles can be sourced locally and don't require a lead time for ordering. • Pile has a factored reserve capacity for drag load of 2,000 kN > 1.25 (1,500 kN[1A]) = 1,875 kN.	Structural: • None noted.	Structural • Higher costs of piles compared to HP 310x110.	Structural: None noted.
Driven High Grade Steel H-piles (HP 310 x 132, Grade 65) **Pile Length = 50 m** ***Drag Load <u>is</u> Accommodated in Design*** Preload or surcharging period not required, but piles must be driven into "100-blow" till (pile length = 50 m). Monitoring recommended, but not strictly required.	• **YES.** • Pile does have structural capacity for dead load + drag load.	Foundations: • No preload or surcharge period required. • Monitoring not required; however, it is recommended to improve understanding of settlement and stresses in the piles to inform future design.	Foundations: • Longer (50 m) piles required to be driven into the till deposit to meet the SLS requirement for pile group settlement.	Foundations: • Cost savings realized since no double handling of fill for preload or surcharge. • Monitoring is not strictly required, but is recommended to improve understanding of settlement and stresses in the piles to inform future design. ***Instrumentation Cost[7] = approx. $80K***	Foundations: • No schedule impacts and uncertainty of preload/surcharge period avoided. • Risk of damage to piles driven into till containing cobbles and boulders.
		Structural: • Piles have larger structural reserve compared to similar and lighter piles. • Pile has a factored reserve capacity for drag load of 2760 kN > 1.25(2,000[1B] kN) = 2,500 kN	Structural: • Two to three months lead time to order piles.	Structural: • Higher pile costs compared to similar and lighter piles. • Longer piles (i.e. 50 m) required to satisfy SLS requirement of pile group settlement.	Structural: • None noted.

Notes:

1A. For design, an unfactored drag load of 1,500 kN may be used for 40 m long piles (design pile tip Elevation 192.5 m).

1B. For design, an unfactored drag load of 2,000 kN may be used for 50 m long piles (design pile tip Elevation 181 m).

2. An option with integral abutment piles supported on a shallow raft foundation was discussed at the Foundations Workshop meeting on May 31, 2018. MTO noted that this design option has not been previously in Ontario and is reluctant to use an untested design on a 400-series highway structure. It was concluded that this design option would be better suited to a site with lower subsurface soil risks and a less significant structure risk. In addition, the current design is approaching 60% and re-design of the structure at this stage would impact the schedule.

3. Other options presented at the workshop on May 31, 2018 include pre-drilling and mixing the upper sand layer with a bentonite slurry prior to pile driving. However, this option may introduce design issues associated with the lateral resistance of the piles. Additional costs are incurred for this process and it may not be economically feasible due to the number of piles at the abutments.

4. An option of coating the piles with bitumen prior to driving was discussed, as the bitumen would reduce the friction between the sand/clay and the steel H-pile. However, for the size of this project, the costs associated with sourcing/set-up of a bitumen plant would likely make this option economically unfeasible.

5. Regarding the probability that the settlements will be achieved in the durations specified, we have not carried out probabilistic analyses considering the range and distribution of c_v values and their effect on the predicted rates of settlement; however, we provide the following comments on the risk associated with the different preload periods:

    - 1 month – medium risk that settlements will not be completed (estimate 50% probability of sufficient completion of settlement)
    - 2 months - low risk that settlements will not be completed (estimate 80% probability of sufficient completion of settlement)
    - 3 months – very low risk that settlements will not be completed (estimate 90% probability of sufficient completion of settlement)

6. Monitoring consists of 1 Vibrating Wire Inline Extensometer (VWIX), 3 Vibrating Wire Piezometers (VWP), 3 Settlement Plates (SP). Costs include instrumentation and installation and do not include the costs to excavate a trench to put the PVC pipe with the datalogger cables from the VWP and VWIX.

7. Monitoring consists of 3 VWPs, 3 SPs, 1 VWIX and 10 strain gauges on the pile. Costs include instrumentation and installation and does not include the costs to excavate a trench to put the PVC pipe with the datalogger cables from the VWPs and strain gauges.

DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

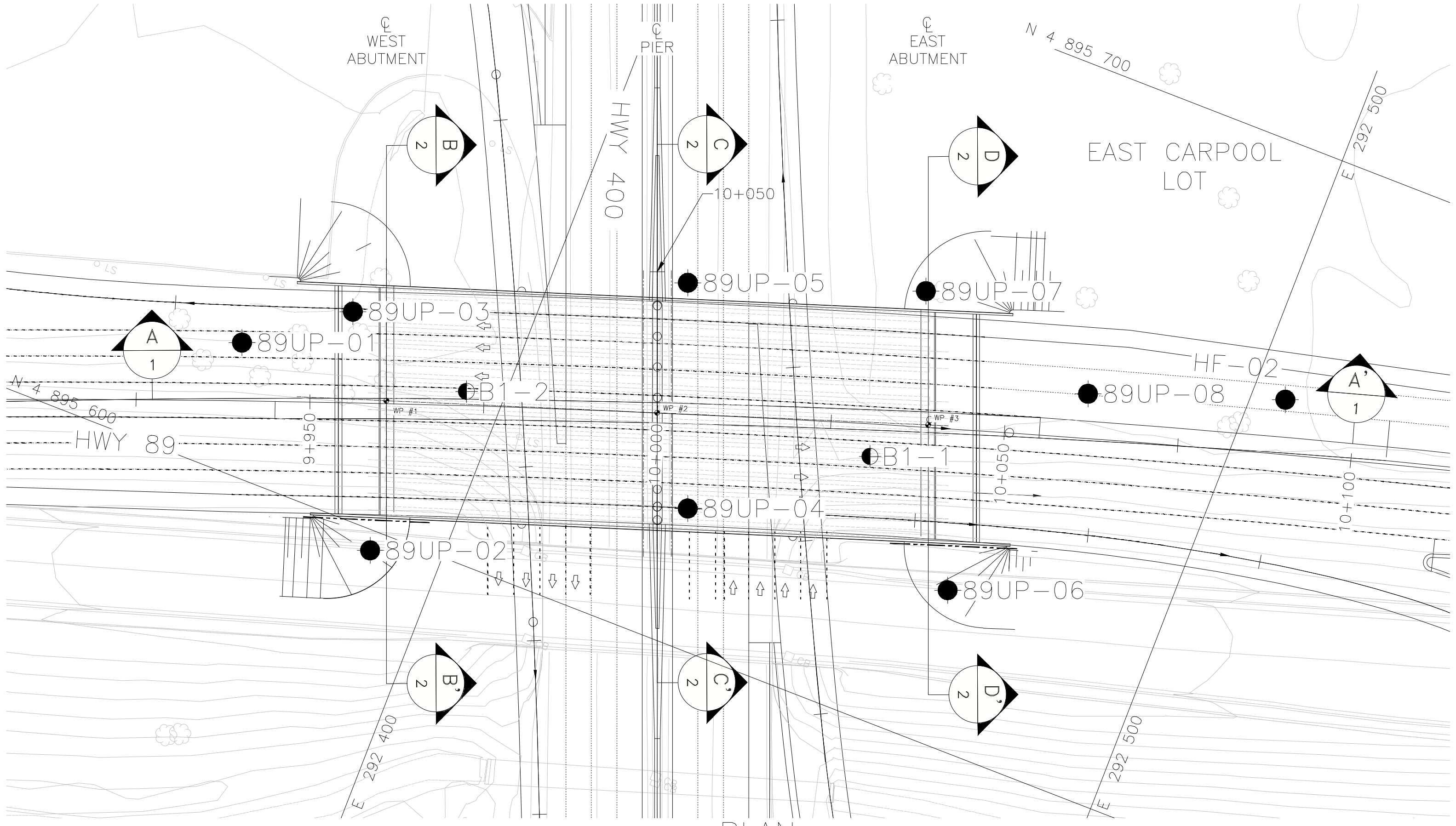
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.2438–13–00

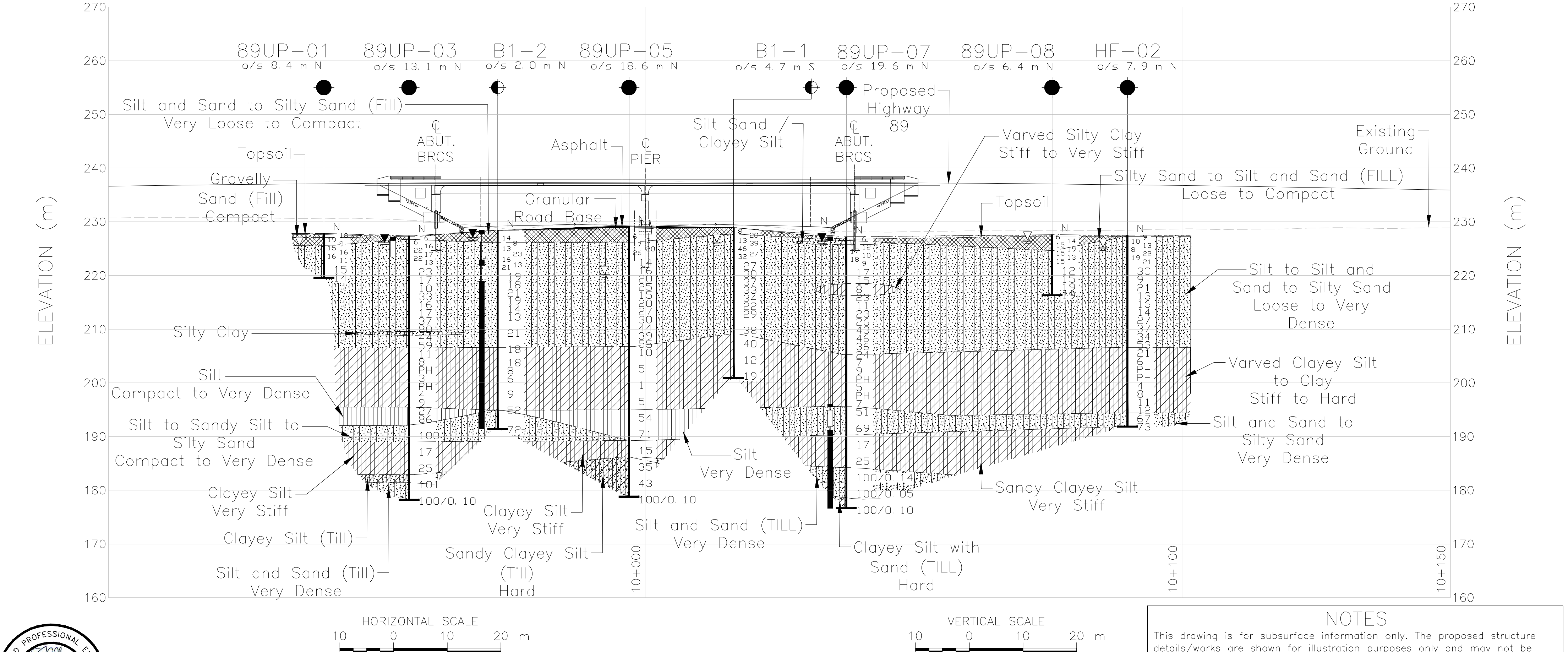
HIGHWAY 400/89 UNDERPASS REPLACEMENT STRUCTURE BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates



PLAN
SCALE
10 0 10 20 m



A–A
1
CENTRELINE PROFILE
HIGHWAY 89

KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
◐	Borehole – 2002 Investigation (Geocres 31D00–465)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on March. 15, 2001 or Sept. 19, 2017.
▽	WL upon completion of drilling or beginning of last day drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
89UP–01	227.8	4895618.4	292361.9
89UP–02	235.4	4895597.2	292389.9
89UP–03	227.4	4895628.3	292375.2
89UP–04	229.3	4895619.3	292430.3
89UP–05	229.2	4895649.6	292418.6
89UP–06	235.4	4895621.9	292469.4
89UP–07	227.2	4895660.9	292451.0
89UP–08	227.6	4895655.5	292478.1
HF–02	227.5	4895665.0	292504.8
B1–1	228.9	4895635.8	292452.1
B1–2	228.4	4895623.5	292394.6

REFERENCE

Base plans provided in digital format by Morrison Hershfield, received May 26, 2017.

PDR Allignment provided in digital format by Morrison Hershfield, drawing file "PDR_Allignment.dwg", received May 31, 2017.

Bridge GA provided by Morrison Hershfield, drawing file "1170121–01.dwg", received Sept. 19, 2017.

NOTES

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

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

NO.	DATE	BY	REVISION

Geocres No. 31D–702

HWY. 400/89		PROJECT NO. 1668512	DIST. CENTRAL
SUBM'D. DF	CHKD. DM	DATE: 4/27/2018	SITE: 30–256
DRAWN: SMD	CHKD. SMM	APPD. LCC/JMAC	DWG. 1

L.C. COYNE
Sept 11, 2018

S. McGaghran
Sept 11, 2018

PLOT DATE: September 12, 2018
FILENAME: S:\Clients\MTO\Hwy_400_and_Hwy_89_Interchange\99_PROJ\1668512_Detailed_Design\40_PROD\0003_Underpass\1668512–0003–BG–0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

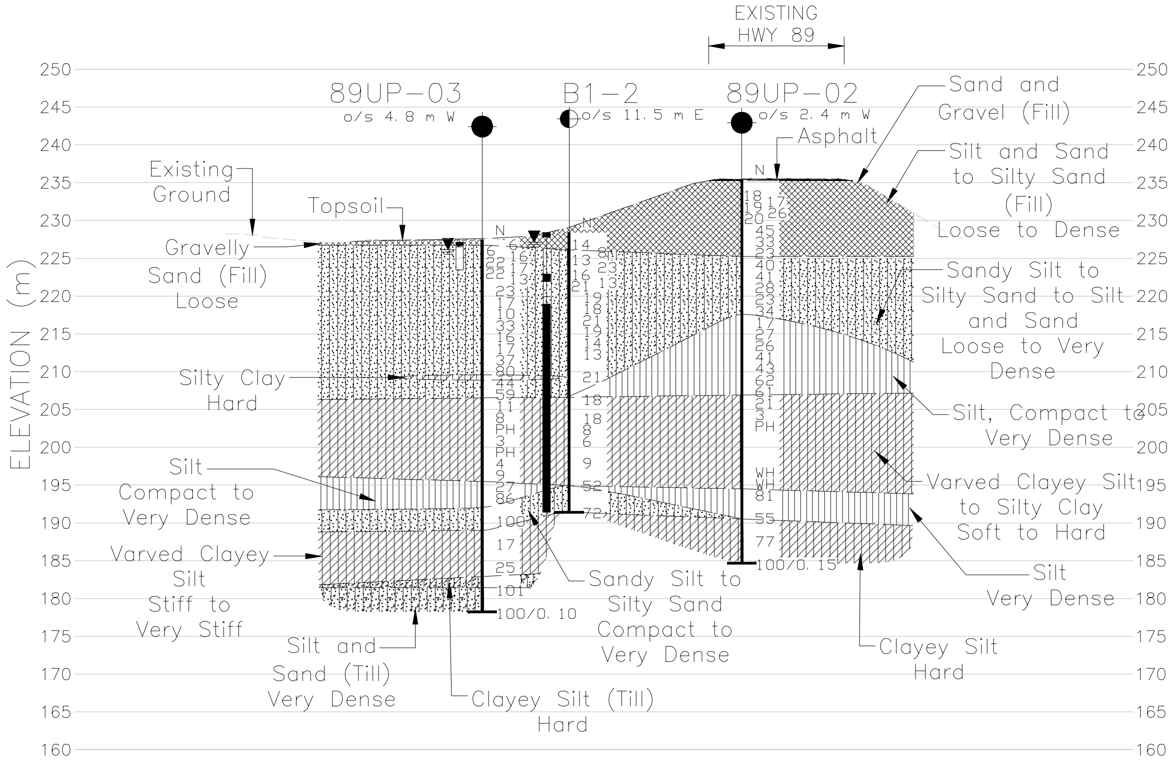
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.2438-13-00

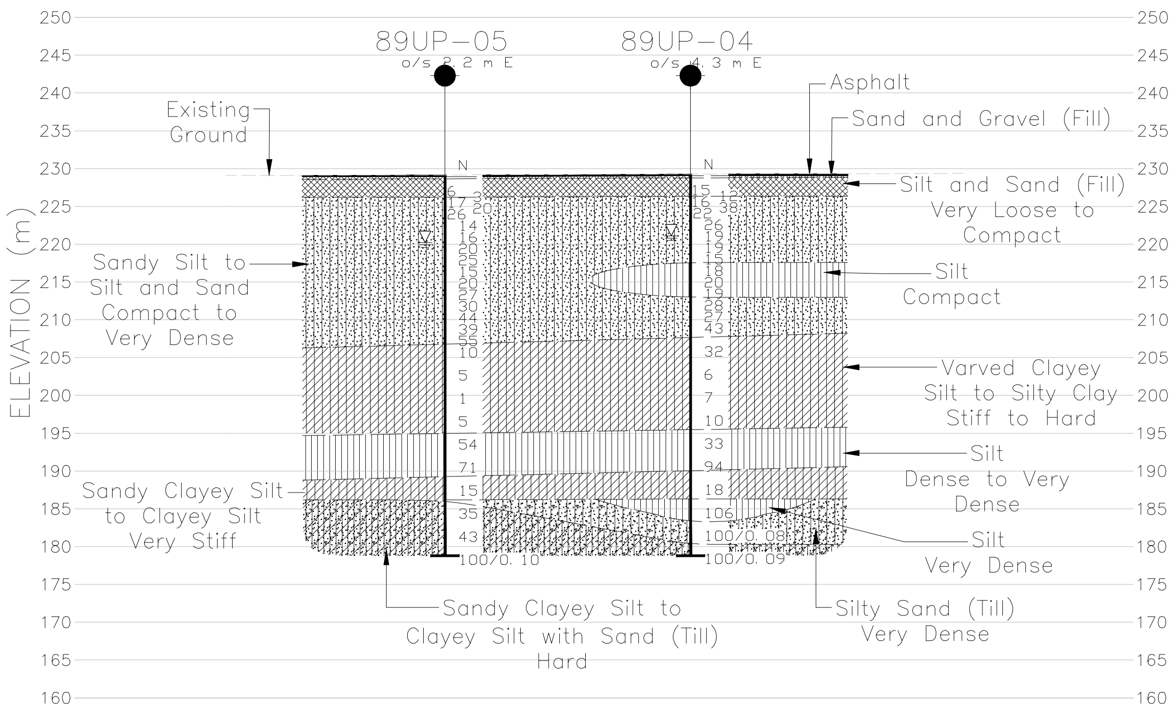
HIGHWAY 400/89 UNDERPASS REPLACEMENT STRUCTURE SOIL STRATA

SHEET

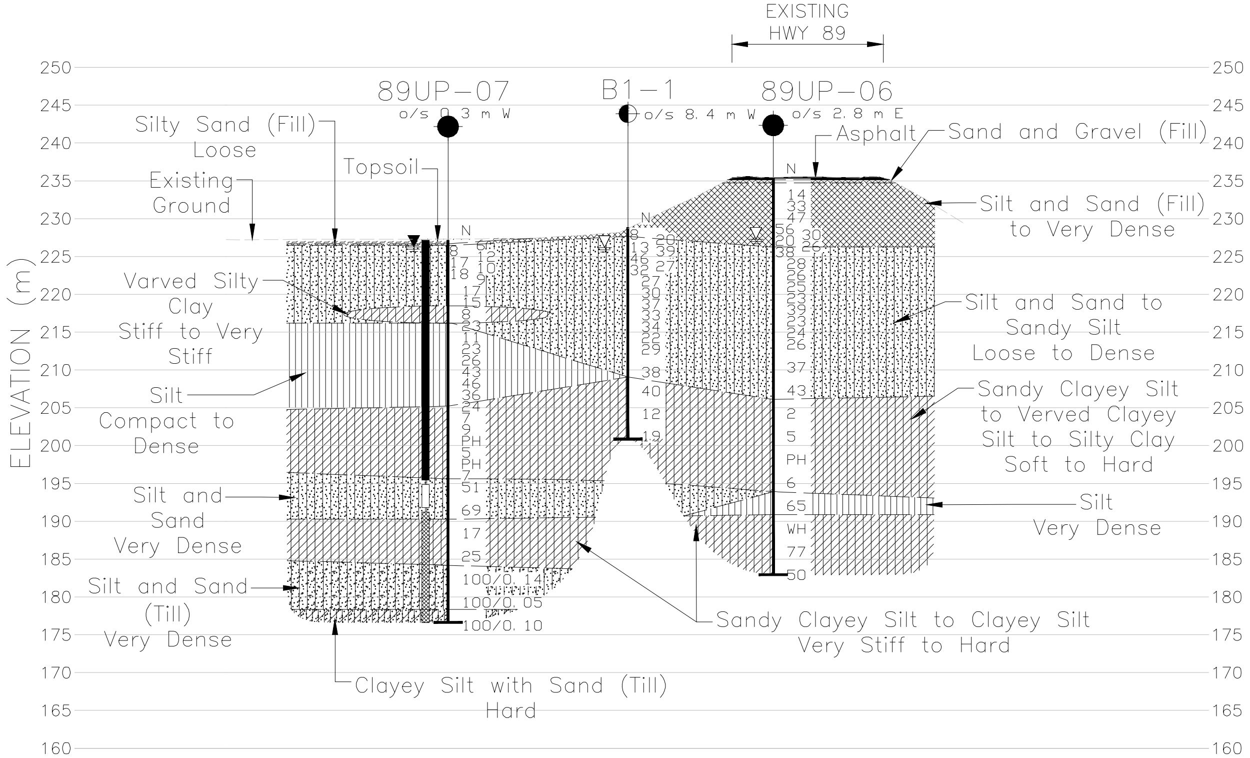
Golder Associates



SCALE 1:500 B-B' ② WEST ABUTMENT HIGHWAY 89



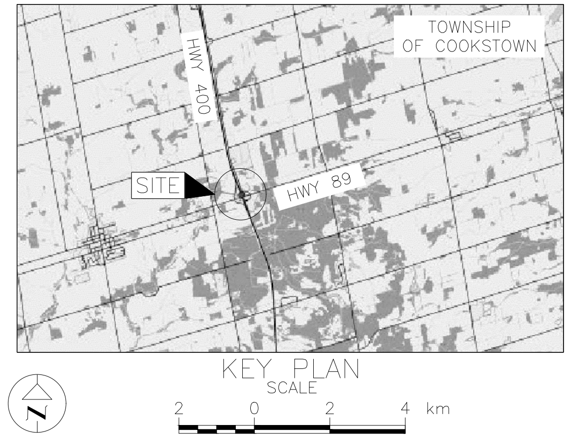
SCALE 1:500 C-C' ② CENTRE PIER HIGHWAY 89



SCALE 1:500 D-D' ② EAST ABUTMENT HIGHWAY 89

HORIZONTAL SCALE
10 0 10 20 m

VERTICAL SCALE
10 0 10 20 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
◐	Borehole – 2002 Investigation (Geocres 31D00-465)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on March. 15, 2001 or Sept. 19, 2017.
▽	WL upon completion of drilling or beginning of last day drilling

BOREHOLE CO-ORDINATES

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L.C. COYNE
Sept 11, 2018

S. McGaghran
Sept 11, 2018

NO.	DATE	BY	REVISION

Geocres No. 31D-702

HWY. 400	PROJECT NO. 1668512	DIST. CENTRAL
SUBM'D. DF	CHKD. DM	DATE: 4/27/2018	SITE: 30-256
DRAWN: SMD	CHKD. SMM	APPD. LCC/JMAC	DWG. 2

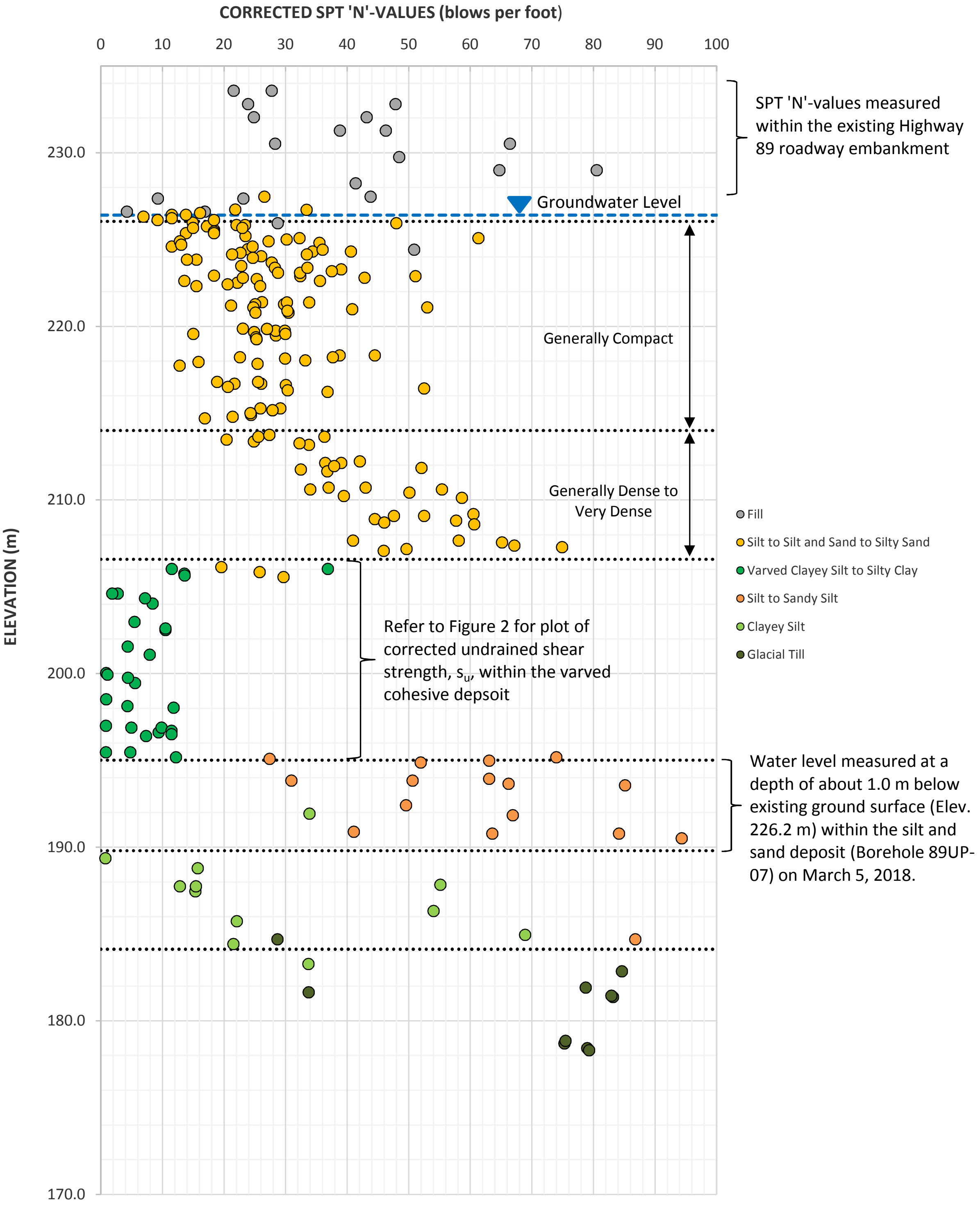
PLOT DATE: September 12, 2018
FILENAME: S:\Clients\MTO\Hwy_400_and_Hwy_89_Interchange\99_PROJ\1668512-0003_Underpass\40_PROD\0003_Underpass\1668512-0003-BG-0003.dwg

FIGURES

CORRECTED SPT 'N'-VALUES VERSUS ELEVATION

Highway 89 Underpass - East and West Approach Embankments

FIGURE 1



Date: November 2018

Project No: 1668512

Prepared By: TZ

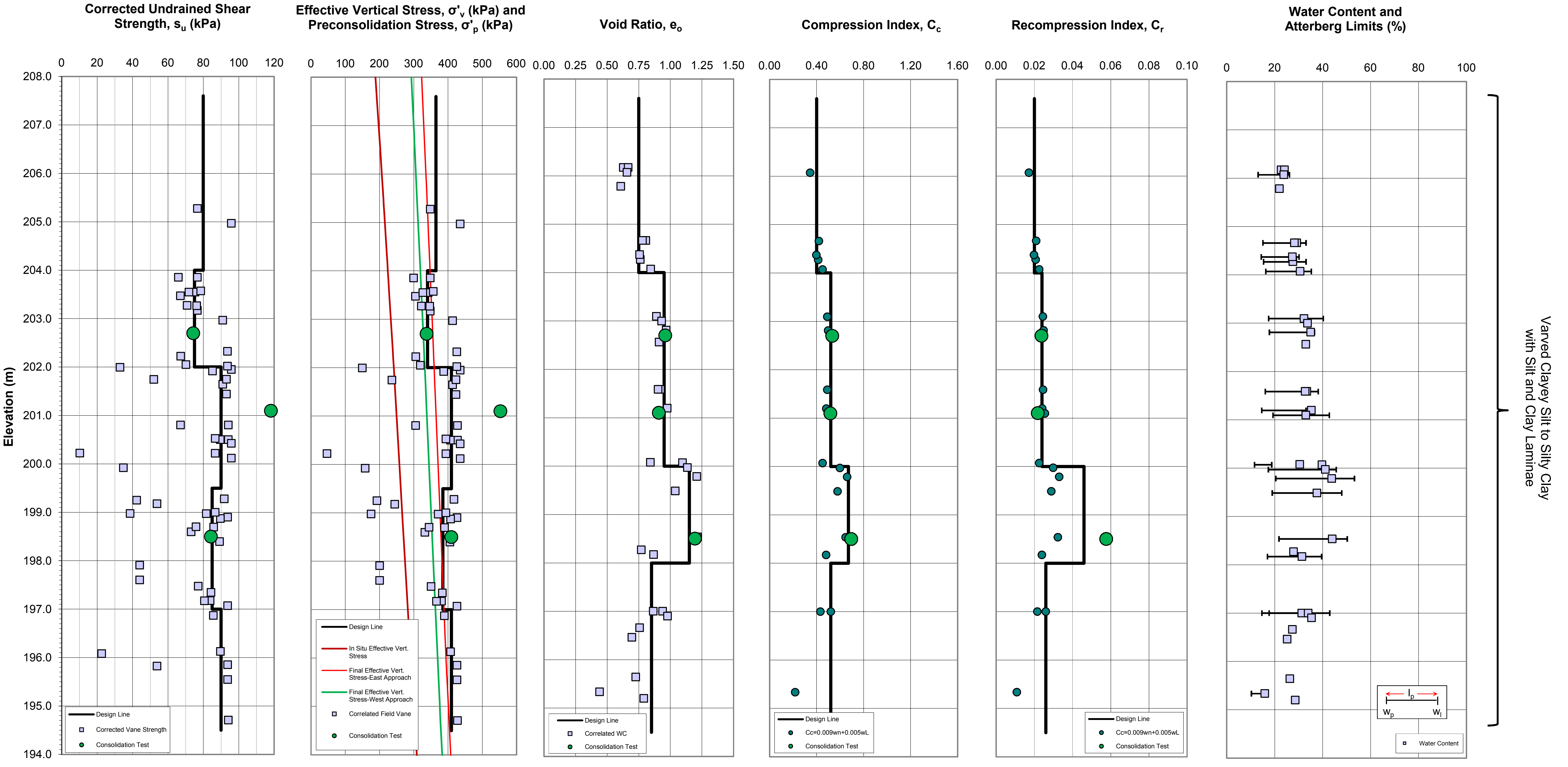
Checked By: SMM/LCC

Golder Associates Ltd.

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR VARVED COHESIVE DEPOSITS

Highway 89 Underpass - East and West Approach Embankments

FIGURE 2



Date: November 2018

Project No: 1668512

Prepared By: TZ

Checked By: SMM/LCC

Golder Associates

Golder Associates Ltd.

https://golderassociates.sharepoint.com/sites/12201g/6 - Deliverables/Fnds/Reports/Hwy 400-89 Underpass/3 - Final/Figures/[Figure 2-Engineering Parameters for Varved Cohesive Deposit.xlsx]Figure 2

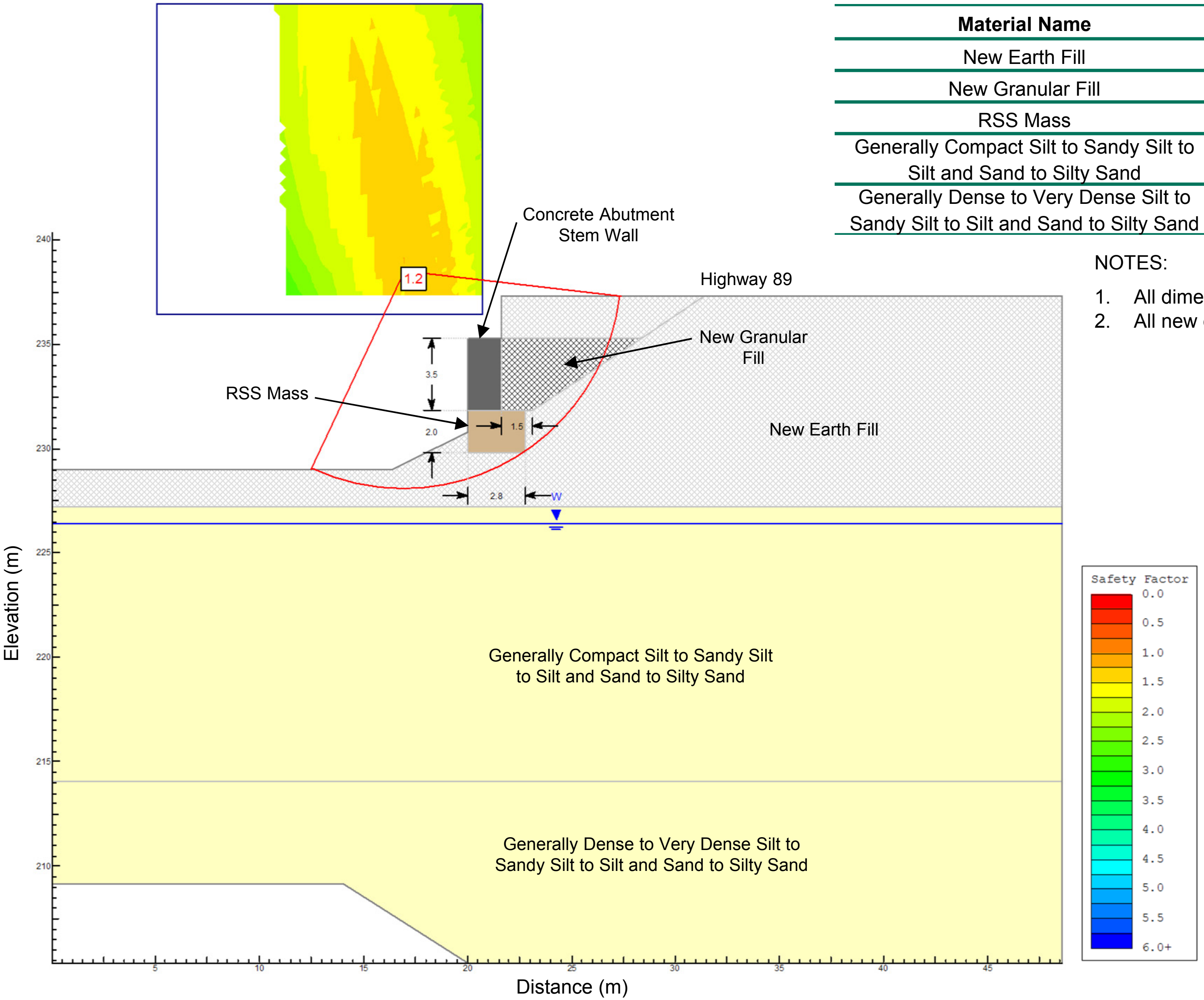
Highway 400/89 Underpass Replacement Structure
East Approach Embankment – Front Slope Stability
Final Configuration (Permanent Condition)

Figure 3A

Material Name	γ (kN/m^3)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

Golder Associates

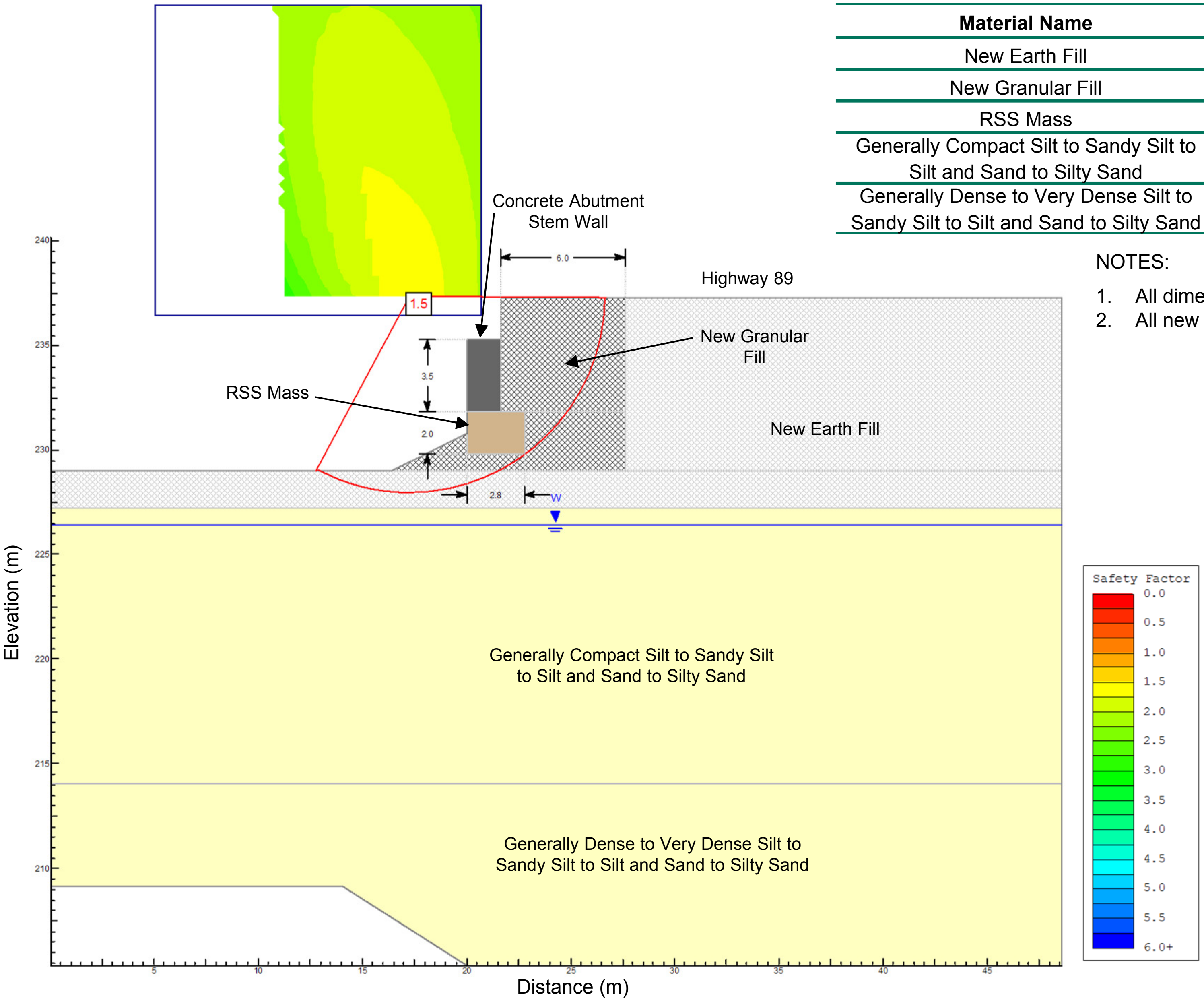
Highway 400/89 Underpass Replacement Structure
East Approach Embankment – Front Slope Stability
Final Configuration – Granular Block (Permanent Condition)

Figure 3B

Material Name	γ (kN/m^3)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

Golder Associates

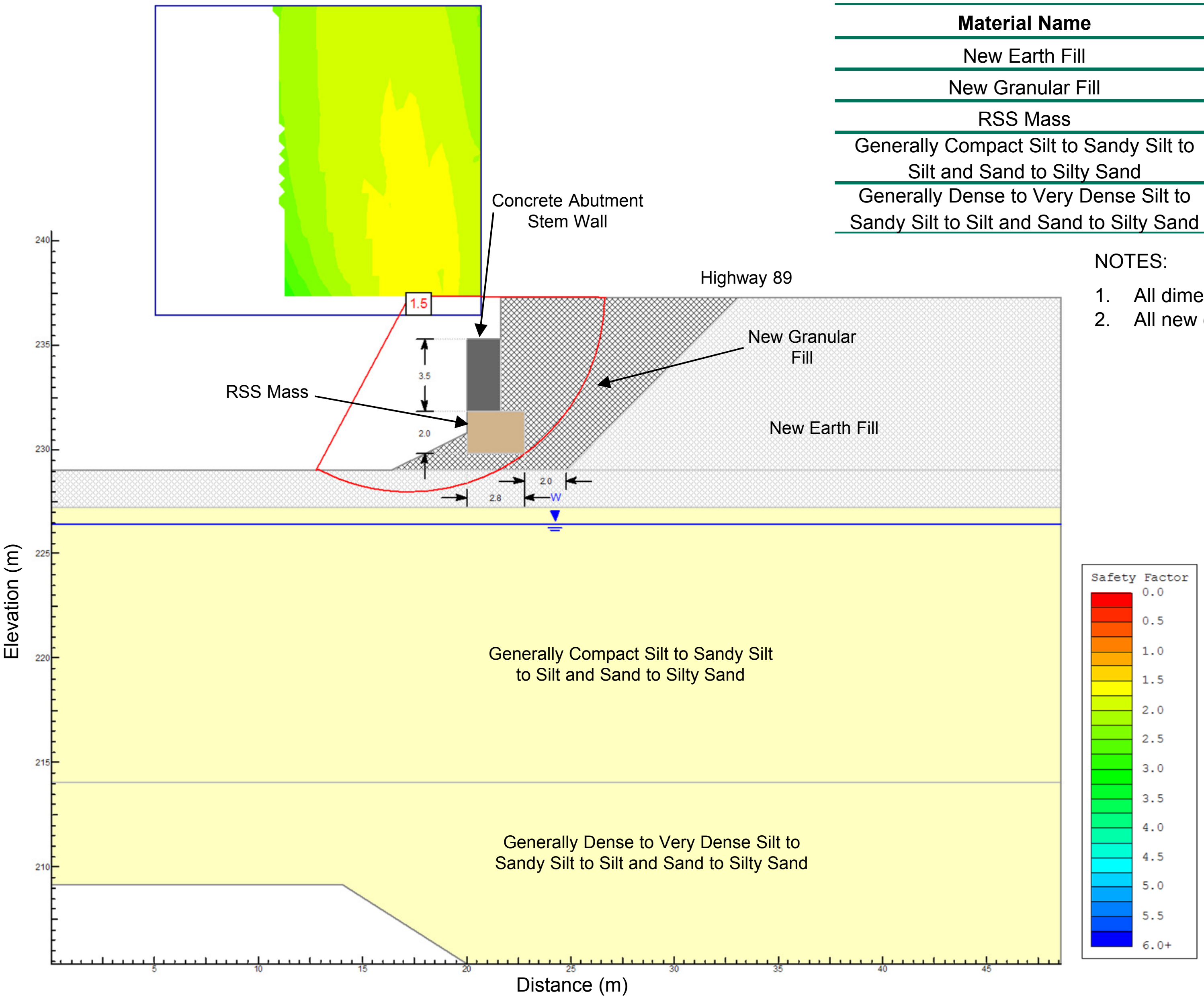
Highway 400/89 Underpass Replacement Structure
East Approach Embankment – Front Slope Stability
Final Configuration – Granular Wedge (Permanent Condition)

Figure 3C

Material Name	ɣ (kN/m³)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

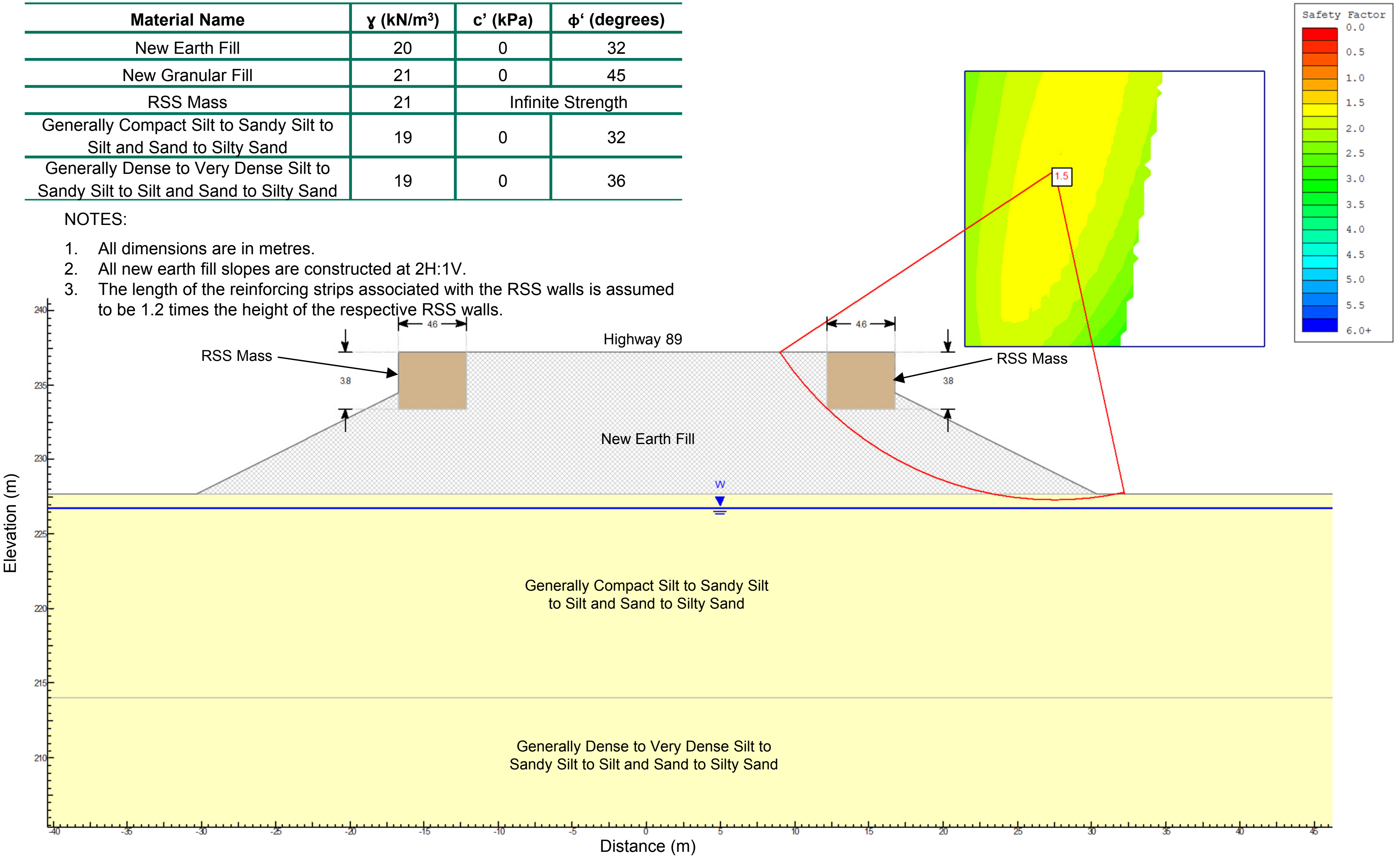
Highway 400/89 Underpass Replacement Structure East Approach Embankment – Side Slope Stability Final Configuration – (Permanent Condition)

Figure 3D

Material Name	ɣ (kN/m³)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.
3. The length of the reinforcing strips associated with the RSS walls is assumed to be 1.2 times the height of the respective RSS walls.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

Golder Associates

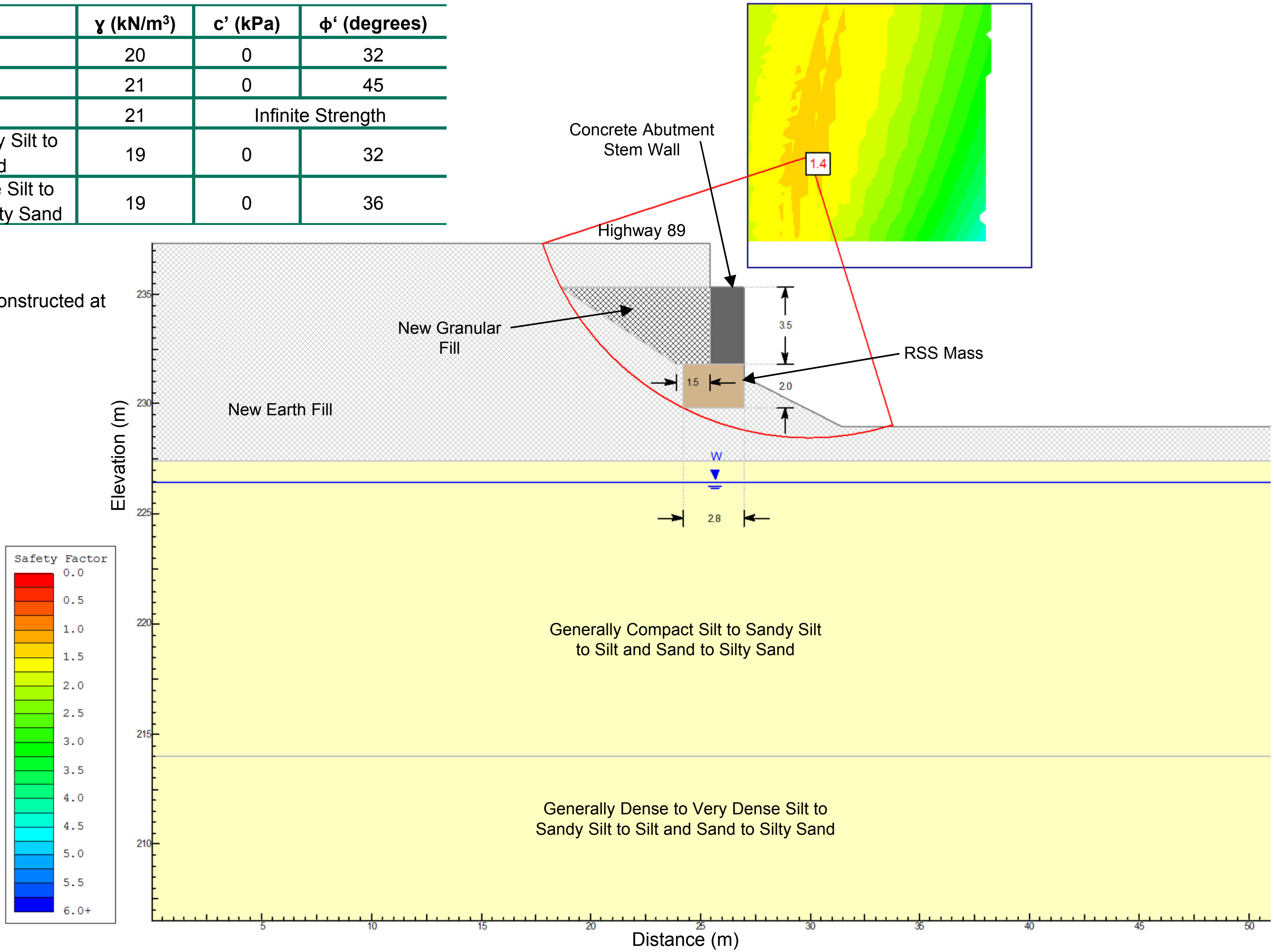
Highway 400/89 Underpass Replacement Structure
West Approach Embankment – Front Slope Stability
Final Configuration (Permanent Condition)

Figure 4A

Material Name	γ (kN/m^3)	c’ (kPa)	ϕ‘ (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

Golder Associates

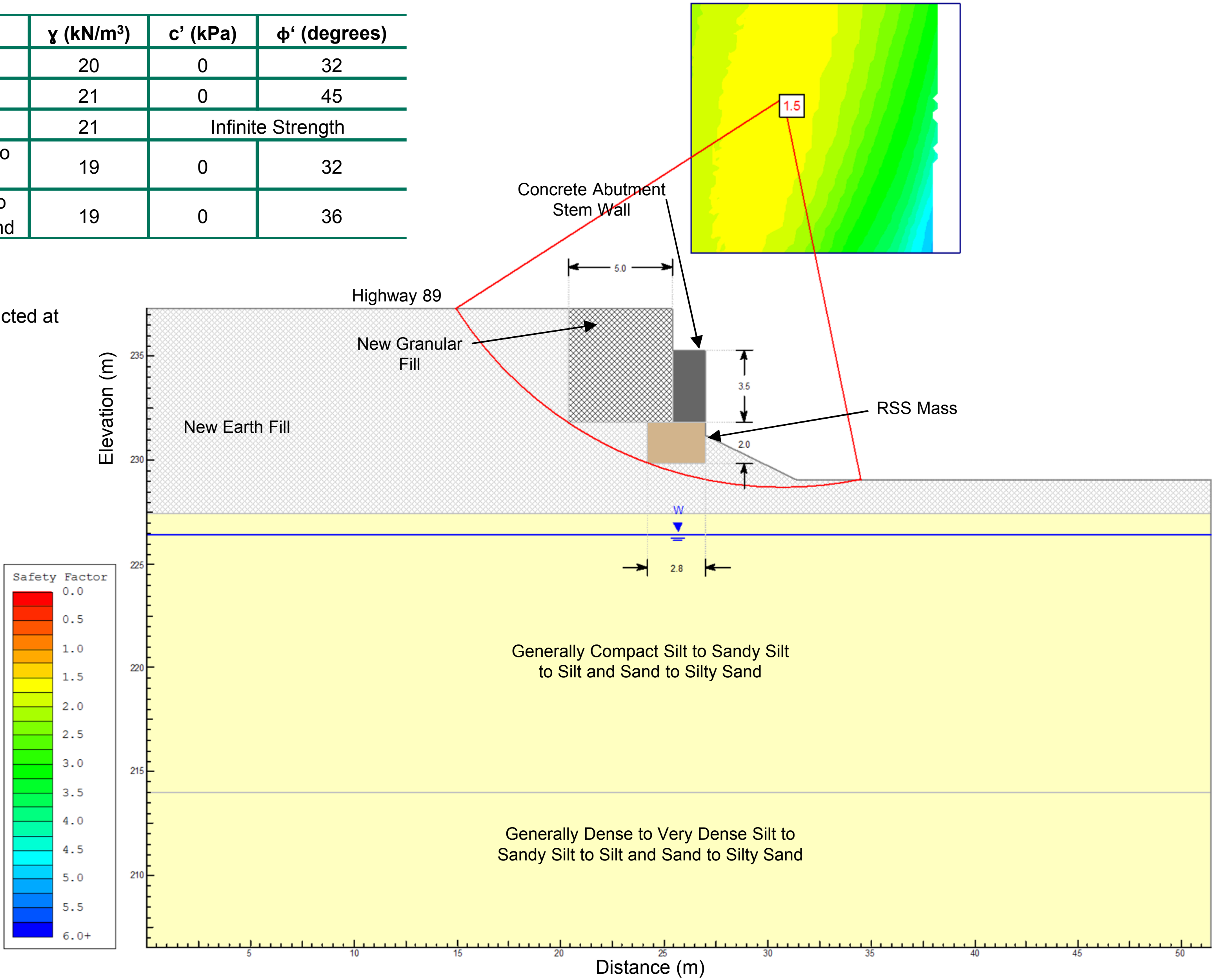
Highway 400/89 Underpass Replacement Structure
West Approach Embankment – Front Slope Stability
Final Configuration – Granular Block (Permanent Condition)

Figure 4B

Material Name	γ (kN/m^3)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

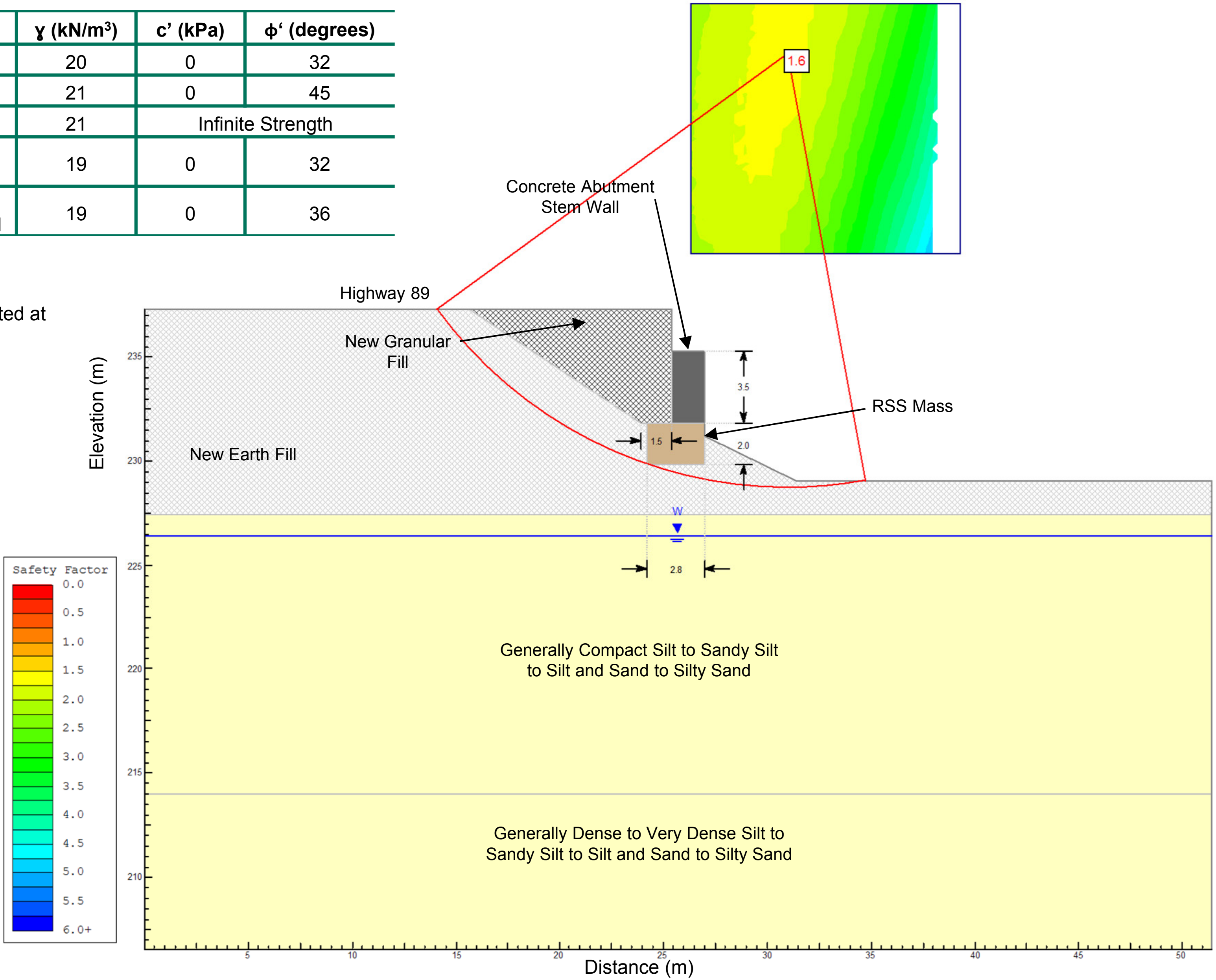
Highway 400/89 Underpass Replacement Structure
West Approach Embankment – Front Slope Stability
Final Configuration – Granular Wedge (Permanent Condition)

Figure 4C

Material Name	γ (kN/m^3)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

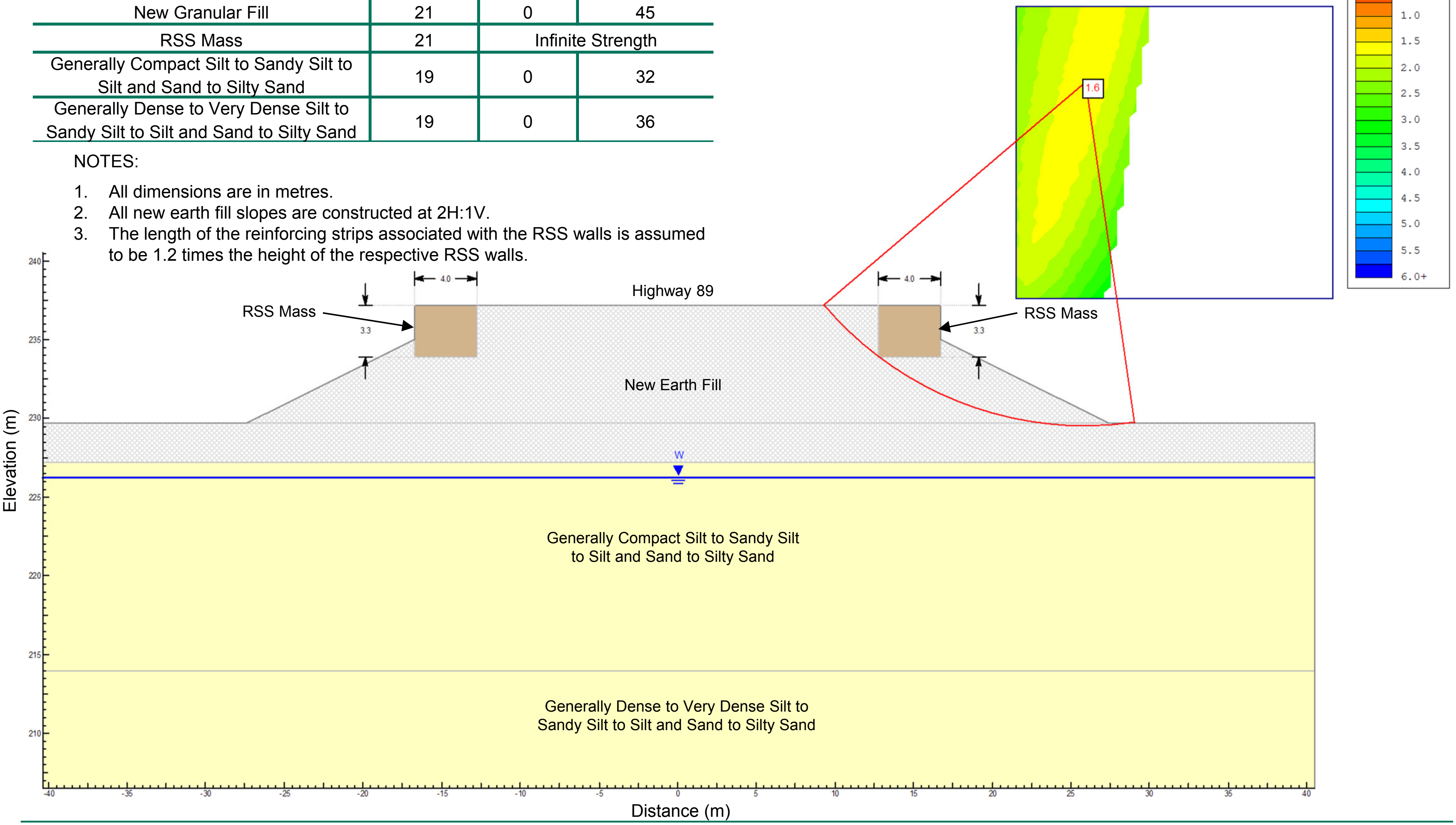
Highway 400/89 Underpass Replacement Structure West Approach Embankment – Side Slope Stability Final Configuration – (Permanent Condition)

Figure 4D

Material Name	ɣ (kN/m³)	c' (kPa)	ϕ' (degrees)
New Earth Fill	20	0	32
New Granular Fill	21	0	45
RSS Mass	21	Infinite Strength	
Generally Compact Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	32
Generally Dense to Very Dense Silt to Sandy Silt to Silt and Sand to Silty Sand	19	0	36

NOTES:

1. All dimensions are in metres.
2. All new earth fill slopes are constructed at 2H:1V.
3. The length of the reinforcing strips associated with the RSS walls is assumed to be 1.2 times the height of the respective RSS walls.



Date: November 16, 2018

Project No: 1668512

Analysis By: TZ **Reviewed By:** LCC

Golder Associates

APPENDIX A

Previous Investigation - MTO GEOCRES No. 31D00-465

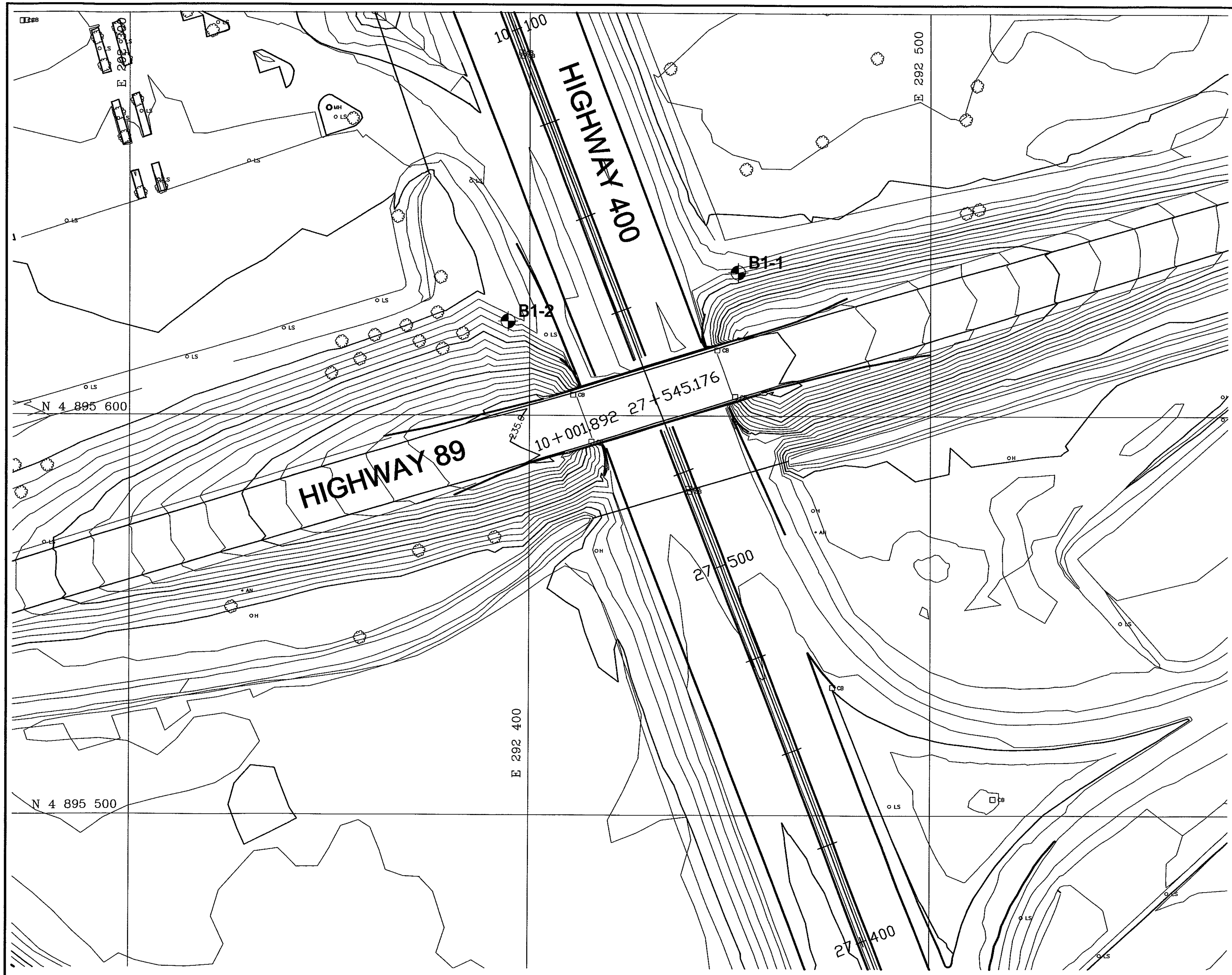
DIST HWY 400
CONT. No.
GWP No. 30-95-00

HIGHWAY 89 UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

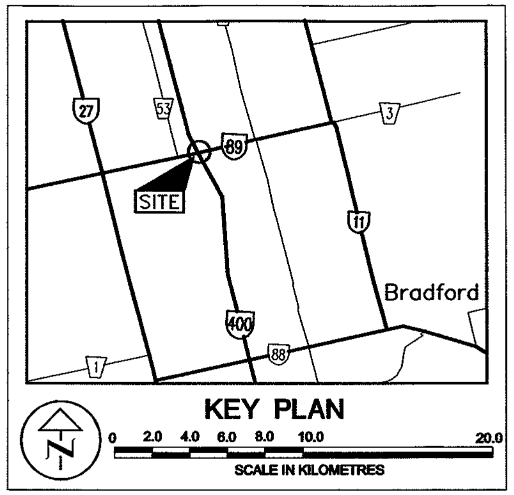
SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



0 10 20 30 40 50 100m
SCALE IN METRES



KEY PLAN

LEGEND

● Borehole, previous investigation

⊕ Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B1-1	228.9	4,895,635.8	292,452.1
B1-2	228.4	4,895,623.5	292,394.6

REFERENCE
This drawing was created from digital file "33811.dwg" provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-256
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F01.DWG

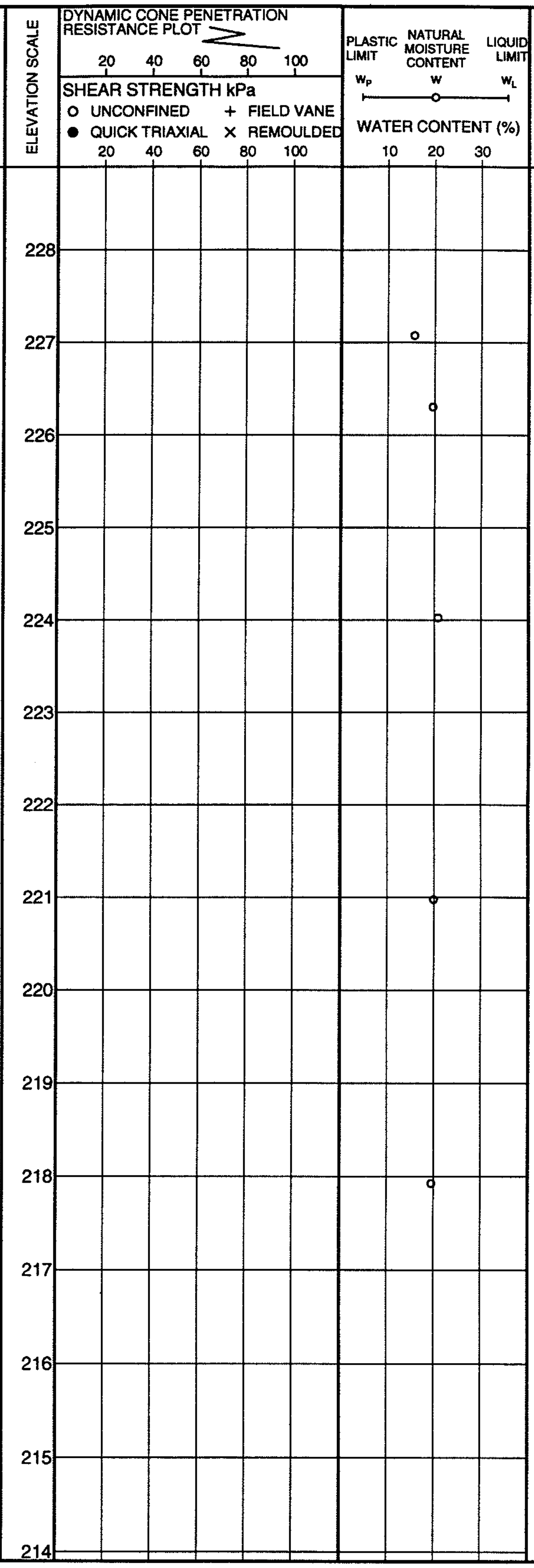
RECORD OF BOREHOLE No B1-1

1 OF 2

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4895635.8; E 292452.1
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Dec.14-18/2000
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
228.9	GROUND SURFACE							
0.0	Silty Sand, trace to some gravel with pockets of clay (Fill) Loose to compact Moist Brown	1	SS	8				
		2	SS	20				
227.7 1.2	Silty Sand to Sandy Silt, trace gravel, trace clay, containing silty clay layers Compact to dense Wet Brown to grey	3	SS	13				
		4	SS	39	0	55	43	2
		5	SS	46				
	Becoming grey below 3.8m depth	6	SS	27				
		7	SS	32	0	24	74	2
		8	SS	27				
		9	SS	30				
		10	SS	37				
	Increasing frequency of silty clay layers, 25mm to 100mm in thickness, below 10.7m depth.	11	SS	33				
		12	SS	34				
		13	SS	22				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

Ministry of Transportation
Ontario

Foundation Design

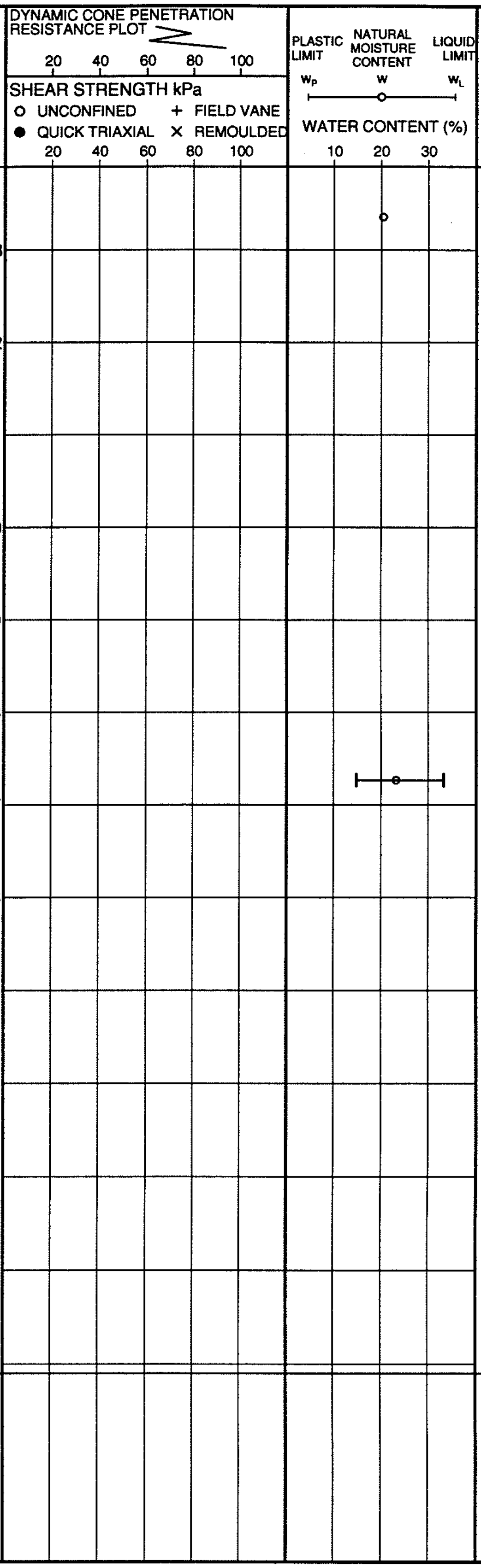
RECORD OF BOREHOLE No B1-1

2 OF 2

METRIC

PROJECT 001-1143F
W.P. 30-95-00
LOCATION N 4895635.8; E 292452.1
ORIGINATED BY PKS
DIST SW HWY 400
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
COMPILED BY LCC
DATUM Geodetic
DATE Dec.14-18/2000
CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —							
	Silty Sand to Sandy Silt, trace gravel, trace clay, containing silty clay layers Compact to dense Wet Brown to grey		14	SS	29		213	
							212	
							211	
			15	SS	38		210	
209.1 19.8	Silty Clay, trace sand Stiff to hard Wet Grey						209	
							208	
			16	SS	40		207	
							206	
							205	
			17	SS	12		204	
							203	
							202	
200.9 28.0			18	SS	19		201	
	END OF BOREHOLE Notes: 1.Hole terminated due to tightening of soil around augers and resulting difficulties in advancing/withdrawing augers. 2.Water level in open borehole on December 15 and 18, 2000 at 2.7m depth (Elev.226.2m)							



UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

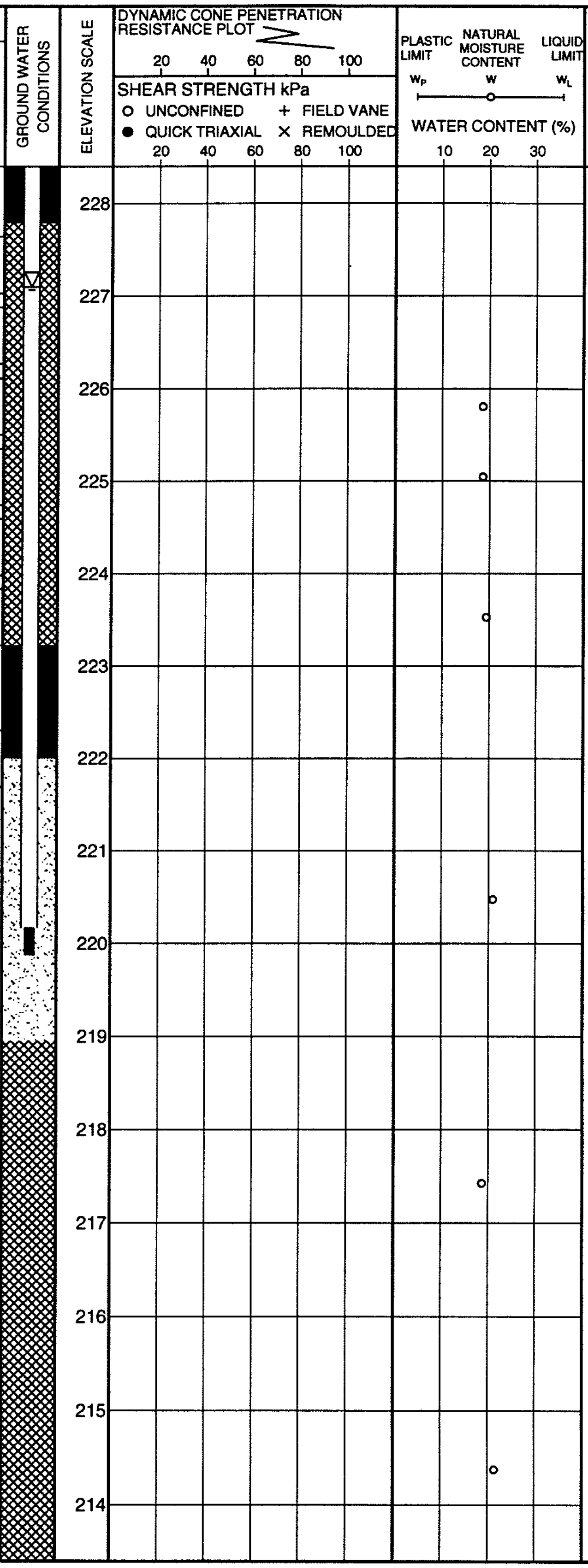
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No B1-2

1 OF 3 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4895623.5; E 292394.6
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING
DATE Dec.14-18/2000
ORIGINATED BY GPD
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
228.4	GROUND SURFACE							
0.0	Silty Sand, trace gravel, trace organics (Fill) Loose to compact Moist Brown	1	SS	14				
		2	SS	8				
226.1 2.3	Silty Sand to Sandy Silt, trace clay Compact Wet Brown	3	SS	13				
		4	SS	23	0	71	29	0
		5	SS	16				
		6	SS	13	0	34	65	1
		7	SS	21				
		8	SS	19				
		9	SS	18				
		10	SS	21				
		11	SS	19				
		12	SS	14				



UNIT WEIGHT γ kN/m³

Continued Next Page

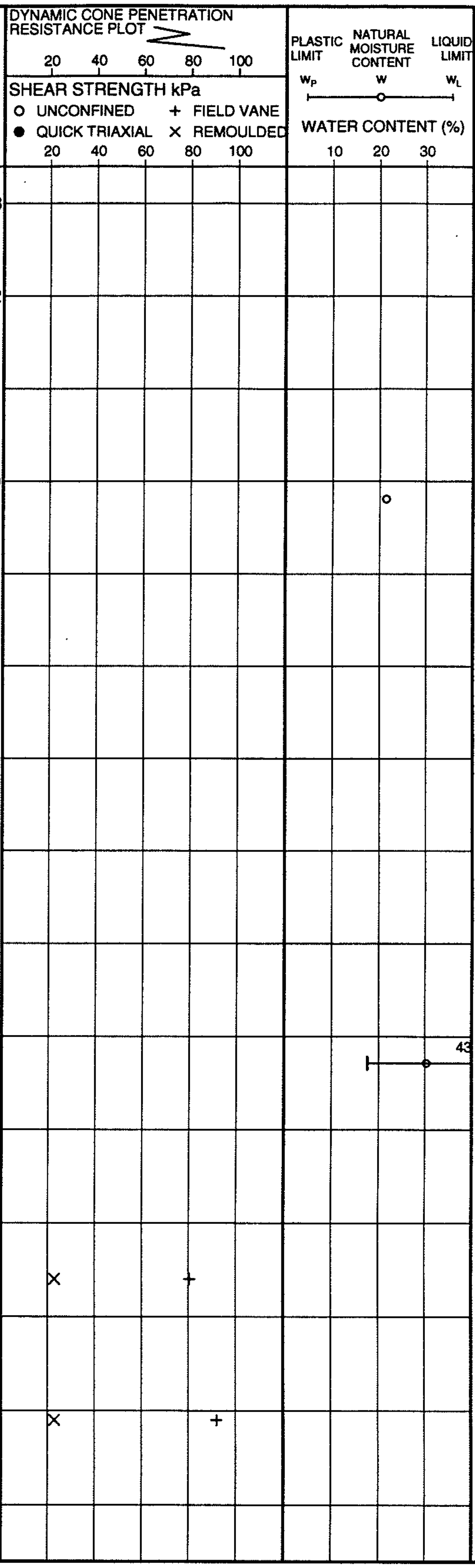
+³, ×³: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No B1-2

2 OF 3 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4895623.5; E 292394.6
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING
DATE Dec.14-18/2000
ORIGINATED BY GPD
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE		SAMPLES			ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION — CONTINUED FROM PREVIOUS PAGE —	NUMBER	TYPE	"N" VALUES				
	Silty Sand to Sandy Silt, trace clay Compact Wet Brown	13	SS	13	213			
					212			
					211			
		14	SS	21	210			
					209			
					208			
		15	SS	18	207			
206.6 21.8	Silty Clay, trace sand Firm to very stiff Moist Grey				206			
					205			
		16	SS	18	204	43		
					203			
		17	SS	8	202			
		18	SS	6	201			
					200			
					199			



Continued Next Page

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

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

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

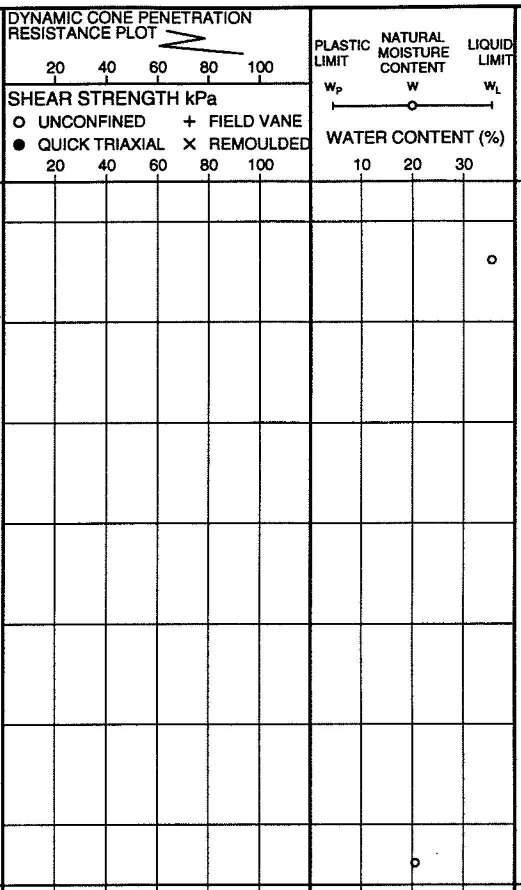
RECORD OF BOREHOLE No B1-2

3 OF 3

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4895623.5; E 292394.6
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING
DATE Dec.14-18/2000
ORIGINATED BY GPD
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
	— CONTINUED FROM PREVIOUS PAGE —						
	Silty Clay, trace sand Firm to very stiff Moist Grey	▨	19	SS	9	▩	198 197 196
194.9 33.5	Silty Sand containing silty clay layers Very dense Wet Grey		20	SS	52	▩	195 194 193 192
			21	SS	72	▩	
191.4 37.0	END OF BOREHOLE						



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Notes:
1. Water level in open borehole at 2.3m depth (Elev.226.1m) on completion of drilling operations.
2. Water level in piezometer at 1.8m depth (Elev.226.6m) on January 19, 2001, and at 1.3m depth (Elev.227.1m) on March 15, 2001.

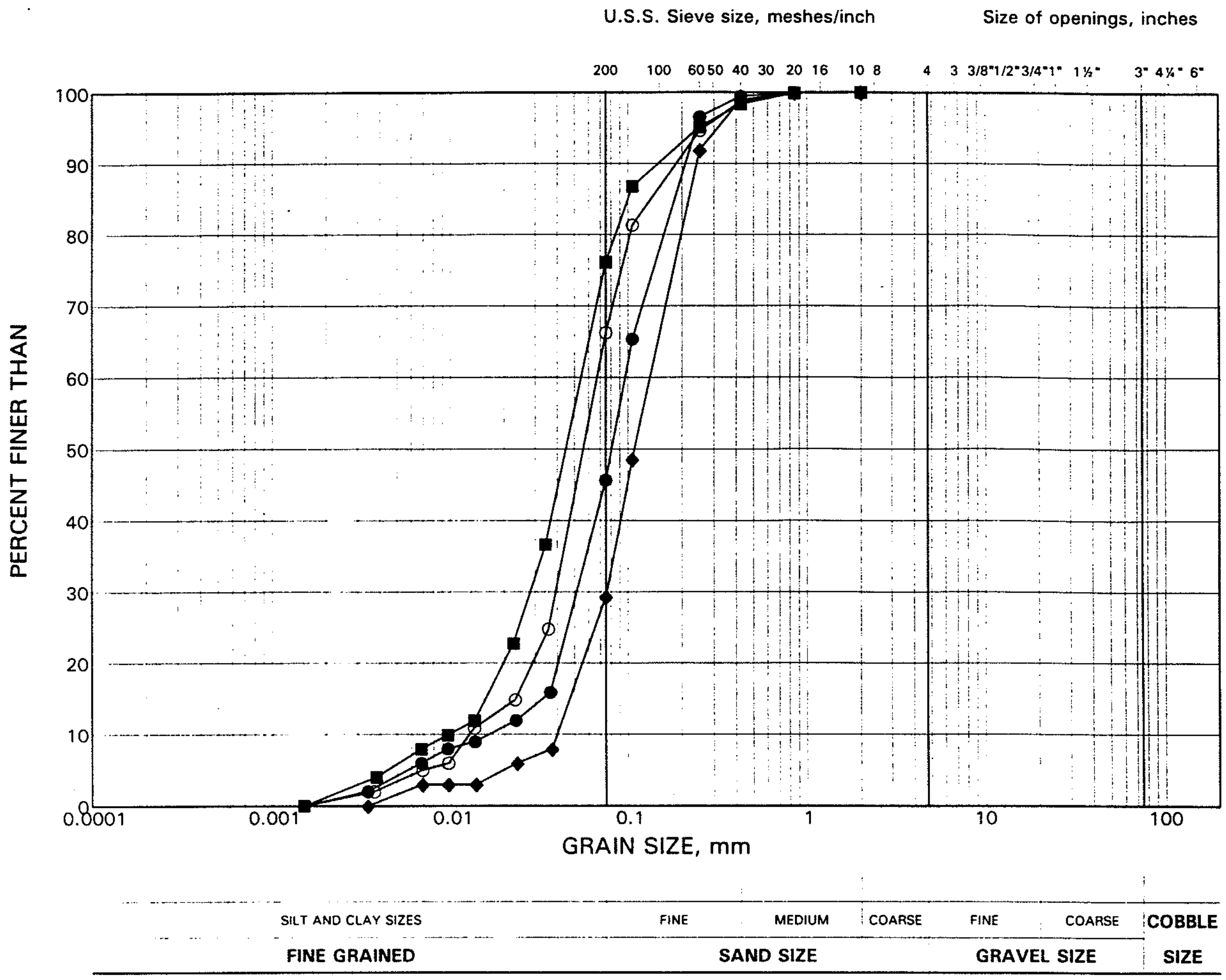
+³, ×³: Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silty Sand to Sandy Silt Deposit

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1-1	4	226.3
■	B1-1	7	224.0
◆	B1-2	4	225.0
○	B1-2	6	223.5

APPENDIX B

Current Investigation - Borehole Records

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

Compactness Condition	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

Version 3 (February 2018)

RECORD OF BOREHOLE No 89UP-01

SHEET 1 OF 1

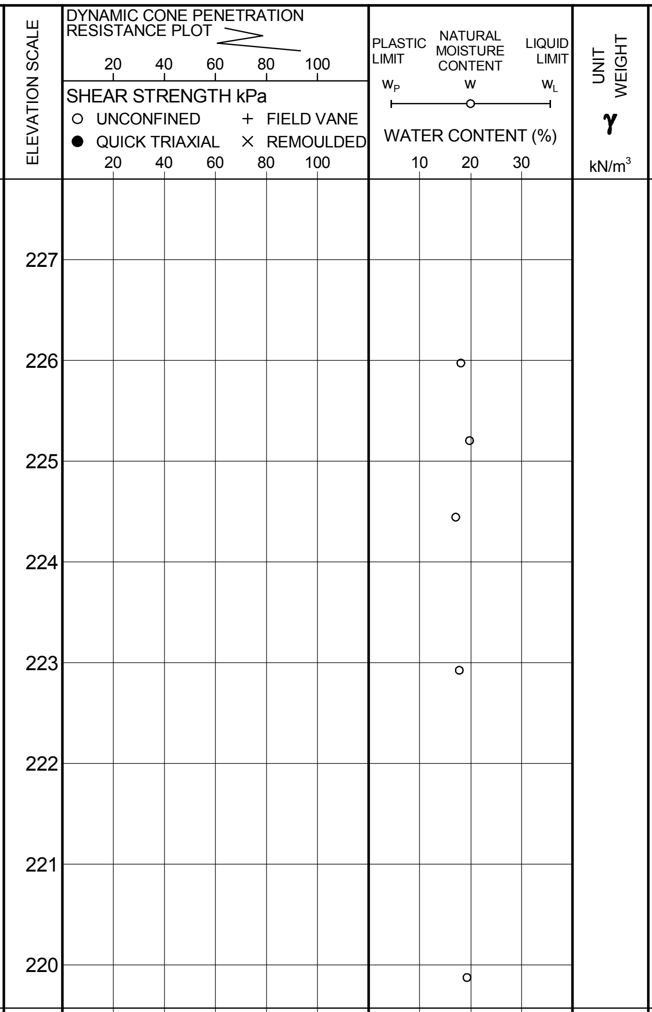
METRIC

PROJECT 1668512
G.W.P. 2438-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4895618.4; E 292361.9 MTM NAD 83 ZONE 10 (LAT. 44.200459; LONG. -79.655616)
BOREHOLE TYPE D50 Track Mount, 203mm O.D. Continuous Flight Hollow Stem Augers
DATE August 15, 2017
ORIGINATED BY DF
COMPILED BY DH
CHECKED BY SMM/TZ

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
227.8	GROUND SURFACE								
0.0	TOPSOIL (200 mm)								
0.2	Gravelly sand, trace silt, trace organics (FILL) Compact Brown Moist	1	SS	18					
227.1 0.7	Silty sand, trace gravel (FILL) Loose to compact Brown mottled with oxidation staining Moist	2	SS	19					
		3	SS	9	∇				
225.6 2.2	SILT and SAND, trace to some clay Compact Grey Wet	4	SS	15					
		5	SS	16		0	57	41	2
		6	SS	16					
		7	SS	11					
		8	SS	15					
		9	SS	14		0	30	64	6
219.6 8.2	END OF BOREHOLE								

NOTE:

1. Water level measured in open borehole at a depth of about 2.2 m below ground surface (Elev. 225.6 m) upon completion of drilling.



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

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_HWY_89_INTERCHANGE\02_DATA\GINT\HWY_400_AND_HWY_89_INTERCHANGE.GPJ GAL-GTA.GDT 09/12/18

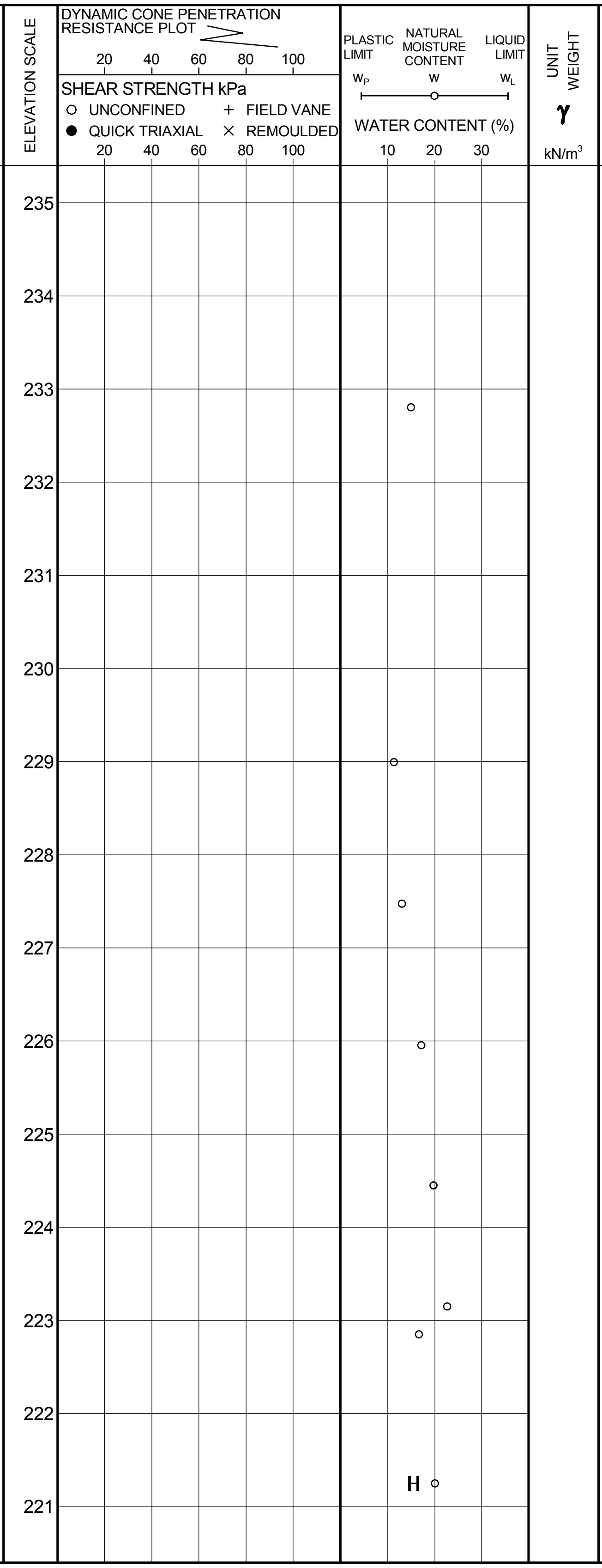
RECORD OF BOREHOLE No 89UP-02

SHEET 1 OF 4

METRIC

PROJECT 1668512			
G.W.P. 2438-13-00	LOCATION N 4895597.2; E 292389.9 MTM NAD 83 ZONE 10 (LAT. 44.200269; LONG. -79.655266)	ORIGINATED BY DF	
DIST Central HWY 400	BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud	COMPILED BY DM	
DATUM Geodetic	DATE June 11 to 15, 2017	CHECKED BY SMM/TZ	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
235.4	GROUND SURFACE									
0.0	ASPHALT (165 mm)									
0.2	Sand and gravel (FILL) (600 mm)									
234.6 0.8	Silt and sand, trace to some clay (FILL) Compact to dense Brown Moist		1	SS	18					
			2	SS	17	∇	0	56	40	4
			3	SS	19					
			4	SS	26					
			5	SS	20					
			6	SS	45		0	51	45	4
			7	SS	33		0	63	30	7
			8	SS	23		0	58	35	7
225.2 10.2	Sandy SILT to SILT and SAND, trace clay Dense Brown to grey Moist to wet - Grey below a depth of about 11.2 m		9A 9B	SS	40					
			10A 10B	SS	41		0	59	39	2
221.4 14.0	SILT, some clay, trace to some sand Compact Grey Moist		11A 11B	SS	28		0	8	78	14



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

RECORD OF BOREHOLE No 89UP-02

SHEET 2 OF 4

METRIC

PROJECT	1668512		
G.W.P.	2438-13-00	LOCATION	N 4895597.2; E 292389.9 MTM NAD 83 ZONE 10 (LAT. 44.200269; LONG. -79.655266)
DIST	Central	HWY	400
BOREHOLE TYPE	D50 Track Mount, NW Casing and Wash Boring with Drilling Mud		
DATUM	Geodetic	DATE	June 11 to 15, 2017
ORIGINATED BY	DF	COMPILED BY	DM
CHECKED BY	SMM/TZ		

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
220.0 15.4	SILT and SAND, trace clay Compact to dense Grey Moist	12A 12B	SS	23	0	42	56	2
		13A 13B	SS	34				
217.6 17.8	SILT, trace to some sand, trace clay Compact to very dense Grey Moist to wet	14	SS	17				
		15	SS	27				
		16	SS	26				
		17	SS	41	0	8	88	4
		18	SS	43				
		19	SS	62				
		20	SS	61	0	19	77	4
206.9 28.5	Varved SILTY CLAY, with silt and clay laminae Firm to Stiff Grey Moist	21	SS	21				

ELEVATION SCALE: 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

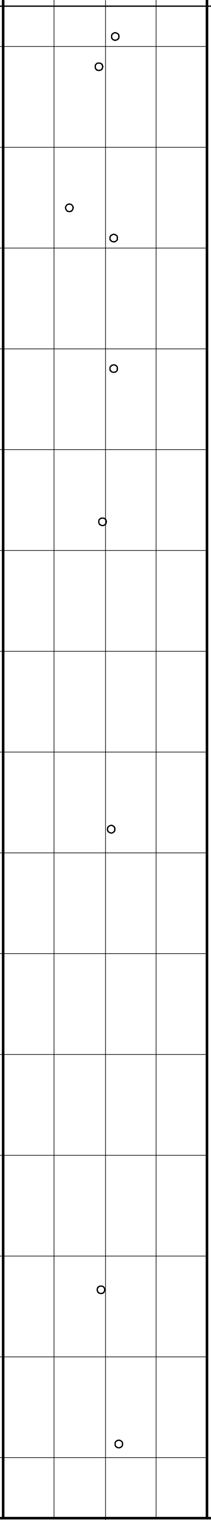
SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100

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 (%)



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+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 89UP-02

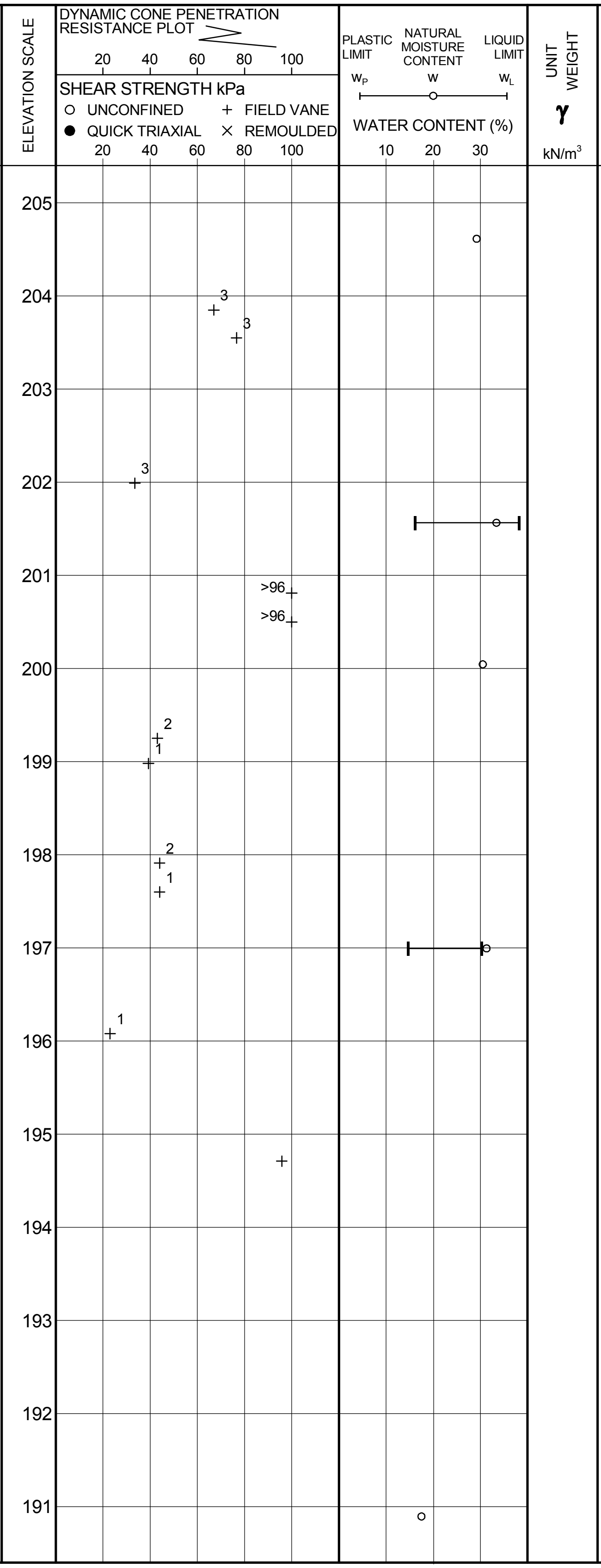
SHEET 3 OF 4

METRIC

PROJECT 1668512
G.W.P. 2438-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4895597.2; E 292389.9 MTM NAD 83 ZONE 10 (LAT. 44.200269; LONG. -79.655266)
BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud
DATE June 11 to 15, 2017
ORIGINATED BY DF
COMPILED BY DM
CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Varved SILTY CLAY, with silt and clay laminae Firm to Stiff Grey Moist	22	SS	3	
		23	TO	PH	
		24	SS	WH	
		25	SS	WH	
197.4 38.0	CLAYEY SILT, trace sand Soft to firm Grey Wet	26	SS	WH	
		27	SS	WH	
194.5 40.9	SILT, trace to some sand, trace clay Very dense Grey Moist	28	SS	81	
		29	SS	55	0 9 89 2



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

Golder Associates

RECORD OF BOREHOLE No 89UP-02

SHEET 4 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895597.2; E 292389.9 MTM NAD 83 ZONE 10 (LAT. 44.200269; LONG. -79.655266)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE June 11 to 15, 2017

ORIGINATED BY DF

COMPILED BY DM

CHECKED BY SMM/TZ

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---													
	SILT, trace to some sand, trace clay Very dense Grey Moist						190							
189.4 46.0	CLAYEY SILT, trace sand Hard Grey Moist						189							
			30	SS	77		188				0	2	75	23
							187							
							186							
			31	SS	100/0.15		185							
184.6 50.8	END OF BOREHOLE													

NOTE:

1. Water level measurements in the casing at the beginning of each work shift:

Date	Depth (m)	Elev. (m)
12/06/17	11.3	224.1
13/06/17	9.3	226.1
14/06/17	5.0	230.4
15/06/17	2.0	233.4

The water level measurements are not considered to be representative of the groundwater level due to introduction of water/drilling mud during wash boring operations.

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

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

RECORD OF BOREHOLE No 89UP-03

SHEET 1 OF 4

METRIC

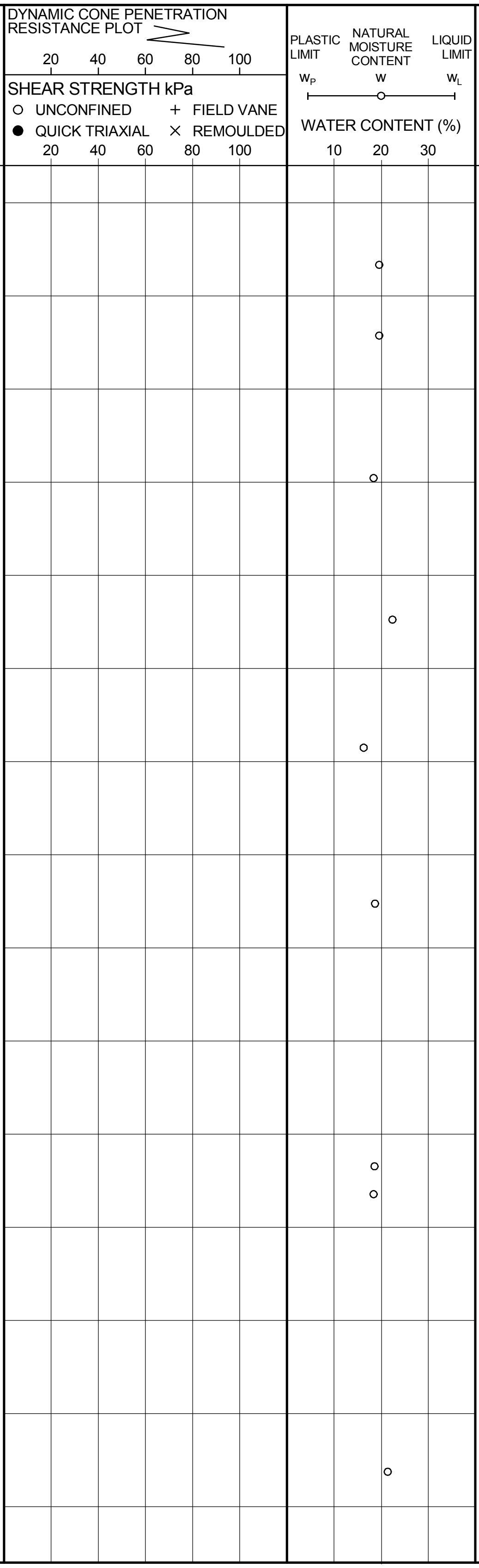
PROJECT 1668512	RECORD OF BOREHOLE No 89UP-03		
G.W.P. 2438-13-00	LOCATION N 4895628.3; E 292375.2 MTM NAD 83 ZONE 10 (LAT. 44.200549; LONG. -79.655451)	ORIGINATED BY DF
DIST Central HWY 400	BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud	COMPILED BY DH
DATUM Geodetic	DATE July 17 to 21, 2017	CHECKED BY SMM/TZ

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
227.4	GROUND SURFACE
0.0	TOPSOIL
0.2	Gravelly sand, some silt (FILL) Loose Brown Moist
226.7 0.7	SILT, trace to some sand to SILT and SAND, trace to some clay Loose to very dense Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	6	227				
2	SS	6	226				
3	SS	16					
4	SS	22	225				
5	SS	17	224	0	44	48	8
6	SS	22	223				
7	SS	13	222				
8A 8B	SS	23	221				
9	SS	17	220				
10	SS	10	218				
11A 11B	SS	33	217				
12	SS	16	215				
13	SS	17	214	0	13	75	12



UNIT WEIGHT γ kN/m³

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+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 89UP-03 SHEET 2 OF 4 METRIC

PROJECT	1668512		
G.W.P.	2438-13-00	LOCATION	N 4895628.3; E 292375.2 MTM NAD 83 ZONE 10 (LAT. 44.200549; LONG. -79.655451)
DIST	Central HWY 400	BOREHOLE TYPE	D50 Track Mount, NW Casing and Wash Boring with Drilling Mud
DATUM	Geodetic	DATE	July 17 to 21, 2017
ORIGINATED BY	DF	COMPILED BY	DH
CHECKED BY	SMM/TZ		

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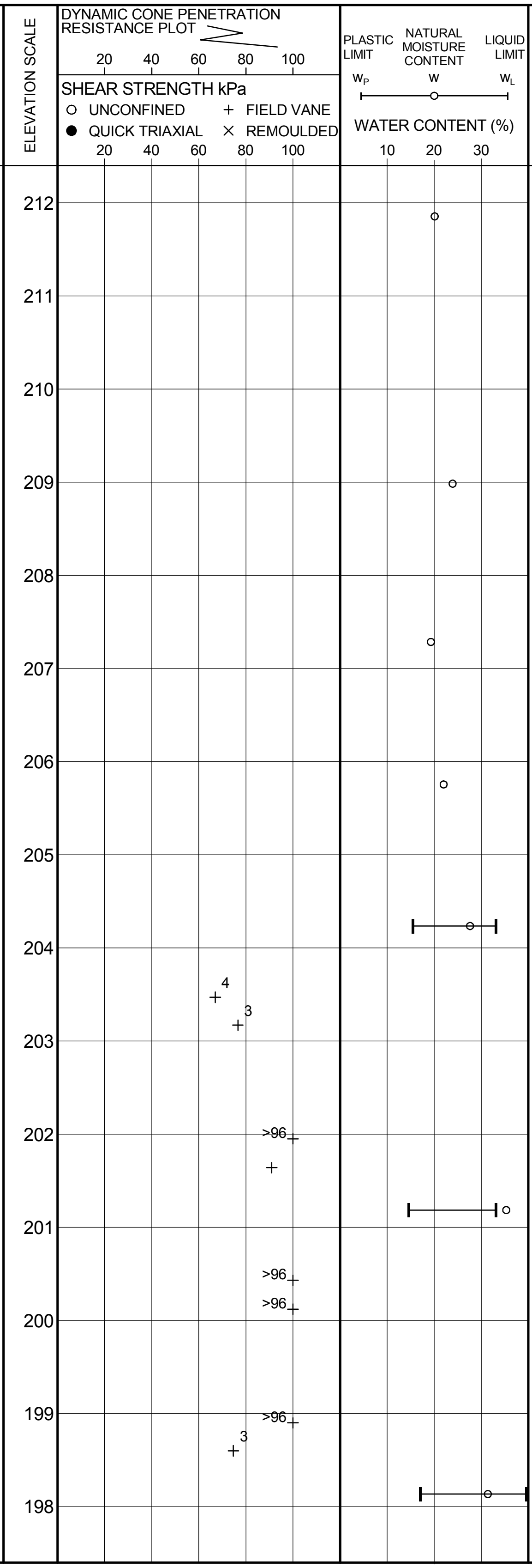
ELEV DEPTH	SOIL PROFILE DESCRIPTION
	SILT, trace to some sand to SILT and SAND, trace to some clay Loose to very dense Grey Wet
209.6 17.8	SILTY CLAY, trace sand Grey Moist
208.9 18.5	SILT and SAND Dense to very dense Grey Wet
206.5 20.9	Varved CLAYEY SILT to SILTY CLAY with silt and clay laminae Stiff to very stiff Grey Moist - Sand inclusions from 20.9 m to 22.4 m

SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
14	SS	37	
15	SS	80	
16A 16B	SS	44	
17	SS	59	
18	SS	11	
19	SS	8	
20	TO	PH	18.5 (C)
21	SS	3	
22	TO	PH	
23	SS	4	



REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Foundation Design

RECORD OF BOREHOLE No 89UP-03

SHEET 3 OF 4

METRIC

PROJECT 1668512			
G.W.P. 2438-13-00	LOCATION N 4895628.3; E 292375.2 MTM NAD 83 ZONE 10 (LAT. 44.200549; LONG. -79.655451)	ORIGINATED BY DF
DIST Central HWY 400	BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud	COMPILED BY DH
DATUM Geodetic	DATE July 17 to 21, 2017	CHECKED BY SMM/TZ

ELEV DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES — NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---							
	Varved CLAYEY SILT to SILTY CLAY with silt and clay laminae Stiff to very stiff Grey Moist	24	SS	9				
195.5 31.9	SILT, some sand, trace clay Compact to very dense Grey Wet - Clayey silt inclusions encountered between depths of about 32.0 m and 32.6 m	25	SS	27				
		26	SS	86	0	15	84	1
192.0 35.4	Sandy SILT, trace clay Very dense Grey Wet	27A 27B	SS	100				
189.0 38.4	CLAYEY SILT, some sand Very stiff Grey Moist	28	SS	17				
		29	SS	25	0	16	63	21
182.9 44.5	CLAYEY SILT (TILL) Grey Moist							

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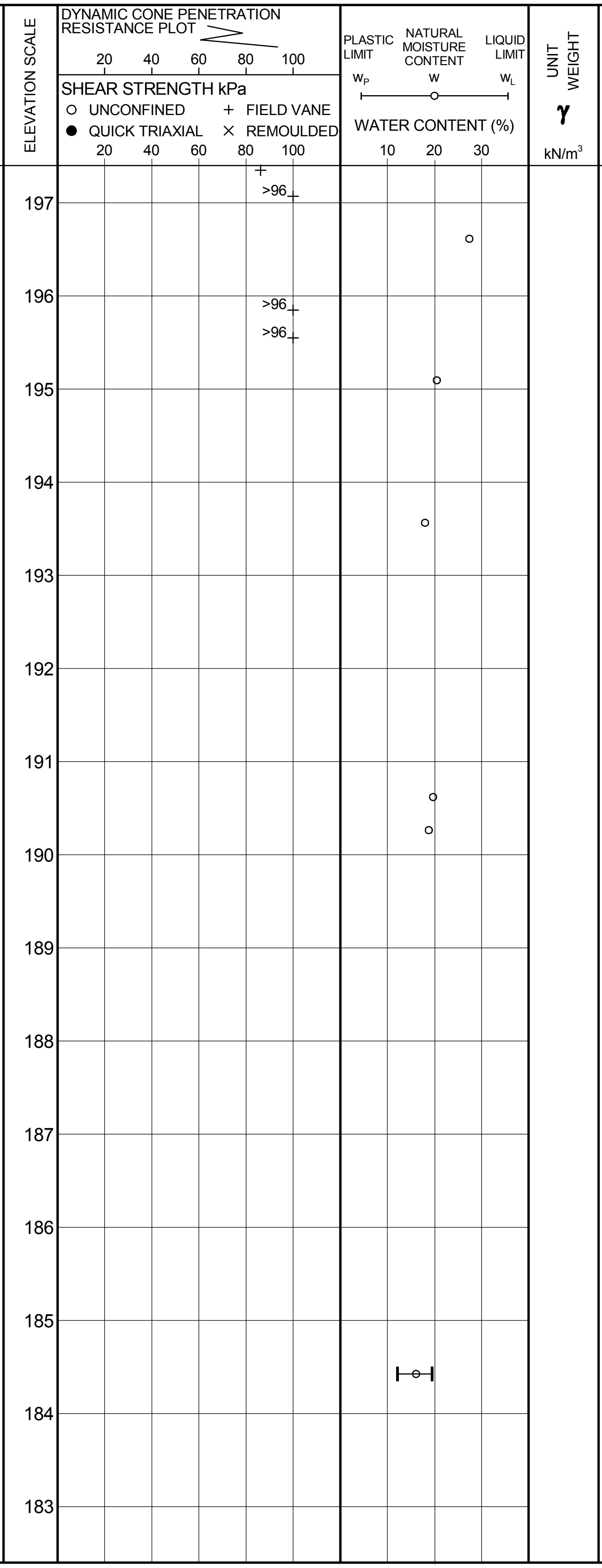


+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_HWY_89_INTERCHANGE\02_DATA\GINT\HWY_400_AND_HWY_89_INTERCHANGE.GPJ GAL-GTA.GDT 09/12/18

RECORD OF BOREHOLE No 89UP-03

SHEET 4 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895628.3; E 292375.2 MTM NAD 83 ZONE 10 (LAT. 44.200549; LONG. -79.655451)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 17 to 21, 2017

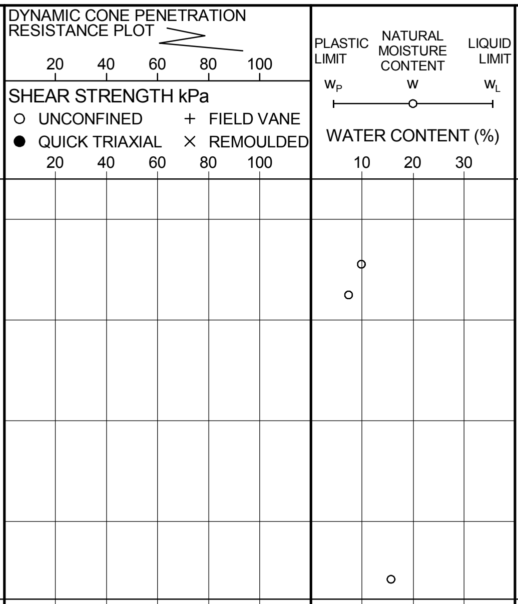
ORIGINATED BY DF

COMPILED BY DH

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
	CLAYEY SILT (TILL) Grey Moist					182				
181.4 46.0	SILT and SAND, trace gravel, trace clay (TILL) Very dense Grey Wet	30A 30B	SS	101		181 180 179				
		31	SS	100/0.10			3	54	42	1
178.2 49.2	END OF BOREHOLE									



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:

1. Water level measurements in the casing at the begining of each work shift:

Date	Depth (m)	Elev. (m)
18/07/17	0.7	226.7
19/07/17	1.6	225.8
20/07/17	0.0	227.4
21/07/17	3.3	224.1

2. A borehole was advanced to a depth of about 4.0 m immediatly next to borehole 89UP-03 in order to install a standpipe piezometer.

3. Water level measurements in standpipe piezometer:

Date	Depth (m)	Elev. (m)
03/08/17	1.0	226.4
10/08/17	1.0	226.4
15/08/17	1.2	226.2
19/09/17	1.3	226.1
05/03/18	0.7	226.7
16/05/18	0.5	226.9

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

RECORD OF BOREHOLE No 89UP-04

SHEET 1 OF 4

METRIC

PROJECT 1668512
G.W.P. 2438-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4895619.3; E 292430.3 MTM NAD 83 ZONE 10 (LAT. 44.200469; LONG. -79.654761)
BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud
DATE July 4, 5, 24 and 25, 2017
ORIGINATED BY DF
COMPILED BY DH
CHECKED BY SMM/TZ

SOIL PROFILE

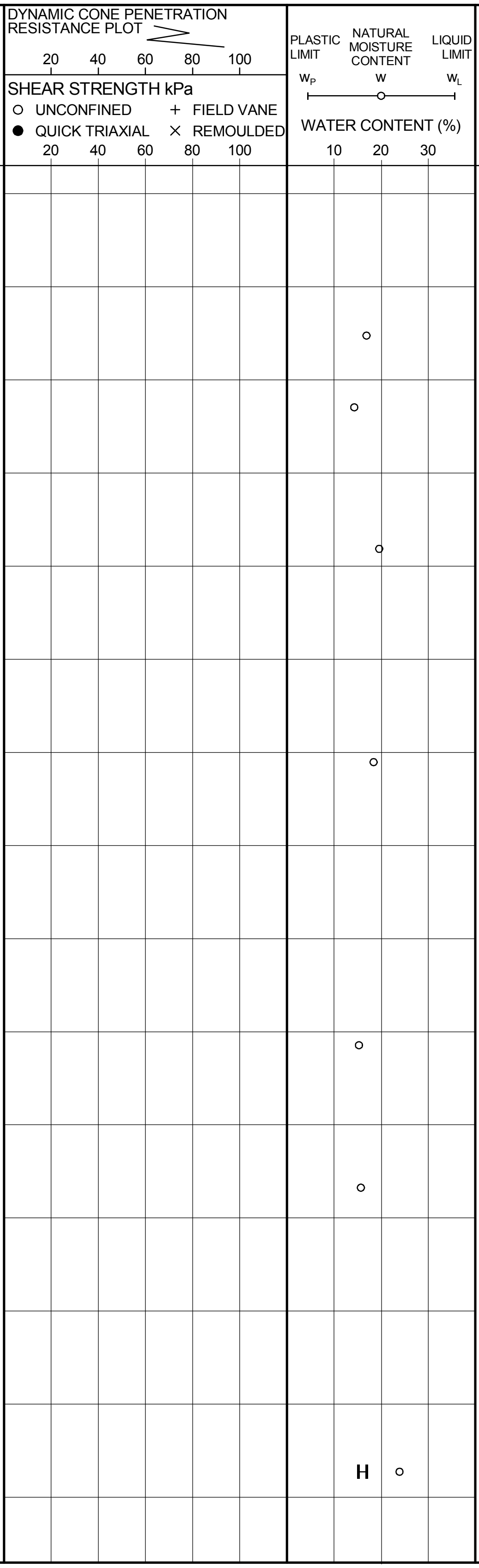
ELEV DEPTH	DESCRIPTION
229.3	GROUND SURFACE
0.0	ASPHALT (250 mm)
	Sand and gravel (FILL) (280 mm)
228.8 0.5	Silt and sand, trace clay, trace organics (FILL) Compact Brown to grey Moist
226.3 3.0	SILT and SAND, trace clay Compact to dense Grey Moist to wet
217.6 11.7	SILT, trace to some sand, trace to some clay Compact Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
1	SS	15		0	67	31	2
2	SS	12					
3	SS	16					
4	SS	38					
5	SS	22					
6	SS	26		0	53	42	5
7	SS	19	∇				
8	SS	19					
9	SS	15					
10	SS	18					
11	SS	20		0	8	84	8

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

RECORD OF BOREHOLE No 89UP-04

SHEET 2 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895619.3; E 292430.3 MTM NAD 83 ZONE 10 (LAT. 44.200469; LONG. -79.654761)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 4, 5, 24 and 25, 2017

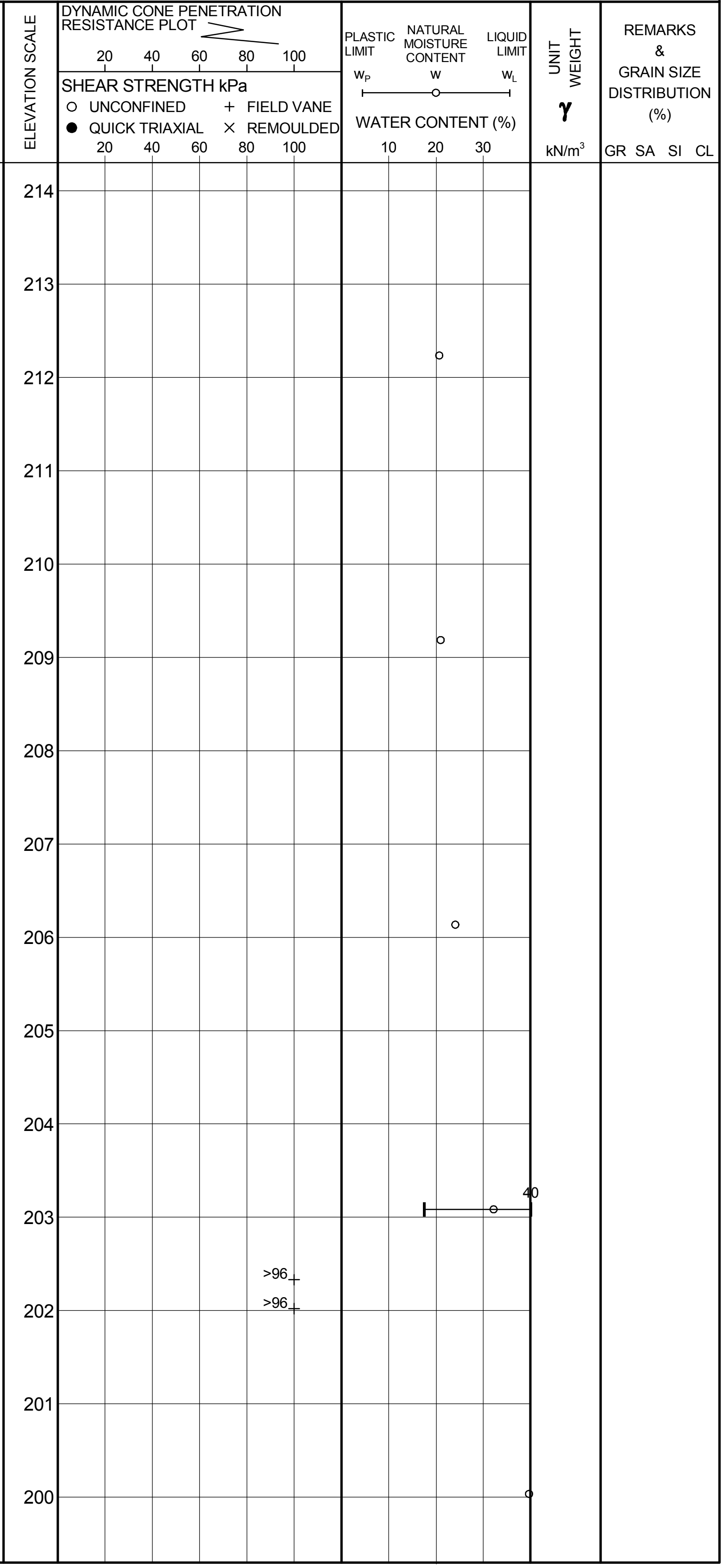
ORIGINATED BY DF

COMPILED BY DH

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES
	SILT, trace to some sand, trace to some clay Compact Grey Wet	12	SS	19
213.0 16.3	Silty SAND Compact to dense Grey Wet	13	SS	28
	- Silty clay layer encountered between depths of about 18.3 m and 18.4 m	14	SS	27
		15	SS	43
207.7 21.6	Varved SILTY CLAY, with silt and clay laminae Very stiff to hard Grey Moist	16	SS	32
		17	SS	6
		18	SS	7



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

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

RECORD OF BOREHOLE No 89UP-04

SHEET 3 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895619.3; E 292430.3 MTM NAD 83 ZONE 10 (LAT. 44.200469; LONG. -79.654761)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 4, 5, 24 and 25, 2017

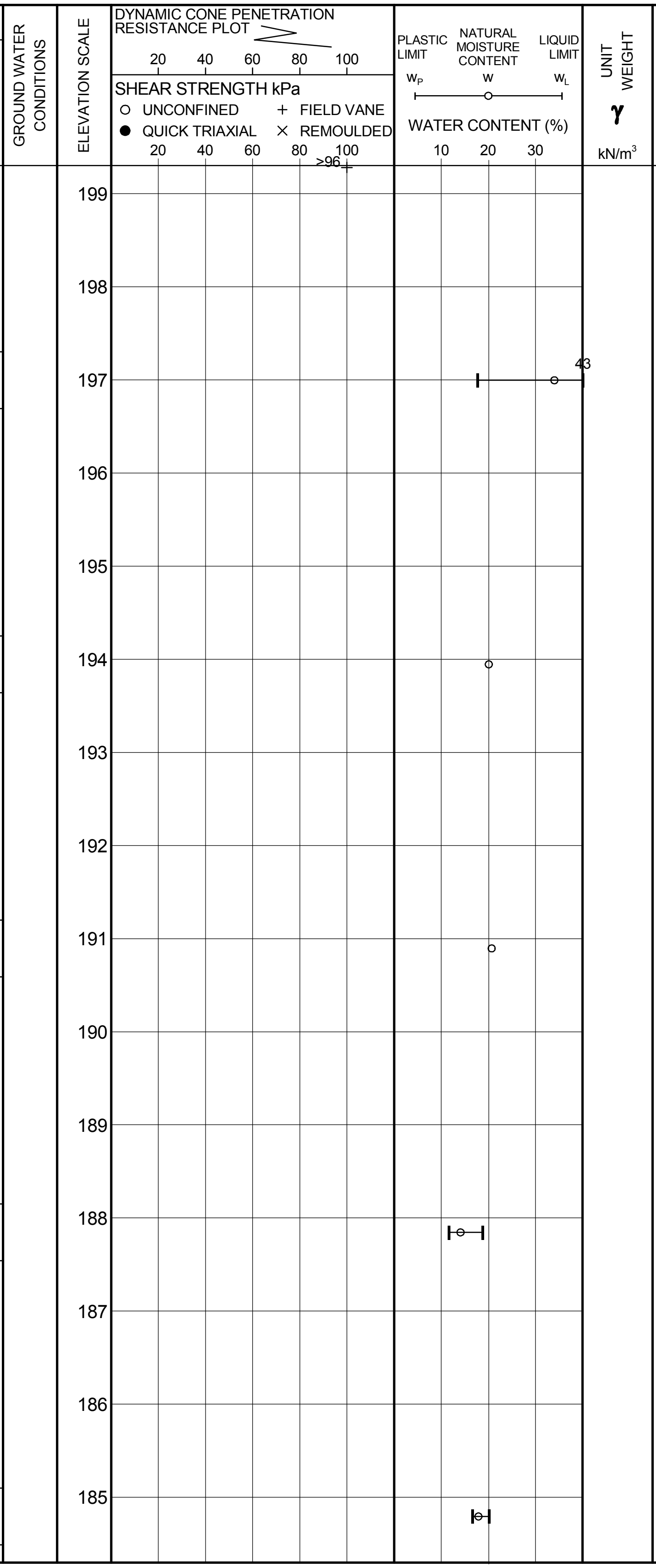
ORIGINATED BY DF

COMPILED BY DH

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	Varved SILTY CLAY, with silt and clay laminae Very stiff to hard Grey Moist							
		19	SS	10				
195.5 33.8	SILT, trace sand, trace clay Dense to very dense Grey Wet							
		20	SS	33				
		21	SS	94	0	5	93	2
190.0 39.3	Sandy CLAYEY SILT, some sand Very stiff Grey Moist							
		22	SS	18	1	20	59	20
186.3 43.0	SILT, trace to some clay, trace sand Very dense Grey Moist							
		23	SS	106				



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

RECORD OF BOREHOLE No 89UP-04

SHEET 4 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895619.3; E 292430.3 MTM NAD 83 ZONE 10 (LAT. 44.200469; LONG. -79.654761)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 4, 5, 24 and 25, 2017

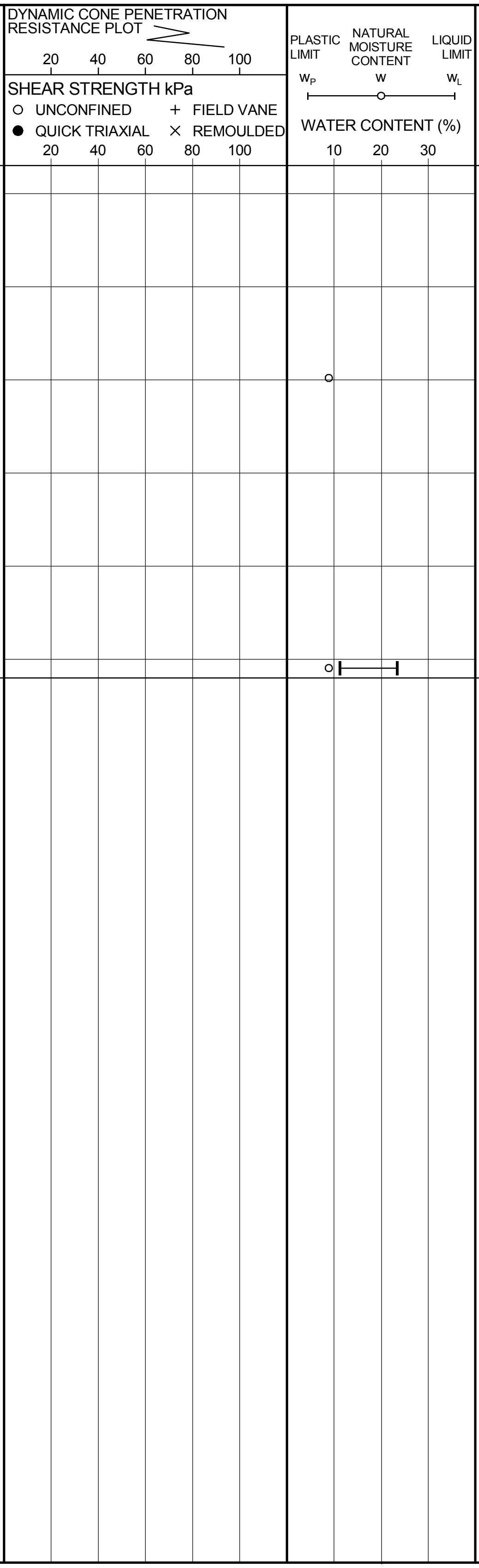
ORIGINATED BY DF

COMPILED BY DH

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT, trace to some clay, trace sand Very dense Grey Moist					
183.3 46.0	Silty SAND, trace to some gravel, trace clay (TILL) Very dense Grey Moist	24	SS	100/0.08		
180.3 49.0	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist	25	SS	100/0.09		5 30 46 19
178.8 50.5	END OF BOREHOLE					



ELEVATION SCALE: 184, 183, 182, 181, 180, 179

UNIT WEIGHT γ kN/m³

NOTE:

1. Water level measurements in the casing at the beginning of each work shift:

Date	Depth (m)	Elev. (m)
05/07/17	0.8	228.5
24/07/17	3.8	225.5
25/07/17	8.2	221.1

The water level measurements are not considered to be representative of the groundwater level due to introduction of water/drilling mud during wash boring operations.

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

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

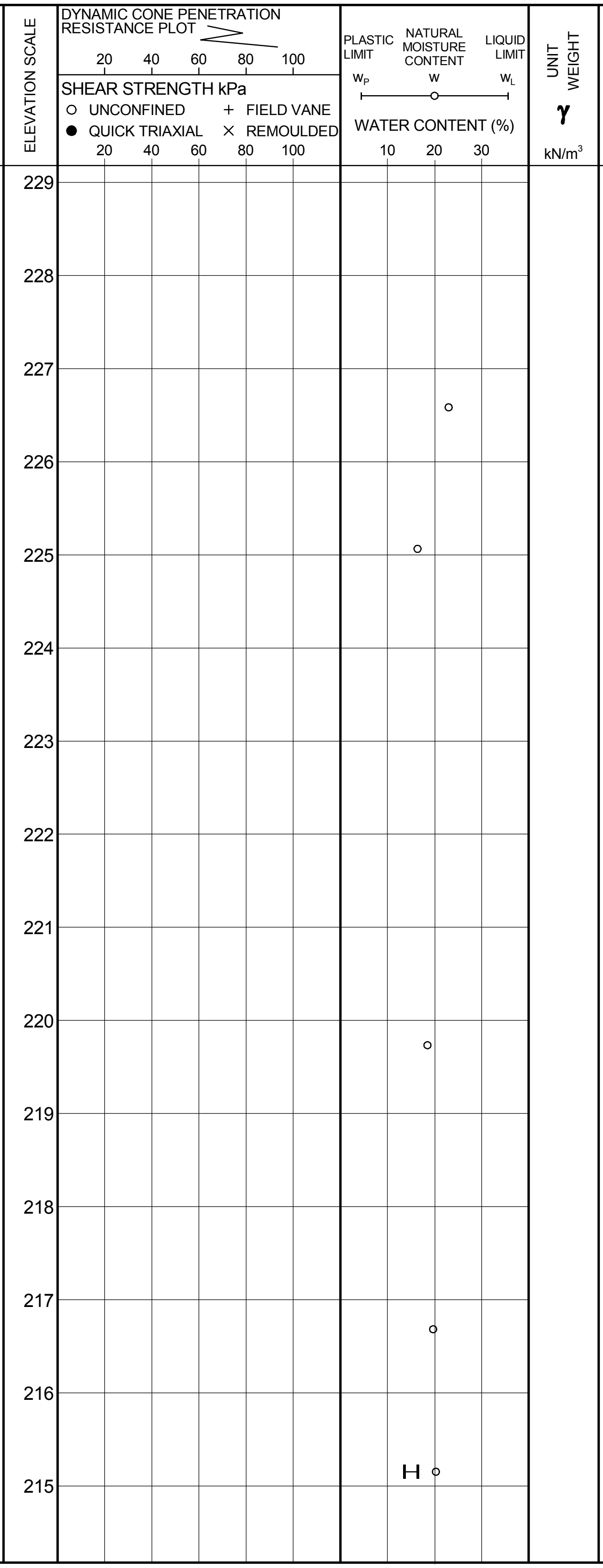
RECORD OF BOREHOLE No 89UP-05

SHEET 1 OF 4

METRIC

PROJECT 1668512			
G.W.P. 2438-13-00	LOCATION N 4895649.6; E 292418.6 MTM NAD 83 ZONE 10 (LAT. 44.200750; LONG. -79.654912)	ORIGINATED BY DF	
DIST Central HWY 400	BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud	COMPILED BY DM	
DATUM Geodetic	DATE June 26 to 29 and July 3, 2017	CHECKED BY SMM/TZ	

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
229.2	GROUND SURFACE								
0.0	ASPHALT (250 mm)								
	Sand and gravel (FILL) (250 mm)								
0.5	Silt and sand, trace to some clay with organic odour (FILL) Very loose to loose Brown mottled with grey Moist to wet								
		1	SS	6					
		2	SS	3		0	45	46	9
226.2 3.0	Silty SAND Compact Grey Wet								
		3	SS	17					
		4	SS	20					
		5	SS	26					
223.6 5.6	SILT to Sandy SILT, trace clay with clayey silt pockets Compact to very dense Grey Wet								
		6	SS	14					
		7	SS	16					
					∇				
		8	SS	20		0	26	73	1
		9	SS	25					
		10	SS	15					
		11	SS	20		0	11	77	12



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

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

RECORD OF BOREHOLE No 89UP-05

SHEET 2 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895649.6; E 292418.6 MTM NAD 83 ZONE 10 (LAT. 44.200750; LONG. -79.654912)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE June 26 to 29 and July 3, 2017

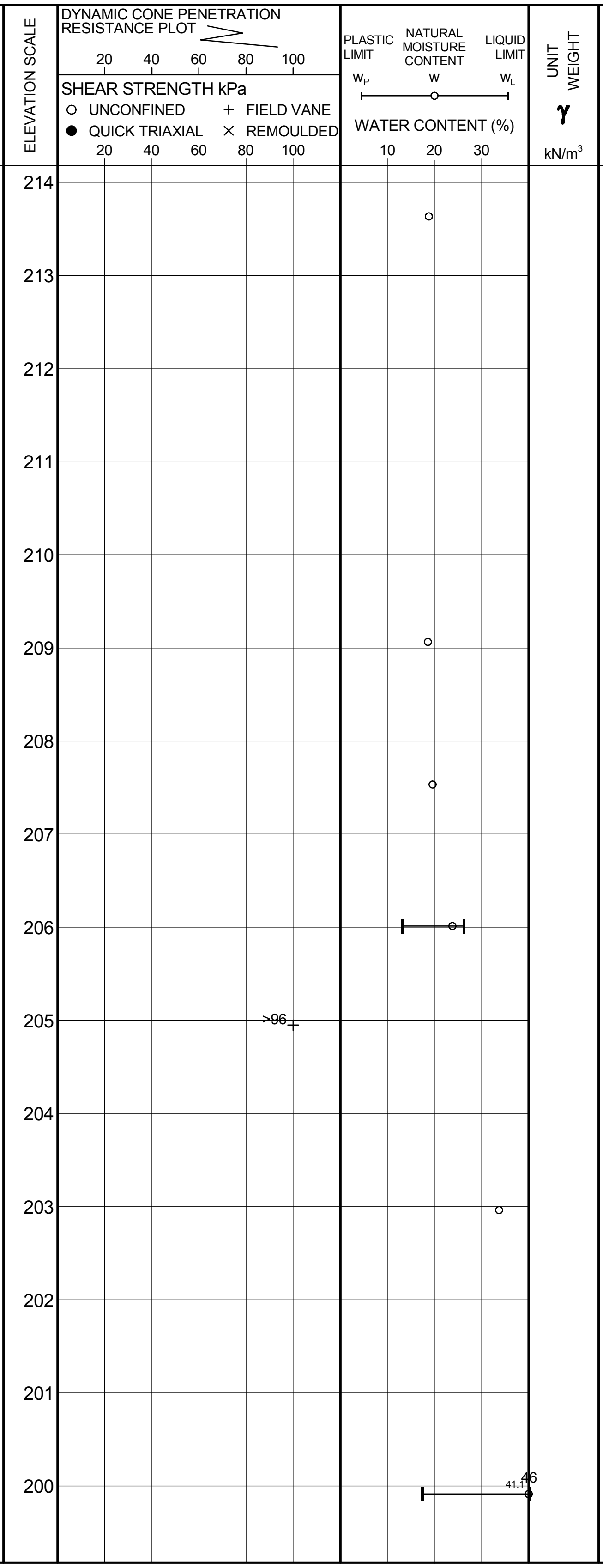
ORIGINATED BY DF

COMPILED BY DM

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
	SILT to Sandy SILT, trace clay with clayey silt pockets Compact to very dense Grey Wet	12	SS	27		0	10	87	3
		13	SS	30					
		14	SS	44					
		15	SS	39					
		16	SS	55					
206.8 22.4	Varved CLAYEY SILT, with silt and clay laminae Stiff to very stiff Grey Wet	17	SS	10		0	0	62	38
204.1 25.1	Varved SILTY CLAY, with silt and clay laminae Stiff to very stiff Grey Wet	18	SS	5					
		19	SS	1					



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

○ 3% STRAIN AT FAILURE

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RECORD OF BOREHOLE No 89UP-05

SHEET 3 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895649.6; E 292418.6 MTM NAD 83 ZONE 10 (LAT. 44.200750; LONG. -79.654912)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

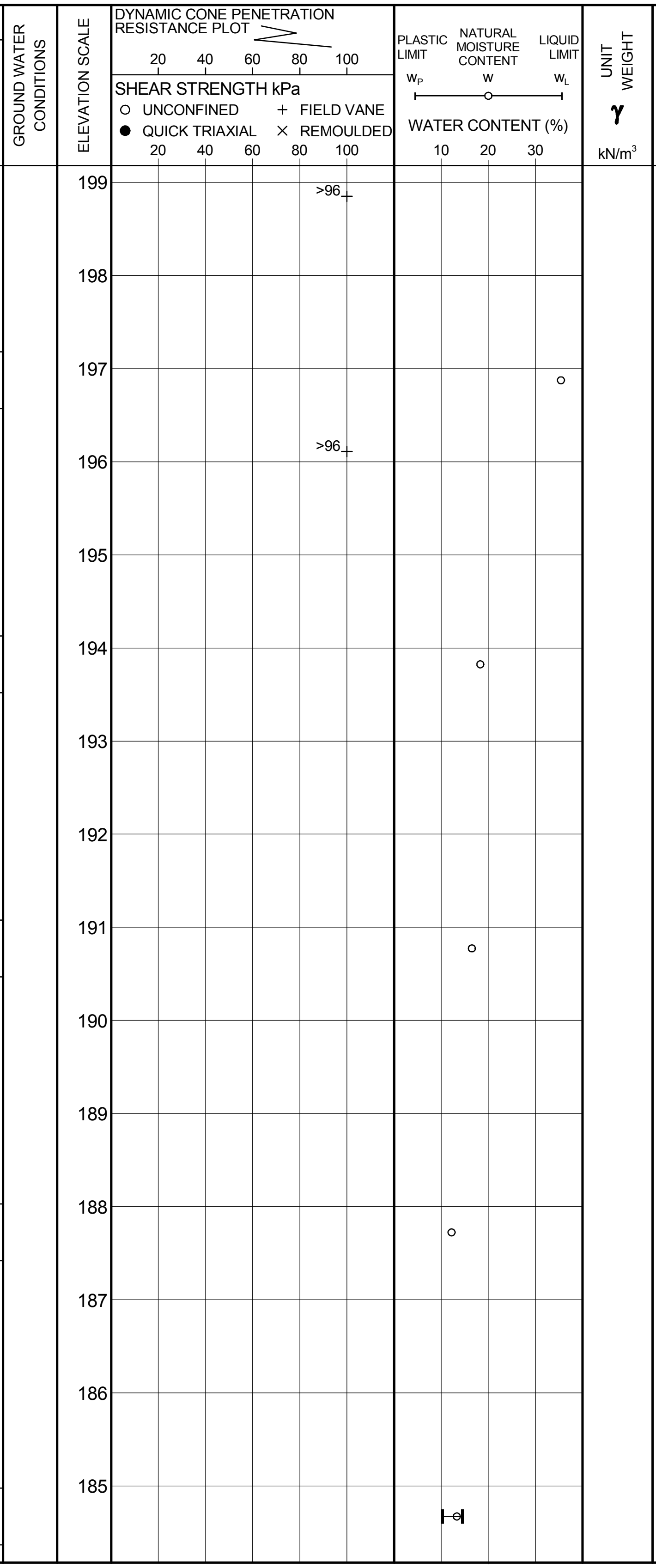
DATE June 26 to 29 and July 3, 2017

ORIGINATED BY DF

COMPILED BY DM

CHECKED BY SMM/TZ

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---							
	Varved SILTY CLAY, with silt and clay laminae Stiff to very stiff Grey Wet							
		20	SS	5				
195.0 34.2	SILT, trace to some sand, trace clay Very dense Grey Wet							
		21	SS	54	0	9	89	2
		22	SS	71				
189.3 39.9	CLAYEY SILT, trace to some sand Very stiff Grey Wet							
		23	SS	15				
186.2 43.0	Sandy CLAYEY SILT, some gravel (TILL) Hard Grey Wet							
		24	SS	35	14	24	48	14



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

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

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_HWY_89_INTERCHANGE\02_DATA\GINT\HWY_400_AND_HWY_89_INTERCHANGE.GPJ GAL-GTA.GDT 09/12/18

RECORD OF BOREHOLE No 89UP-05

SHEET 4 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

LOCATION N 4895649.6; E 292418.6 MTM NAD 83 ZONE 10 (LAT. 44.200750; LONG. -79.654912)

ORIGINATED BY DF

DIST Central HWY 400

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

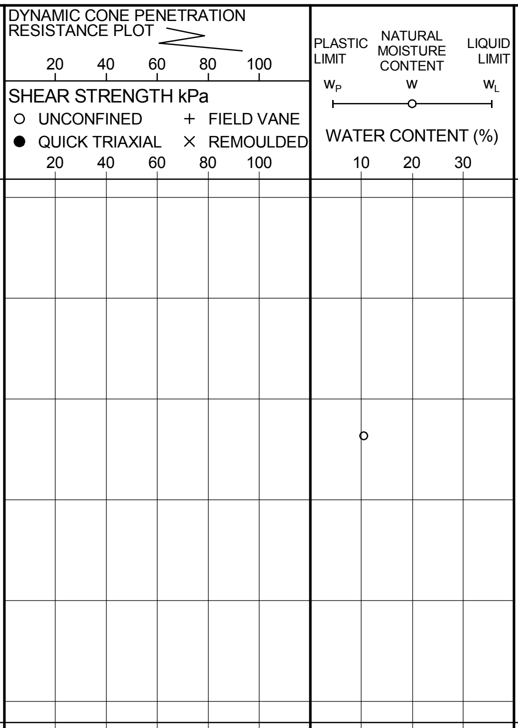
COMPILED BY DM

DATUM Geodetic

DATE June 26 to 29 and July 3, 2017

CHECKED BY SMM/TZ

SOIL PROFILE			SAMPLES						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	Sandy CLAYEY SILT, some gravel (TILL) Hard Grey Wet - Inferred cobbles/boulders encountered between depths of about 46.3 m and 46.9 m						184 183 182		
			25	SS	43		181 180 179		
178.8 50.4			26	SS	100/0.10				
	END OF BOREHOLE								



NOTE:

1. Water level measurements in the casing at the beginning of each work shift:

Date	Depth (m)	Elev. (m)
27/06/17	1.1	228.1
28/06/17	4.3	224.9
29/06/17	1.1	228.1
03/07/17	9.0	220.2

The water level measurements are not considered to be representative of the groundwater level due to introduction of water/drilling mud during wash boring operations.

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

RECORD OF BOREHOLE No 89UP-06

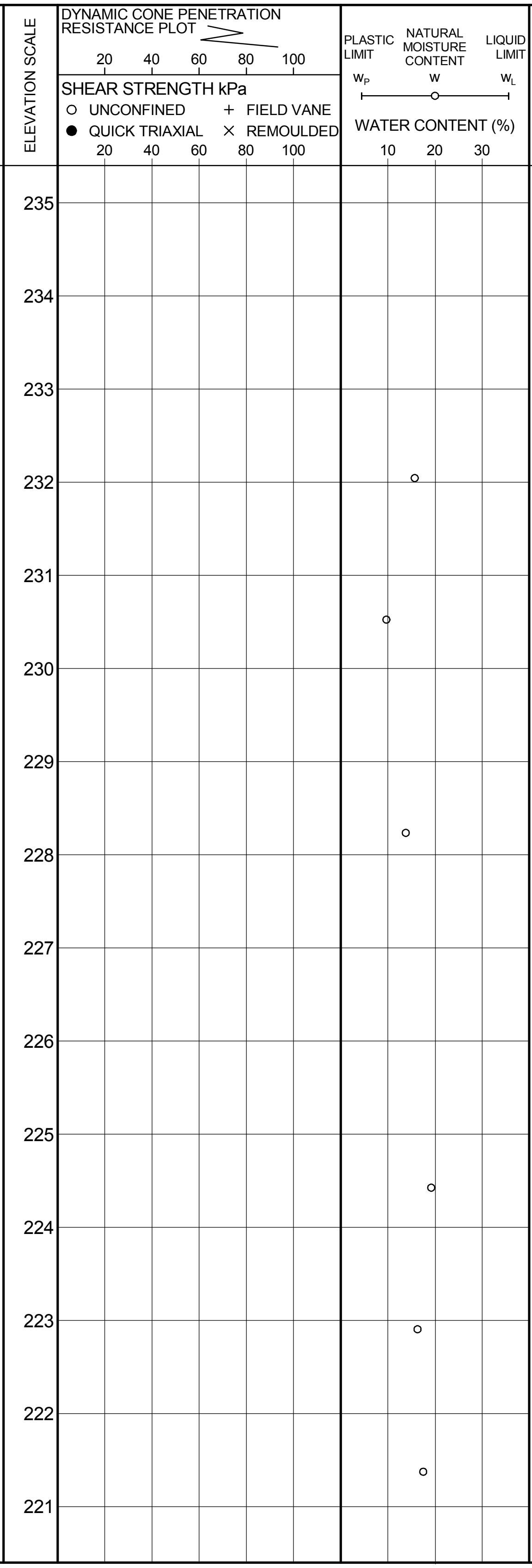
SHEET 1 OF 4

METRIC

PROJECT 1668512
G.W.P. 2438-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4895621.9; E 292469.4 MTM NAD 83 ZONE 10 (LAT. 44.200493; LONG. -79.654271)
BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud
DATE June 18 to 21, 2017
ORIGINATED BY DF
COMPILED BY DM
CHECKED BY SMM/TZ

ELEV DEPTH	SOIL PROFILE DESCRIPTION
235.4	GROUND SURFACE
0.0	ASPHALT (200 mm)
0.2	Sand and gravel (FILL) (430 mm)
234.8 0.6	Silt and sand, trace to some clay (FILL) Compact to very dense Brown Moist - Oxidation staining encountered below a depth of about 4.9 m - Clayey silt pockets encountered below a depth of about 7.2 m
226.3 9.1	SILT and SAND, trace clay Compact to dense Grey Wet - Grey below a depth of about 12.2 m

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
1	SS	14					
2	SS	34					
3	SS	33		0	52	45	3
4	SS	31					
5	SS	47					
6	SS	36					
7	SS	56					
8	SS	30		0	47	43	10
9	SS	20	∇				
10	SS	26					
11	SS	38					
12	SS	28		0	57	41	2
13	SS	26					
14	SS	25					



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No 89UP-06

SHEET 2 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895621.9; E 292469.4 MTM NAD 83 ZONE 10 (LAT. 44.200493; LONG. -79.654271)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE June 18 to 21, 2017

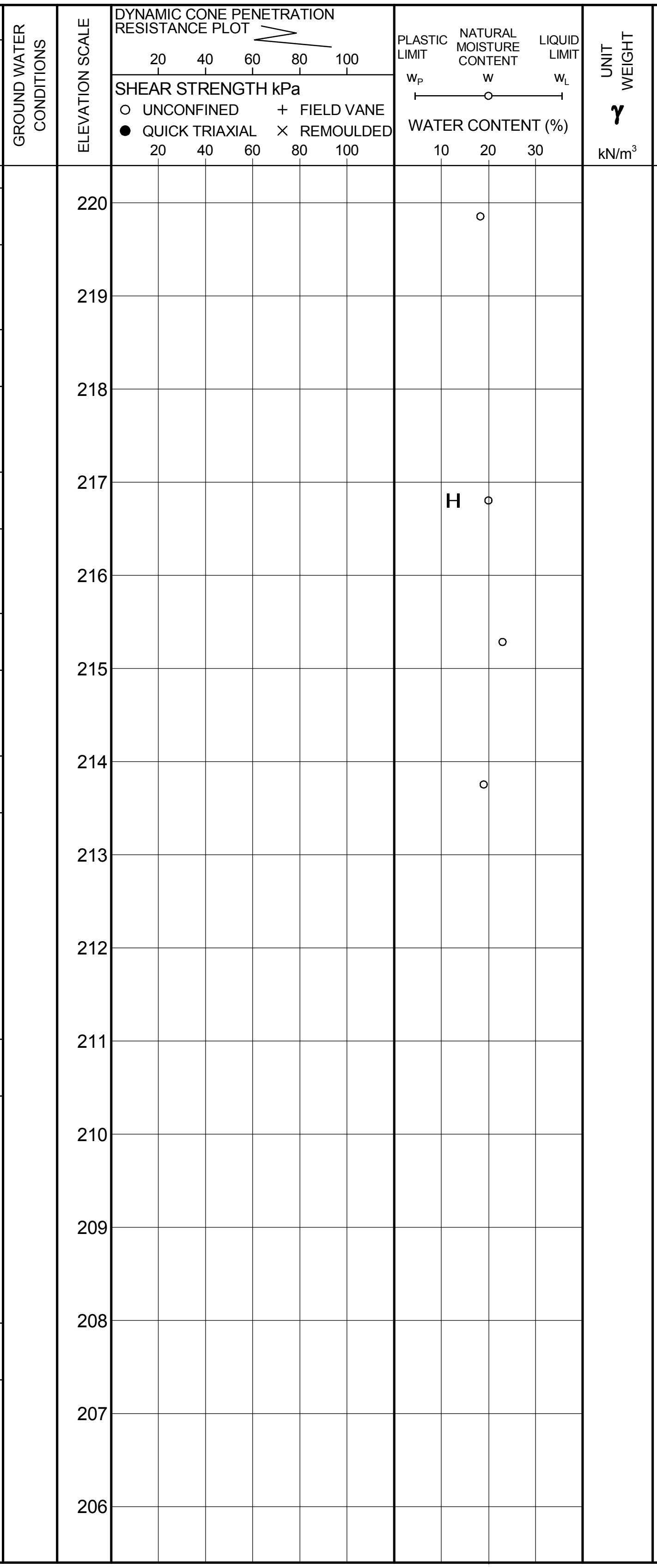
ORIGINATED BY DF

COMPILED BY DM

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	SILT and SAND, trace clay Compact to dense Grey Wet	15	SS	23	0	40	58	2
		16	SS	39				
217.9 17.5	Sandy SILT, trace to some clay Compact to dense Grey Wet	17	SS	23	0	29	59	12
		18	SS	24				
		19	SS	26	0	23	70	7
		20	SS	37				
		21	SS	43				
206.1 29.3								



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+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 89UP-06

SHEET 3 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

LOCATION N 4895621.9; E 292469.4 MTM NAD 83 ZONE 10 (LAT. 44.200493; LONG. -79.654271)

ORIGINATED BY DF

DIST Central HWY 400

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

COMPILED BY DM

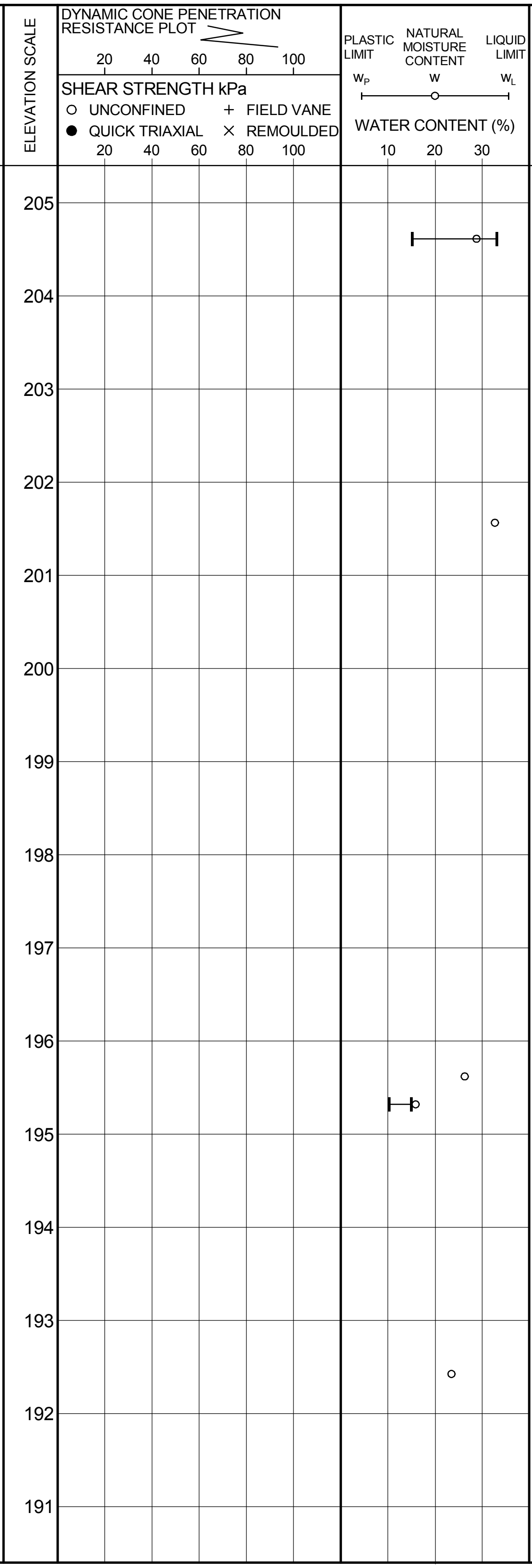
DATUM Geodetic

DATE June 18 to 21, 2017

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Varved CLAYEY SILT to SILTY CLAY with silt and clay laminae Stiff Grey Wet	22	SS	2		0	0	54	46
		23	SS	5					
		24	TO	PH	17.4 (C)				
195.5 39.9	Sandy CLAYEY SILT, trace to some gravel Firm Grey Wet	25A 25B	SS	6		6	29	51	14
193.9 41.5	SILT, trace sand Very dense Grey Wet	26	SS	65					
190.9 44.5									



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 89UP-06

SHEET 4 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895621.9; E 292469.4 MTM NAD 83 ZONE 10 (LAT. 44.200493; LONG. -79.654271)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE June 18 to 21, 2017

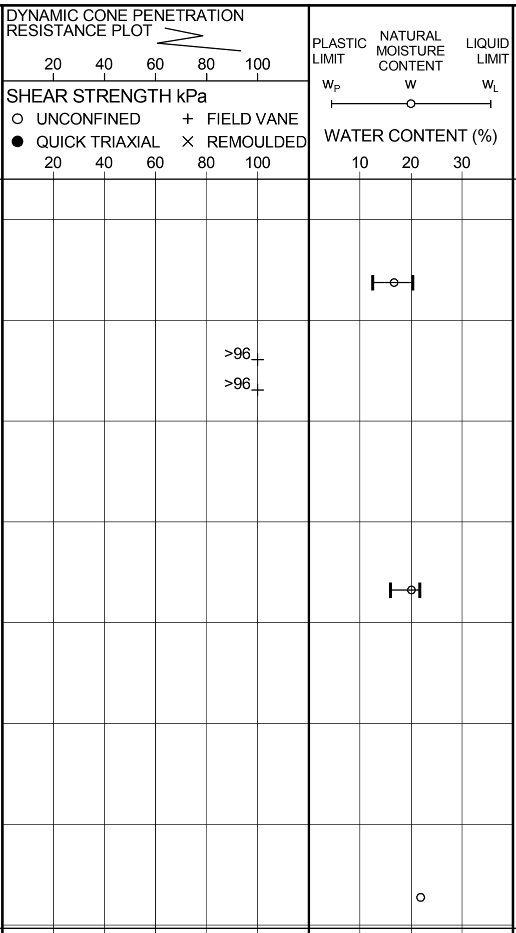
ORIGINATED BY DF

COMPILED BY DM

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FIELD VANE (kPa)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT Very stiff to hard Grey Moist				190		
		27	SS	WH			
					189	>96 >96	
					188		
					187		
		28	SS	77			0 0 67 23
					186		
					185		
					184		
183.0		29	SS	50			
52.4	END OF BOREHOLE				183		



UNIT WEIGHT γ kN/m³

NOTE:

1. Water level measurements in the casing at the beginning of each work shift:

Date	Depth (m)	Elev. (m)
19/06/17	2.9	232.5
20/06/17	1.6	233.8
21/06/17	8.1	227.3

The water level measurements are not considered to be representative of the groundwater level due to introduction of water/drilling mud during wash boring operations.

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

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

RECORD OF BOREHOLE No 89UP-07

SHEET 1 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895660.9; E 292451.0 MTM NAD 83 ZONE 10 (LAT. 44.200843; LONG. -79.654503)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 28 and 31 and August 1 and 2, 2017

ORIGINATED BY DM

COMPILED BY DH

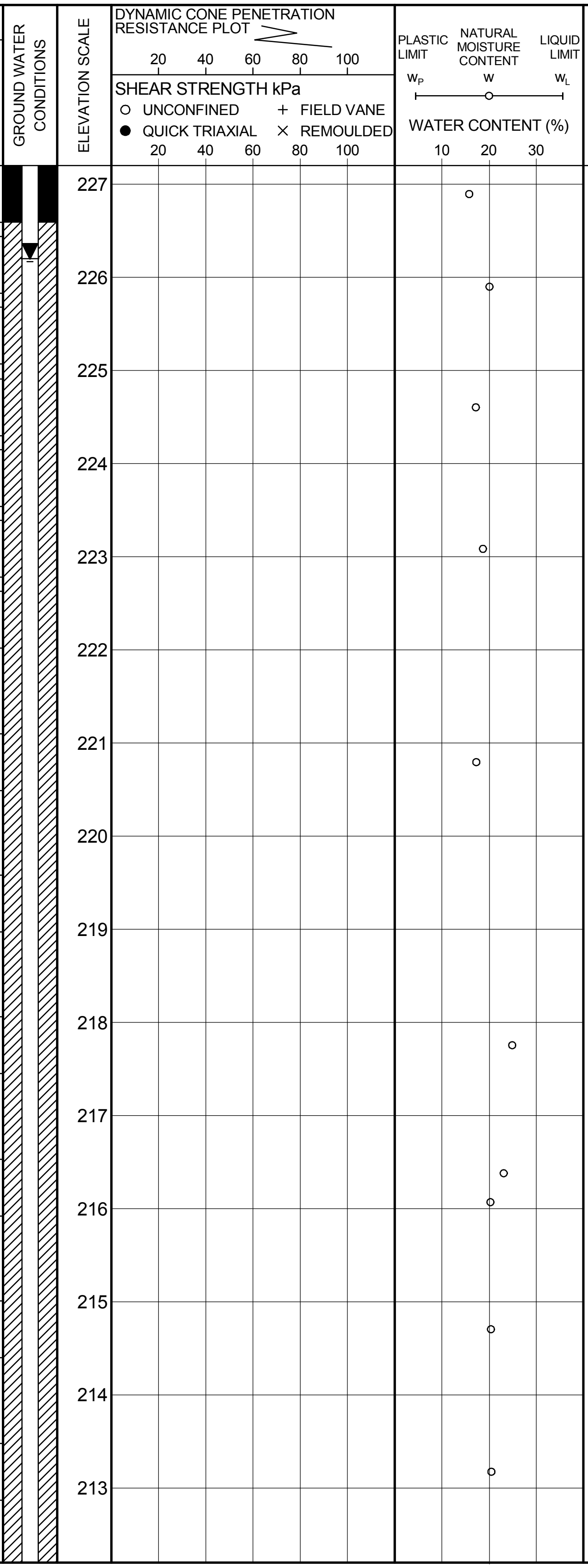
CHECKED BY SMM/TZ

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
227.2	GROUND SURFACE
0.0	TOPSOIL (460 mm) Loose
226.7	Silty sand, trace organics (FILL) Loose Brown Moist
0.7 226.0	Silty SAND Loose Grey to brown Wet
1.5	CLAYEY SILT, some sand Grey Wet
	SILT and SAND, trace clay Loose to compact Grey Wet
218.5 8.7	Varved SILTY CLAY, trace sand, with silt and clay laminae Stiff to very stiff Grey Moist
216.2 11.0	SILT, some sand, trace to some clay Compact to dense Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6	
2A 2B	SS	8	
3	SS	12	
4	SS	17	
5	SS	10	
6	SS	18	0 32 66 2
7	SS	9	
8	SS	17	
9	SS	15	
10	SS	8	
11A 11B	SS	23	
12	SS	11	0 16 77 7
13	SS	23	



UNIT WEIGHT γ kN/m³

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 89UP-07

SHEET 2 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895660.9; E 292451.0 MTM NAD 83 ZONE 10 (LAT. 44.200843; LONG. -79.654503)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 28 and 31 and August 1 and 2, 2017

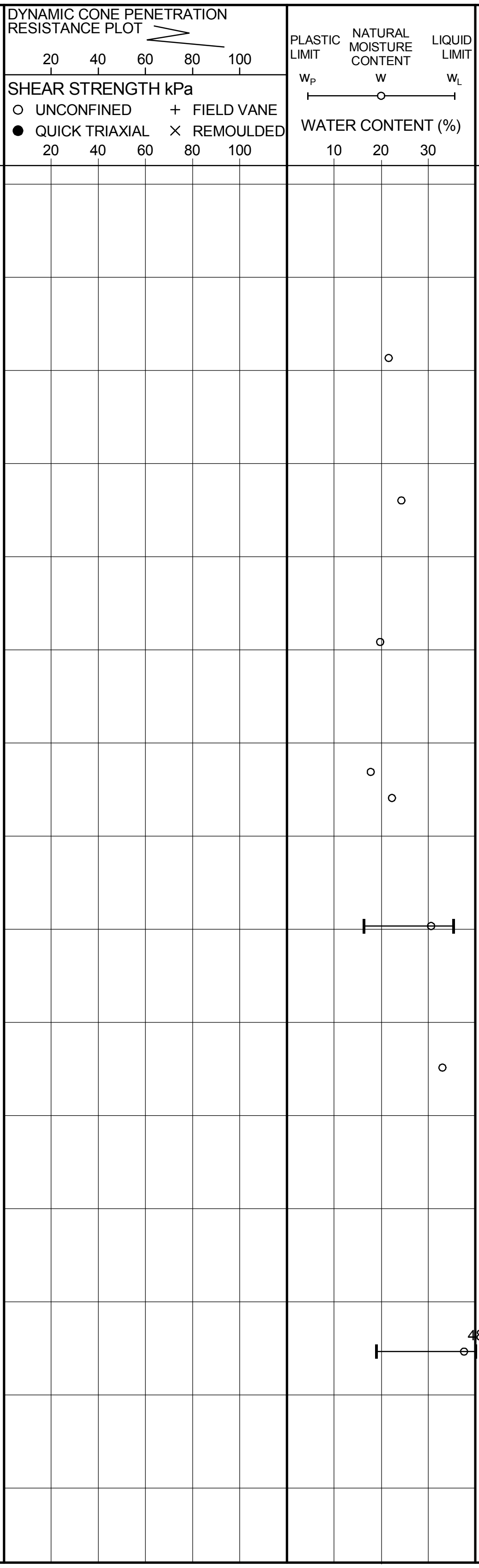
ORIGINATED BY DM

COMPILED BY DH

CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	SILT, some sand, trace to some clay Compact to dense Grey Wet	14	SS	26	212					
		15	SS	43	211 210					
		16	SS	46	209					
		17	SS	36	208 207		0	4	89	7
		18A 18B	SS	24	206					
205.2 22.0	Varved SILTY CLAY, with silt and clay laminae Soft to stiff Grey Moist	19	SS	7	205 204					
		20	SS	9	203 202					
		21	TO	PH	201	18.8 (C)				
		22	SS	5	200 199					
		23	TO	PH	198					



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+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_HWY_89_INTERCHANGE\02_DATA\GINT\HWY_400_AND_HWY_89_INTERCHANGE.GPJ GAL-GTA.GDT 09/12/18

Golder Associates

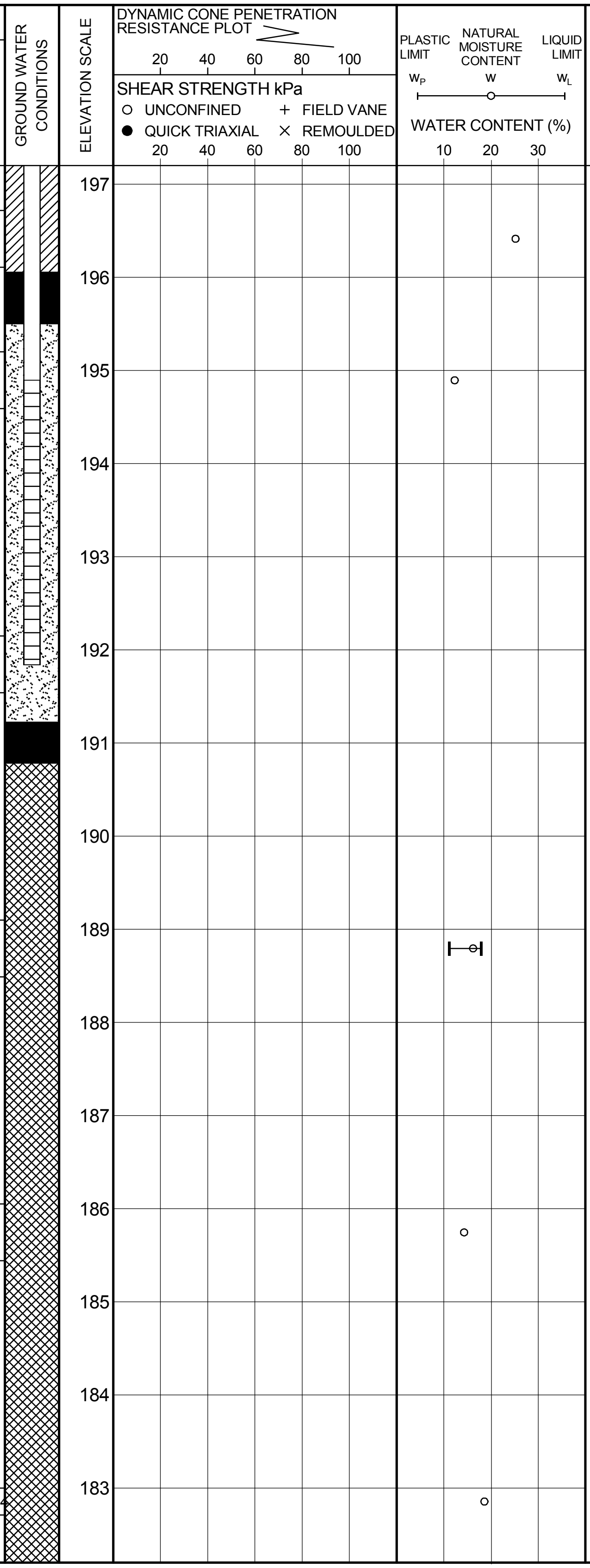
RECORD OF BOREHOLE No 89UP-07

SHEET 3 OF 4

METRIC

PROJECT 1668512
G.W.P. 2438-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4895660.9; E 292451.0 MTM NAD 83 ZONE 10 (LAT. 44.200843; LONG. -79.654503)
BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud
DATE July 28 and 31 and August 1 and 2, 2017
ORIGINATED BY DM
COMPILED BY DH
CHECKED BY SMM/TZ

SOIL PROFILE		SAMPLES			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
	--- *CONTINUED FROM PREVIOUS PAGE* ---								
	Varved SILTY CLAY, with silt and clay laminae Soft to stiff Grey Moist	24	SS	7					
195.6 31.6	SILT and SAND, some gravel, trace clay Very dense Grey Wet	25	SS	51		20	38	39	3
		26	SS	69					
190.3 36.9	Sandy CLAYEY SILT, trace gravel Very stiff Grey Moist	27	SS	17		4	22	56	18
		28	SS	25					
184.2 43.0	SILT and SAND, trace clay (TILL) Very dense Grey Wet	29	SS	100/0.14		0	40	58	2



Continued Next Page

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

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

RECORD OF BOREHOLE No 89UP-07

SHEET 4 OF 4

METRIC

PROJECT 1668512

G.W.P. 2438-13-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4895660.9; E 292451.0 MTM NAD 83 ZONE 10 (LAT. 44.200843; LONG. -79.654503)

BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud

DATE July 28 and 31 and August 1 and 2, 2017

ORIGINATED BY DM

COMPILED BY DH

CHECKED BY SMM/TZ

SOIL PROFILE		SAMPLES			ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	SILT and SAND, trace clay (TILL) Very dense Grey Wet				182				
					181				
		30	SS	100/0.05	180				
					179				
178.4 48.8	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist				178				
					177				
176.6		31	SS	100/0.10		1	31	44	24
50.6	END OF BOREHOLE								

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m^3

NOTE:

1. Water level measurements in the casing at the begining of each work shift:

Date	Depth (m)	Elev. (m)
31/07/17	6.2	221.0
01/08/17	5.9	221.3
02/08/17	7.9	219.3

2. Water level measurements in standpipe piezometer:

Date	Depth (m)	Elev. (m)
10/08/17	0.7	226.5
15/08/17	0.7	226.5
19/08/17	0.9	226.3
05/03/18	1.0	226.2

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

RECORD OF BOREHOLE No 89UP-08

SHEET 1 OF 1

METRIC

PROJECT 1668512			
G.W.P. 2438-13-00	LOCATION N 4895655.5; E 292478.1 MTM NAD 83 ZONE 10 (LAT. 44.200795; LONG. -79.654165)	ORIGINATED BY DF
DIST Central HWY 400	BOREHOLE TYPE D50 Track Mount, NW Casing and Wash Boring with Drilling Mud	COMPILED BY DH
DATUM Geodetic	DATE August 9, 2017	CHECKED BY SMM/TZ

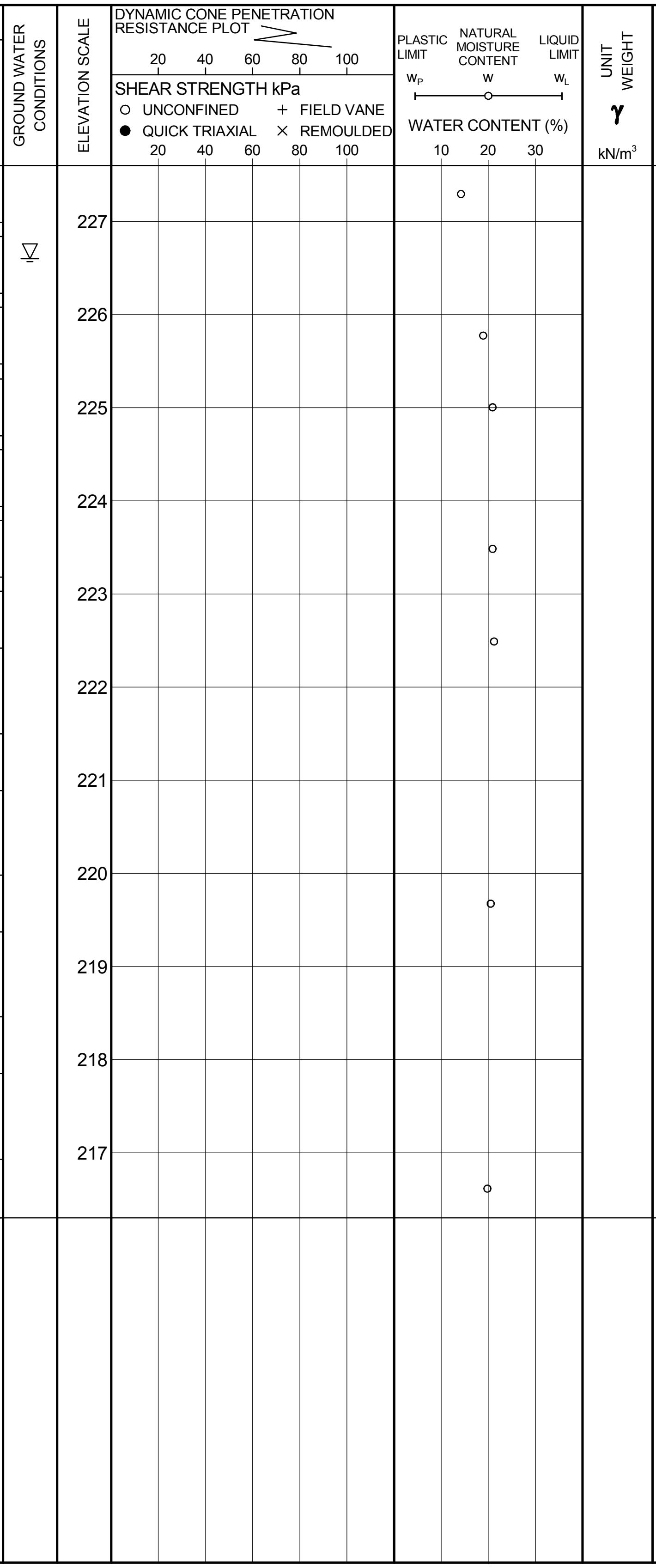
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
227.6	GROUND SURFACE
0.0	TOPSOIL (250 mm)
0.3	Silt and sand to silty sand to sand and silt, trace clay (FILL) Loose to compact Brown mottled with oxidation staining Moist
224.6 3.0	SILT and SAND Compact Brown Wet
222.6 5.0	SILTY CLAY Stiff Grey Moist
222.0 5.6	SILT and SAND, trace to some clay Compact Grey Wet
216.3 11.3	END OF BOREHOLE NOTE: 1. Water level measured in open borehole at a depth of about 1.1 m (Elev. 226.5 m) below ground surface upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	6				
2A 2B	SS	14				
3	SS	15	0	64	33	3
4	SS	19				
5	SS	15				
6	SS	13				
7A 7B	SS	15				
8	SS	12				
9	SS	15	0	50	44	6
10	SS	19				
11	SS	19	0	32	64	4

GROUND WATER CONDITIONS ∇



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

RECORD OF BOREHOLE No HF-02

SHEET 1 OF 3

METRIC

PROJECT 1668512			
G.W.P. 2438-13-00	LOCATION N 4895665.0; E 292504.8 MTM NAD 83 ZONE 10 (LAT. 44.200881; LONG. -79.653830)	ORIGINATED BY	DF
DIST Central HWY 400	BOREHOLE TYPE D50 Track-Mounted, NW Casing and Wash Boring with Drilling Mud	COMPILED BY	DH
DATUM Geodetic	DATE August 4, 8 and 9, 2017	CHECKED BY	SMM/TZ

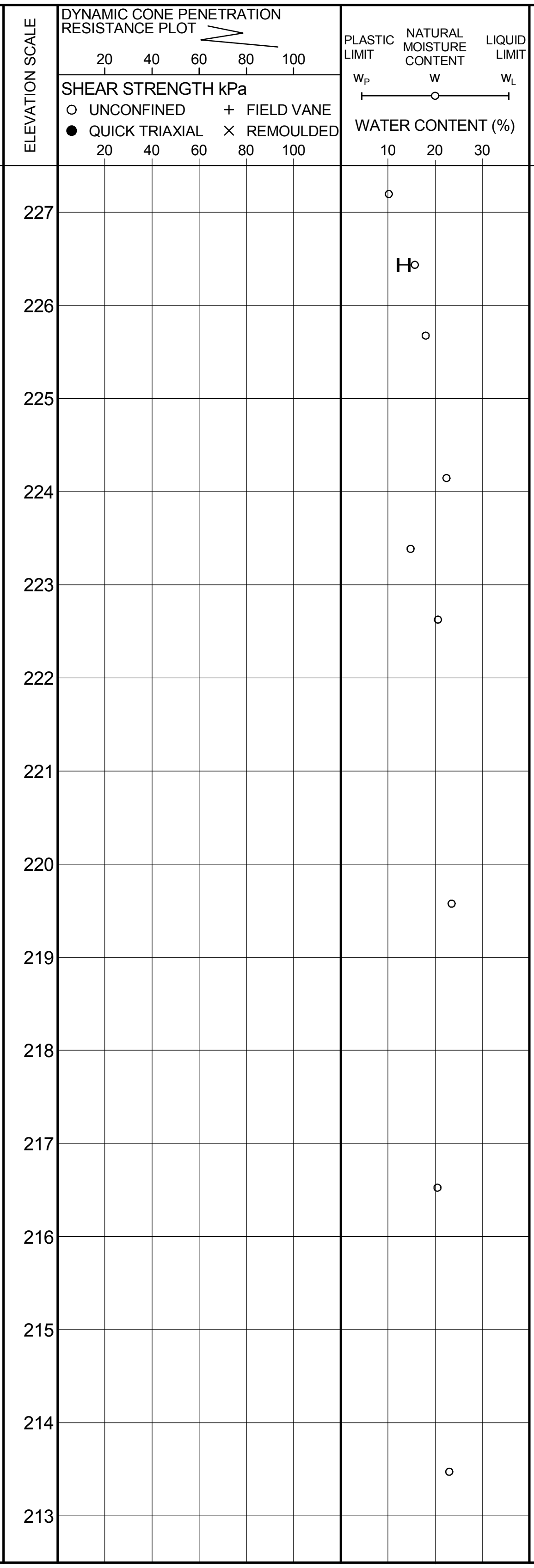
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
227.5	GROUND SURFACE
0.0	TOPSOIL (230 mm)
0.2	Silty SAND, trace rootlets Loose Brown Moist
226.8 0.7	SILT and SAND, trace to some clay Compact Brown Moist
226.0 1.5	Silty SAND, trace clay Loose to compact Grey Wet
221.1 6.4	SILT, trace to some sand to SILT and SAND, trace clay Loose to very dense Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	9					
2	SS	10		0	35	58	7
3	SS	13					
4	SS	8	▽				
5	SS	22					
6	SS	19		0	70	28	2
7	SS	21					
8	SS	30					
9	SS	9		0	36	63	1
10	SS	21					
11	SS	13					
12	SS	16					
13	SS	14					

STRAT PLOT — UNIT WEIGHT γ kN/m³



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

RECORD OF BOREHOLE No HF-02

SHEET 2 OF 3

METRIC

PROJECT 1668512
G.W.P. 2438-13-00
LOCATION N 4895665.0; E 292504.8 MTM NAD 83 ZONE 10 (LAT. 44.200881; LONG. -79.653830)
ORIGINATED BY DF
DIST Central HWY 400
BOREHOLE TYPE D50 Track-Mounted, NW Casing and Wash Boring with Drilling Mud
COMPILED BY DH
DATUM Geodetic
DATE August 4, 8 and 9, 2017
CHECKED BY SMM/TZ

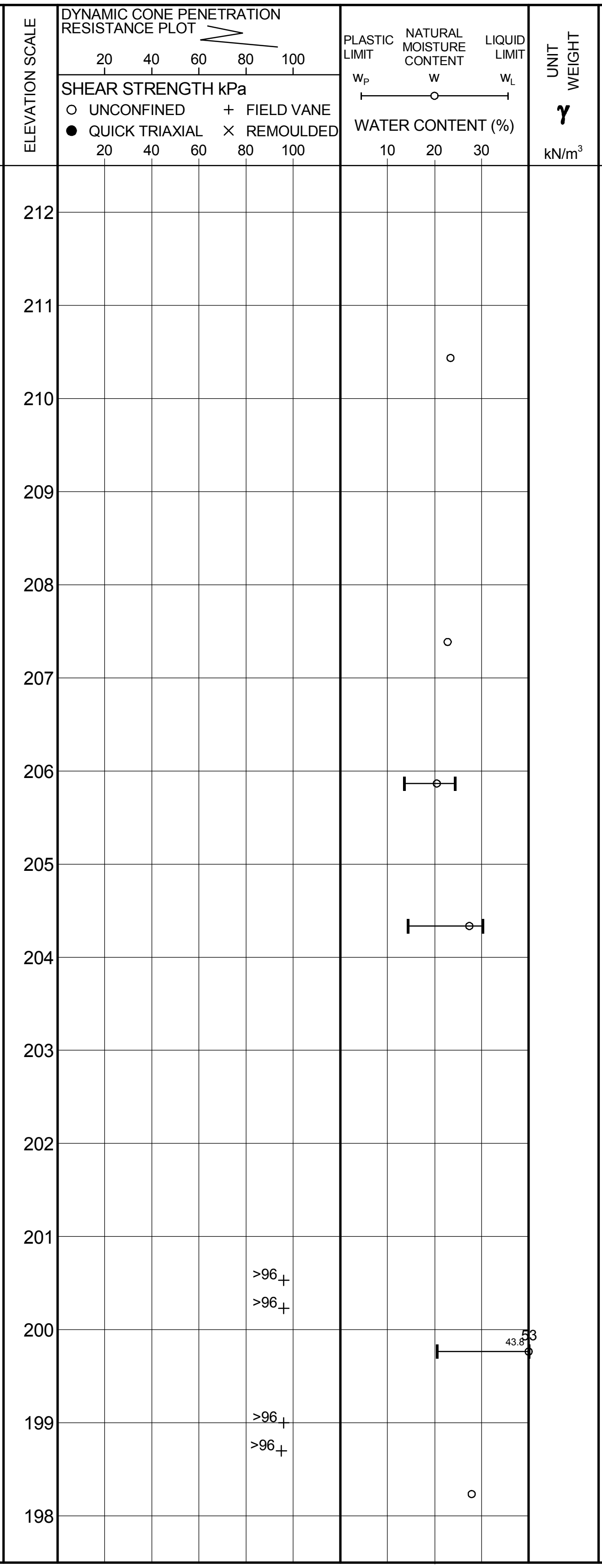
SOIL PROFILE

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION
	SILT, trace to some sand to SILT and SAND, trace clay Loose to very dense Grey Wet
206.6 20.9	Varved CLAYEY SILT to CLAY, with silt and clay laminae Stiff to very stiff Grey Moist - Sand inclusions encountered between depths of about 25.9 m (Elev. 201.6 m) and 26.4 m (Elev. 201.1 m)

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
14	SS	27				
15	SS	37	0	8	88	4
16	SS	34				
17	SS	53				
18	SS	21				
19	SS	6				
20	TO	PH				
21	TO	PH				
22	SS	4				
23	SS	8				



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

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

RECORD OF BOREHOLE No HF-02

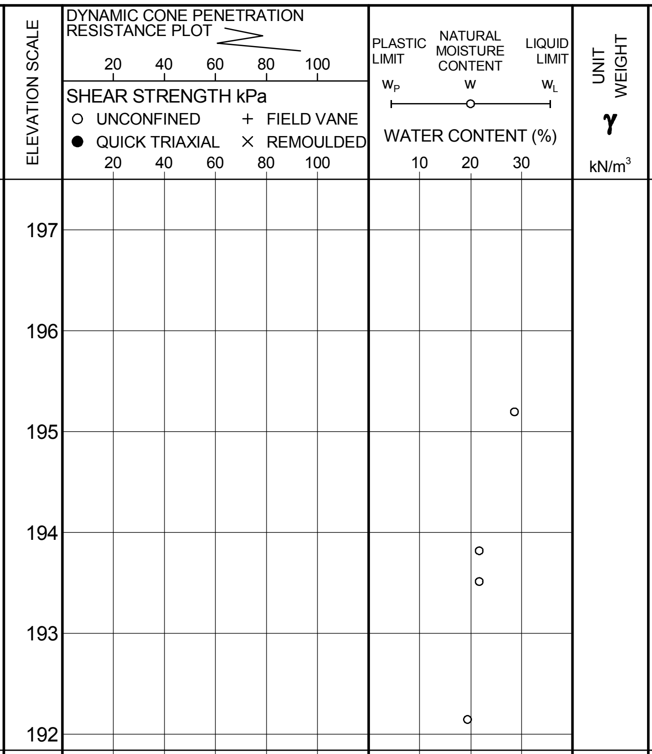
SHEET 3 OF 3

METRIC

PROJECT 1668512
G.W.P. 2438-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4895665.0; E 292504.8 MTM NAD 83 ZONE 10 (LAT. 44.200881; LONG. -79.653830)
BOREHOLE TYPE D50 Track-Mounted, NW Casing and Wash Boring with Drilling Mud
DATE August 4, 8 and 9, 2017
ORIGINATED BY DF
COMPILED BY DH
CHECKED BY SMM/TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	Varved CLAYEY SILT to CLAY, with silt and clay laminae Stiff to very stiff Grey Moist	24	SS	11					
		25	SS	12					
194.4 33.1	Silty SAND, trace clay Very dense Grey Wet	26A 26B	SS	67					
		27	SS	73		0	75	23	2
191.8 35.7	END OF BOREHOLE								



NOTE:

1. Water level measurements in the casing at the beginning of each work shift:

Date	Depth (m)	Elev. (m)
08/08/17	1.3	226.2
09/08/17	2.7	224.8

The water level measurements are not considered to be representative of the groundwater level due to introduction of water/drilling mud during wash boring operations.

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

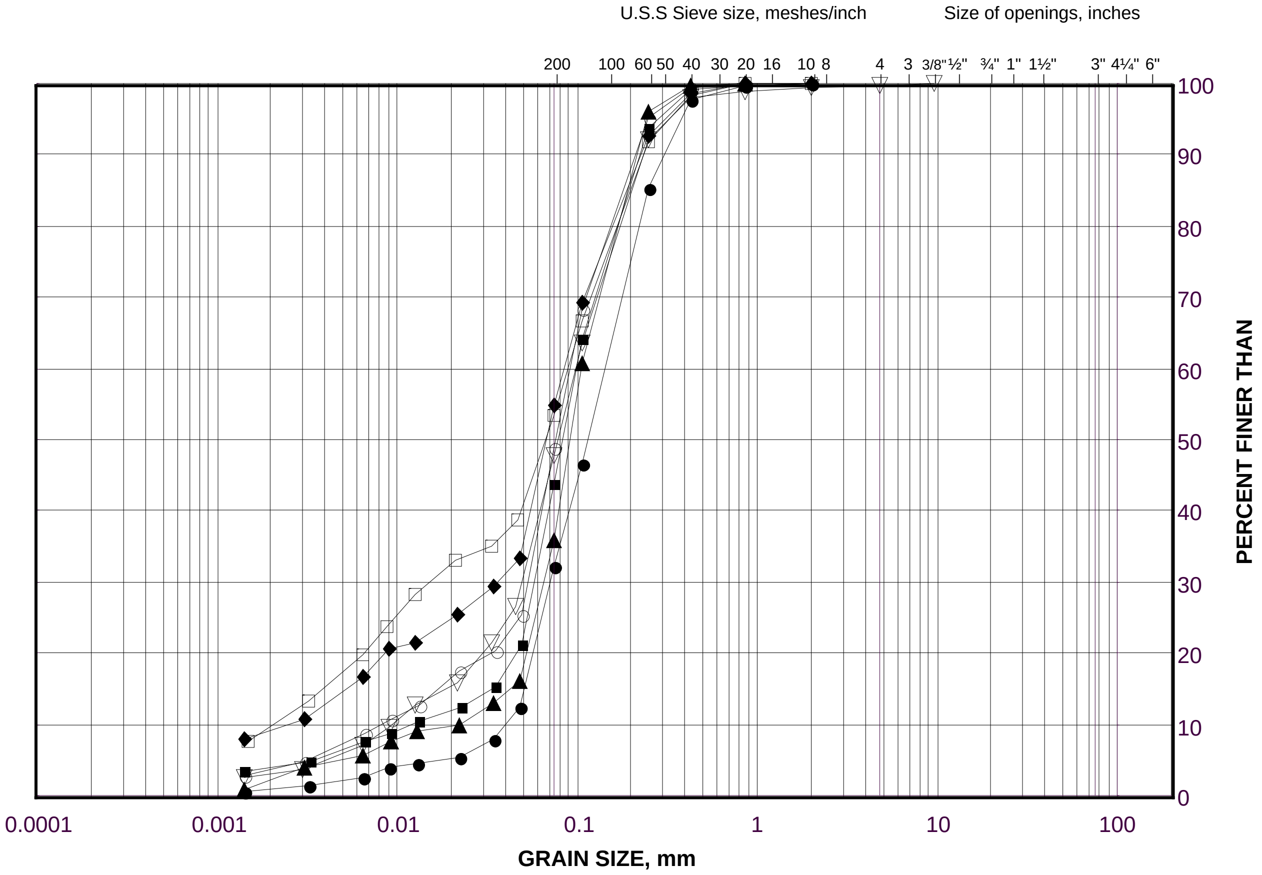
$\bigcirc^{3\%}$ STRAIN AT FAILURE

APPENDIX C

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION
Silt and Sand (FILL)

FIGURE C-1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-04	1	227.5
■	89UP-02	2	232.8
◆	89UP-05	2	226.6
▲	89UP-08	3	225.8
▽	89UP-06	3	232.0
❍	89UP-02	6	229.0
□	89UP-06	8	228.2

Project Number: 1668512

Checked By: SMM

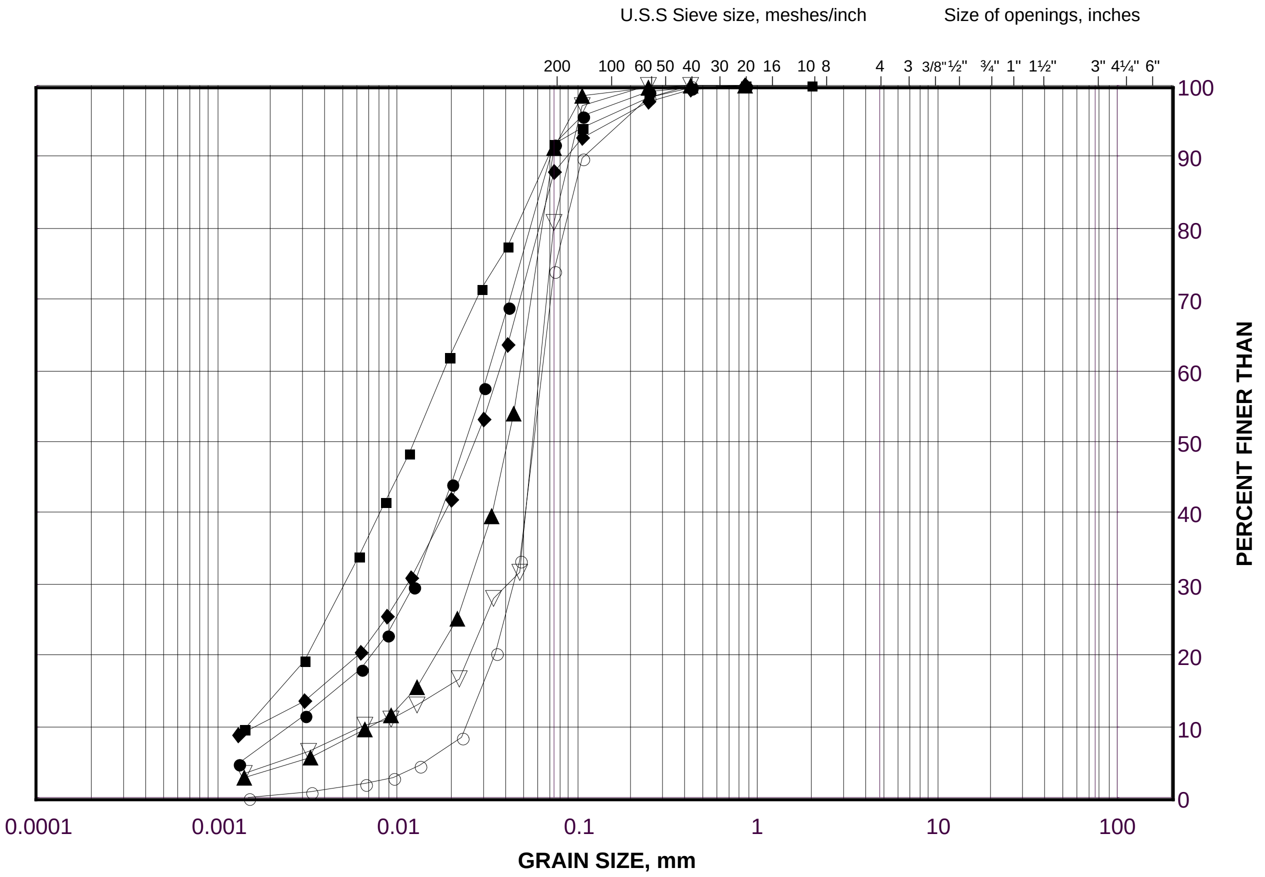
Golder Associates

Date: 05-Oct-17

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt (Upper Granular Deposit)

FIGURE C-2A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-04	11	215.3
■	89UP-02	11B	221.2
◆	89UP-03	13	213.4
▲	89UP-02	17	212.2
▽	89UP-02	20	207.7
❍	89UP-05	8	219.8

Project Number: 1668512

Checked By: SMM

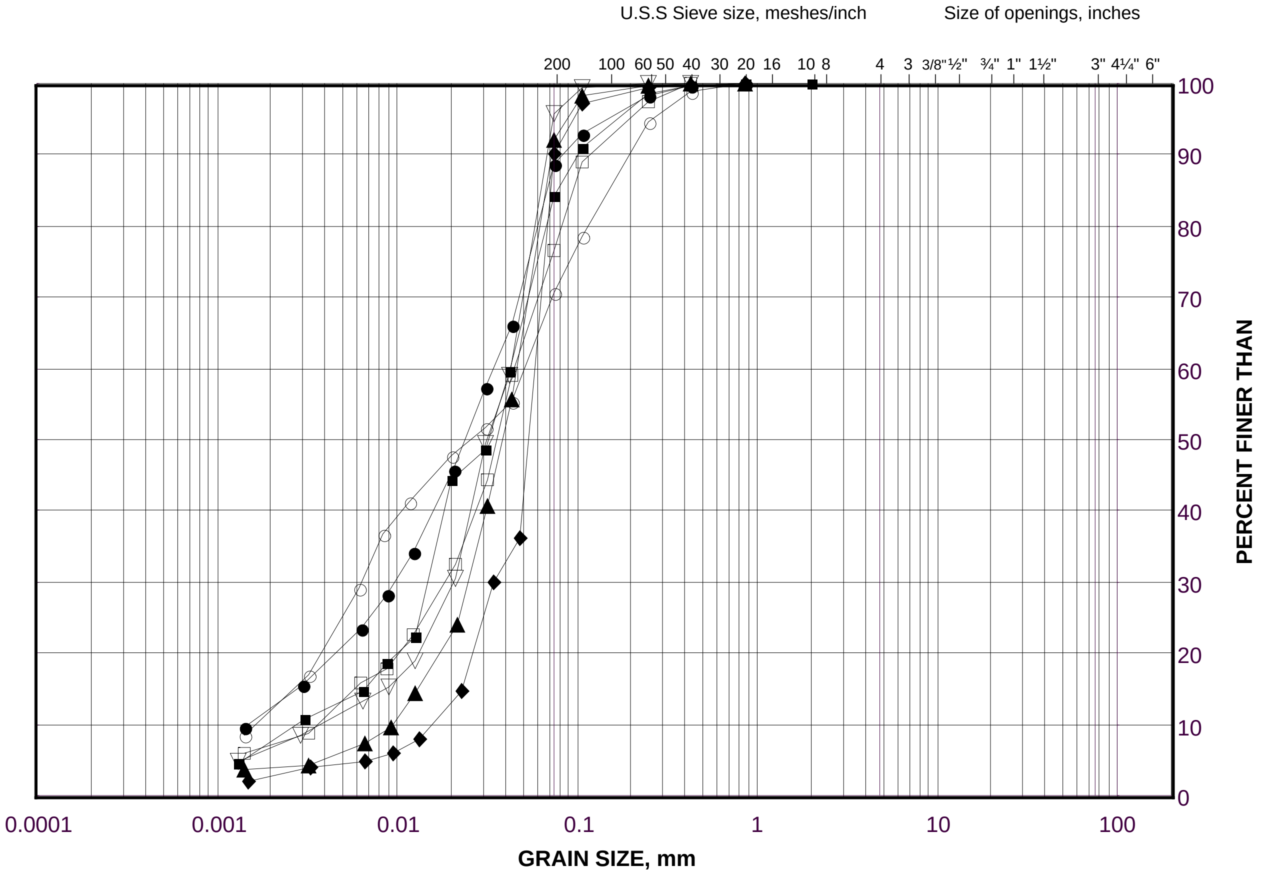
Golder Associates

Date: 01-Dec-17

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt (Upper Granular Deposit)

FIGURE C-2B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-05	11	215.1
■	89UP-07	12	214.7
◆	89UP-05	12	213.7
▲	HF-02	15	210.4
▽	89UP-07	17	207.1
❍	89UP-06	17	216.8
□	89UP-06	19	213.8

Project Number: 1668512

Checked By: SMM

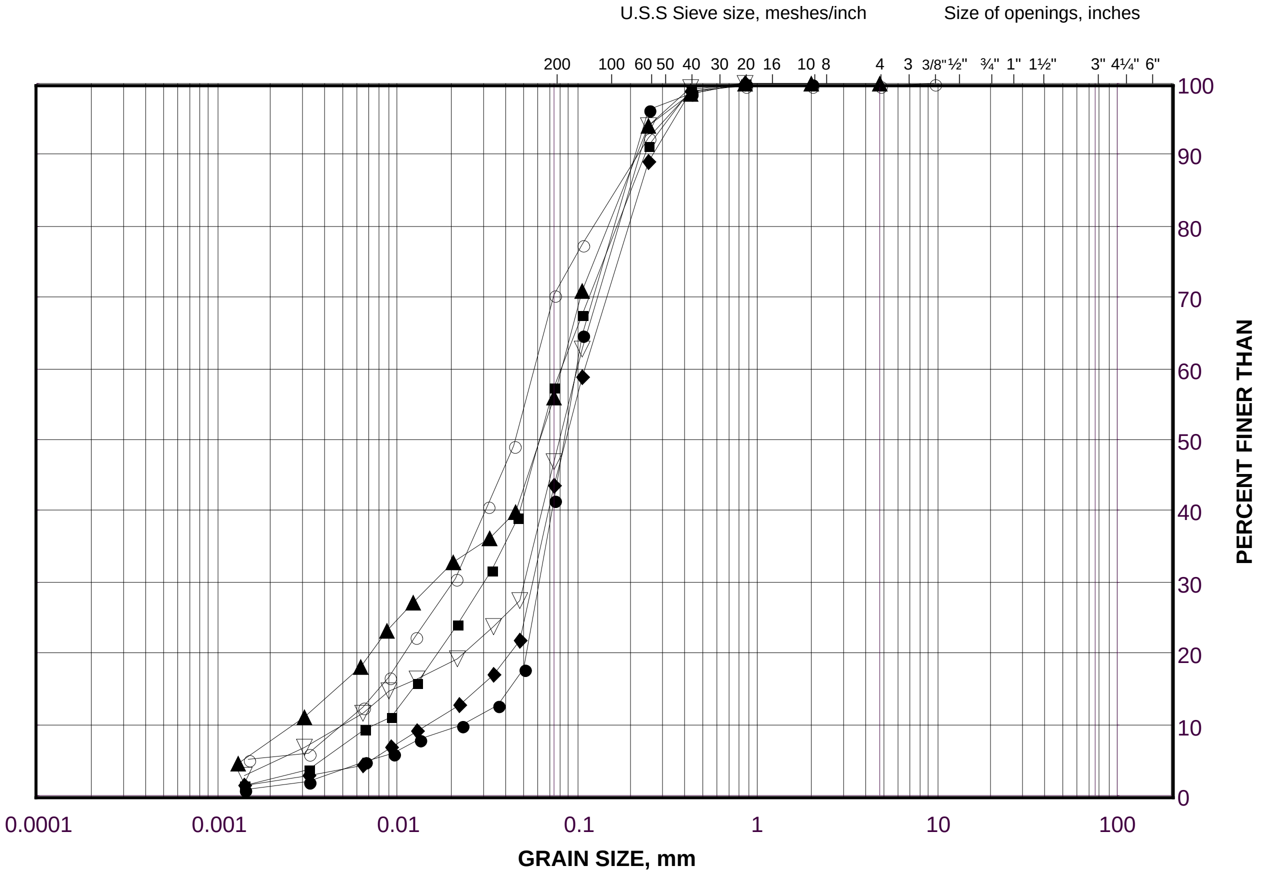
Golder Associates

Date: 25-Oct-17

GRAIN SIZE DISTRIBUTION

Silt and Sand (Upper Granular Deposit)

FIGURE C-3A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-02	10B	222.9
■	89UP-02	12B	219.8
◆	89UP-01	5	224.4
▲	89UP-03	5	224.0
▽	89UP-04	6	222.9
❍	89UP-01	9	219.9

Project Number: 1668512

Checked By: SMM

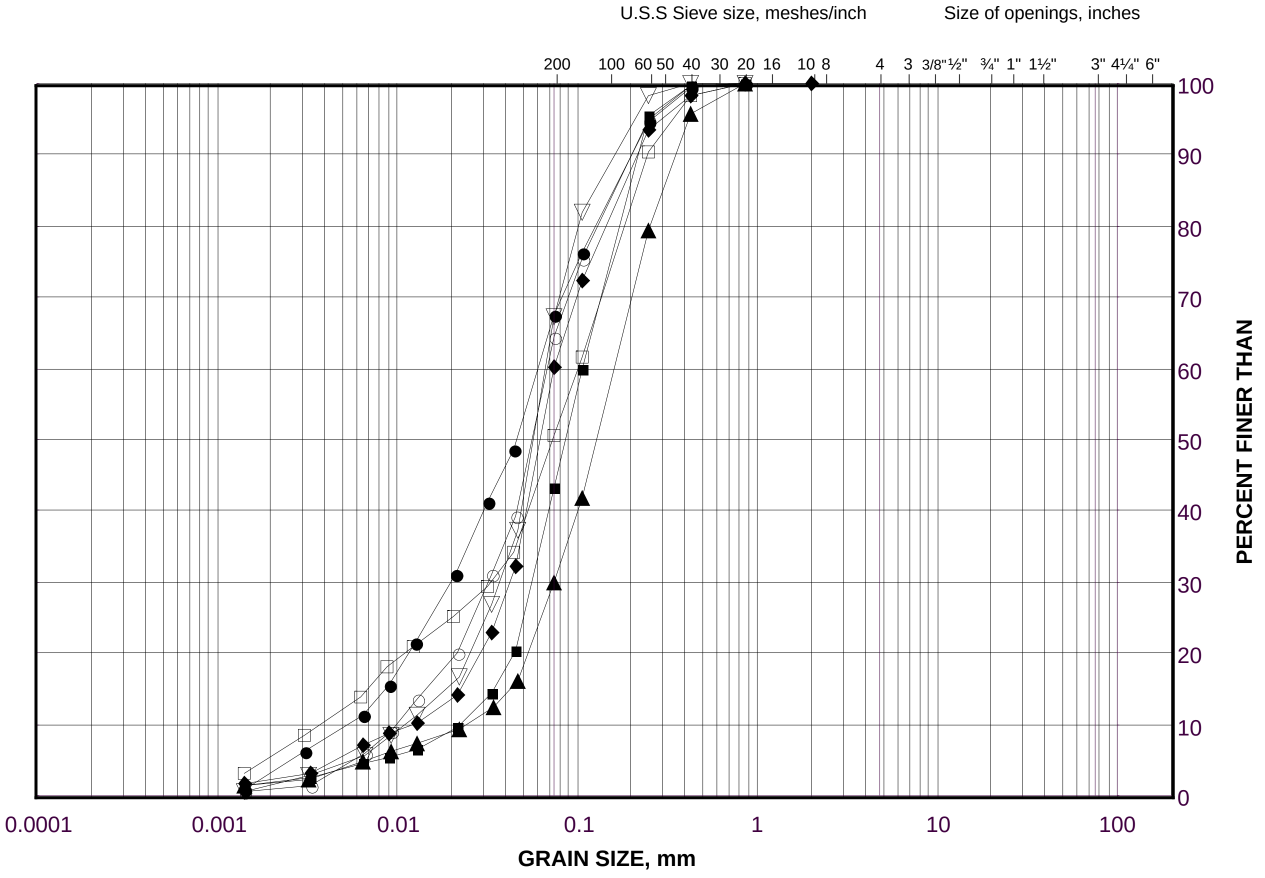
Golder Associates

Date: 25-Oct-17

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand (Upper Granular Deposit)

FIGURE C-3B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-08	11	216.6
■	89UP-06	12	224.4
◆	89UP-06	15	219.9
▲	HF-02	6	223.4
▽	89UP-07	6	223.1
❍	HF-02	9	219.6
□	89UP-08	9	219.7

Project Number: 1668512

Checked By: SMM

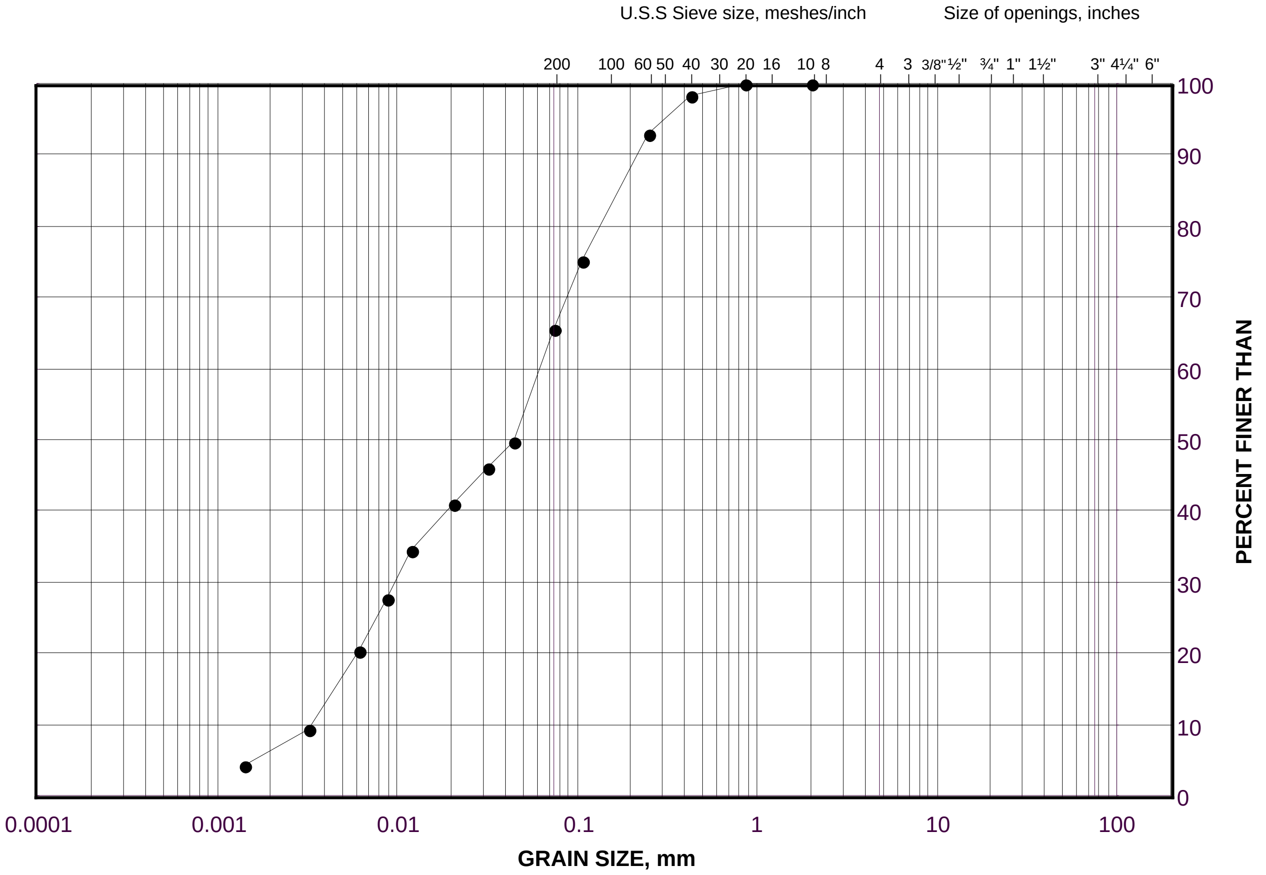
Golder Associates

Date: 25-Oct-17

GRAIN SIZE DISTRIBUTION

Silt and Sand (Upper Granular Deposit)

FIGURE C-3C



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-02	2	226.4

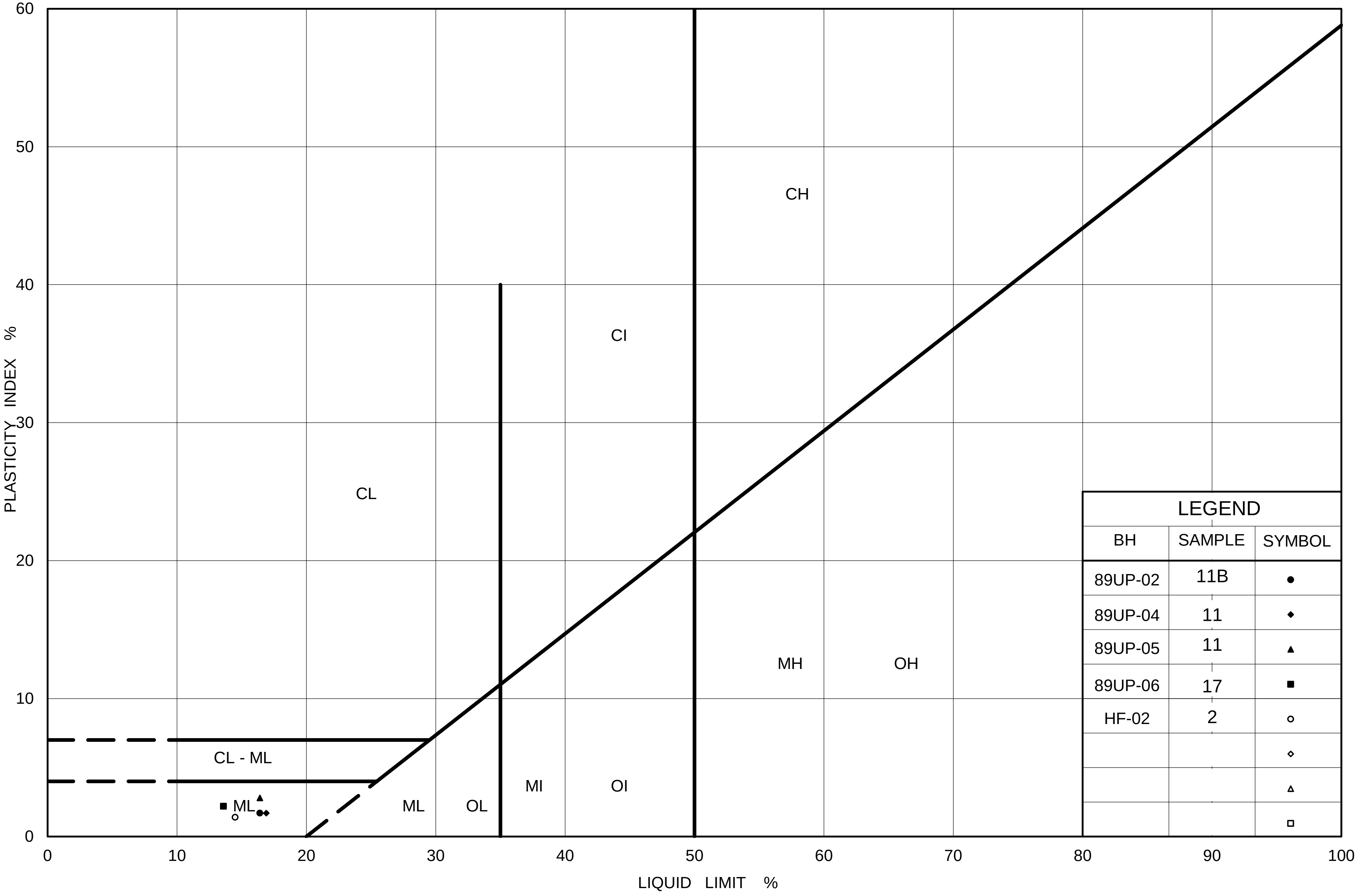
Project Number: 1668512

Checked By: SMM

Golder Associates

Date: 26-Apr-18

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
89UP-02	11B	●
89UP-04	11	◆
89UP-05	11	▲
89UP-06	17	■
HF-02	2	○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART

Silt to Silt and Sand (Upper Granular Deposit) (Slight Plasticity)

Figure No. C-4

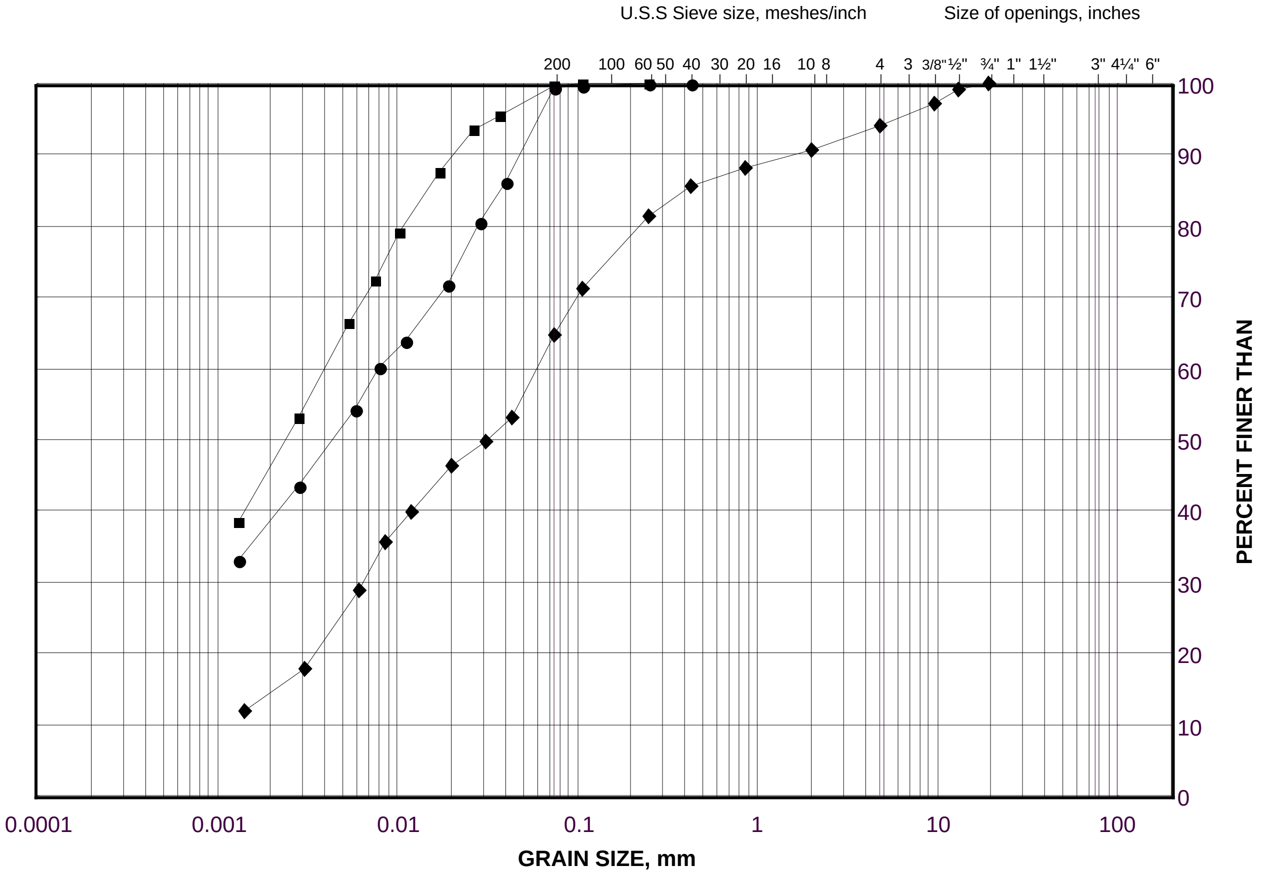
Project No. 1668512

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt (Upper Cohesive Deposit)

FIGURE C-5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-05	17	206.0
■	89UP-06	22	204.6
◆	89UP-06	25B	195.3

Project Number: 1668512

Checked By: SMM

Golder Associates

Date: 01-Dec-17

Highway 400 / 89 Underpass Replacement Structure
Upper Cohesive Deposit – Varved Soil Matrix

Figure C-6

Photograph 1: Soil Sample from Borehole 89UP-03 Sample 20 (Location of Proposed West Abutment)

Photograph 2: Soil Sample from Borehole 89UP-07 Sample 21 (Location of Proposed East Abutment)

Photograph 3: Soil Sample from Borehole 89UP-06 Sample 24 (Location of Proposed East Abutment)

Notes:

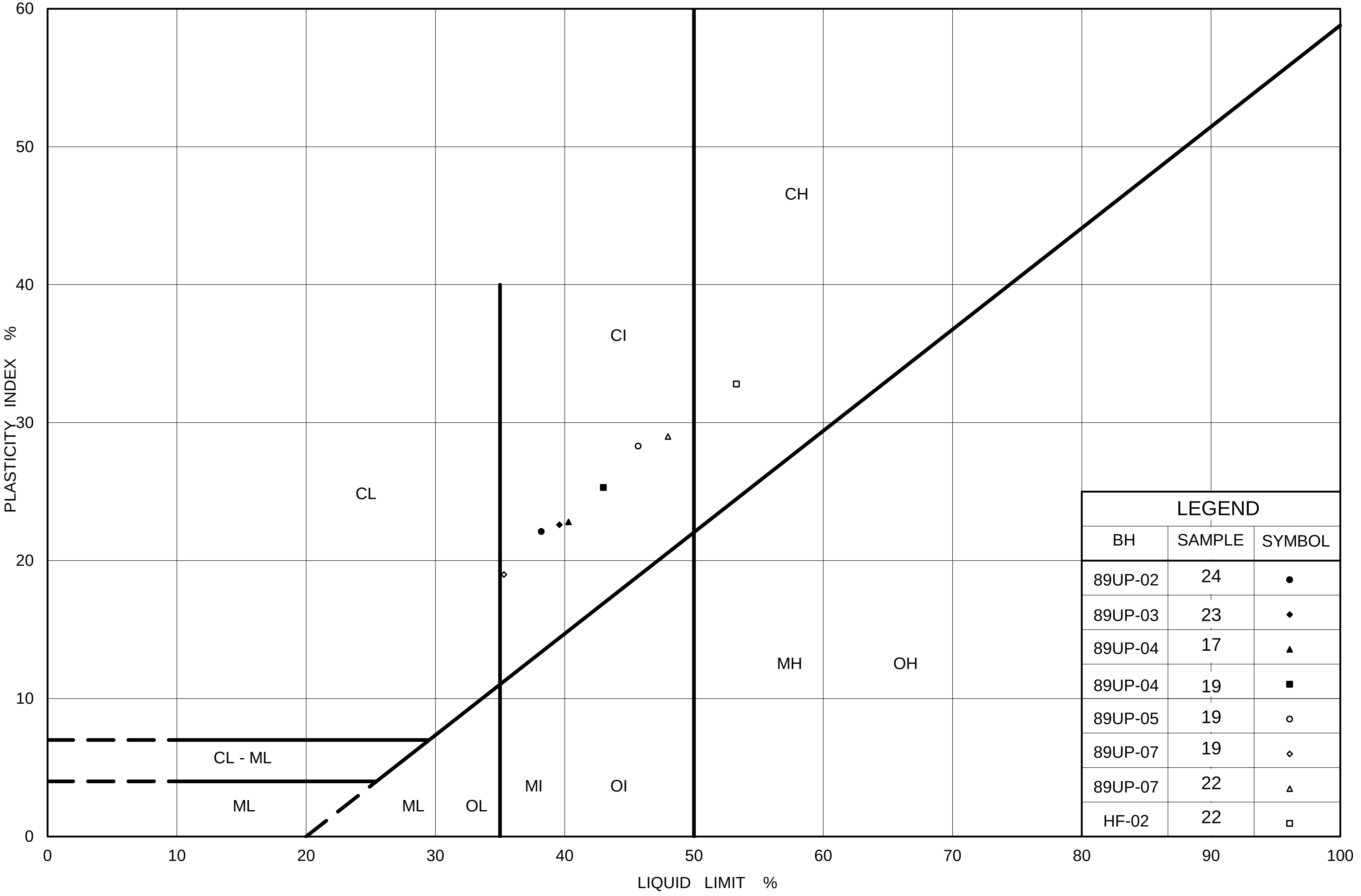
1. The dark bands (i.e., laminae) represent the silty clay of intermediate plasticity to clay of high plasticity, while the lighter varves represent the clayey silt of low plasticity and/or silt.
2. The soil samples were extracted from Shelby tubes and dried to illustrate the distinctions between the various varves.

Date: January 2018

Project No: 1668512

Golder Associates

Oct 75, FF-S-21



Ministry of Transportation

Ontario

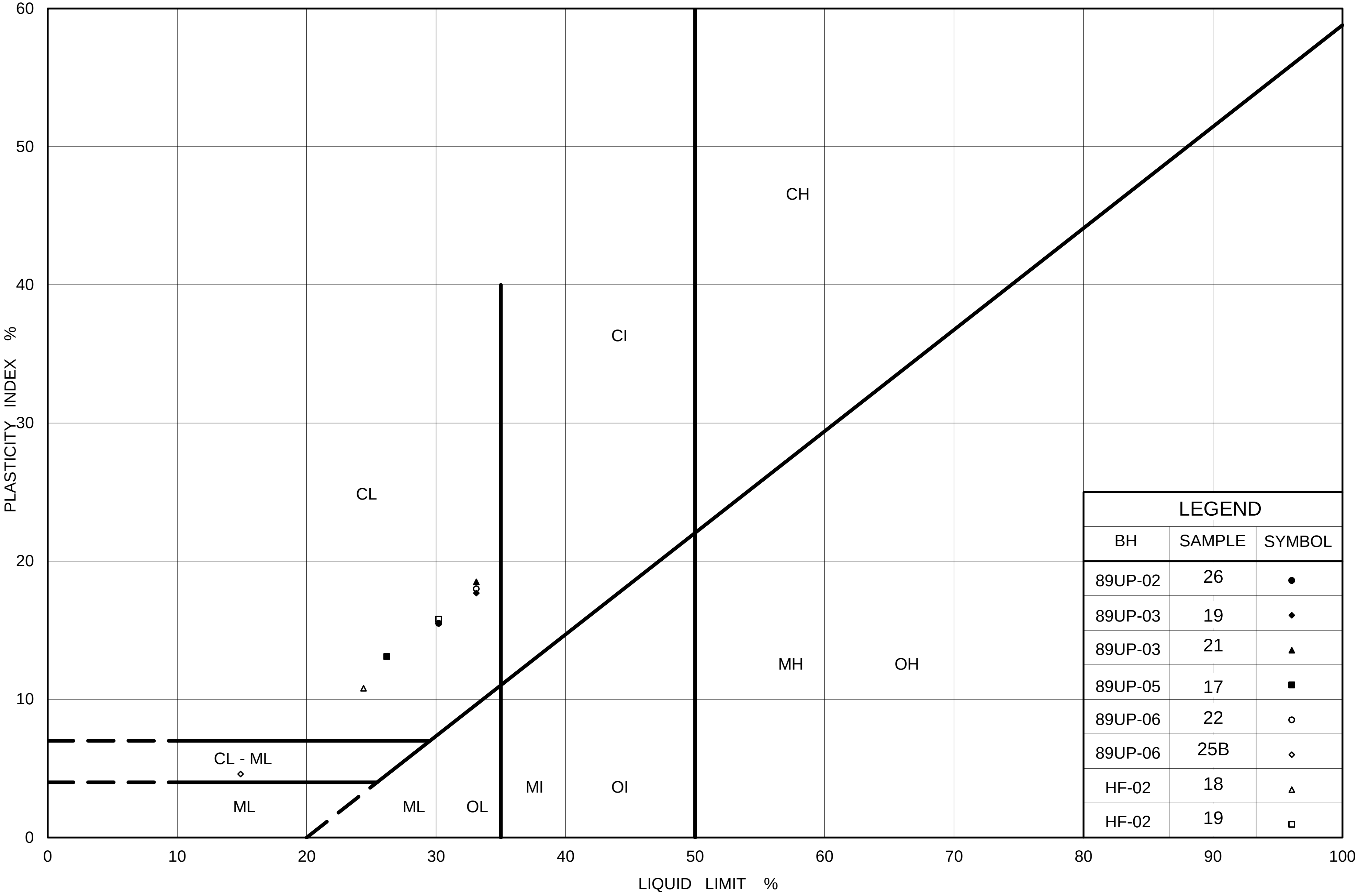
PLASTICITY CHART
Silty Clay to Clay (Upper Deposit)

Figure No. C-7A

Project No. 1668512

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Sandy Clayey Silt to Clayey Silt (Upper Deposit)

Figure No. C-7B

Project No. 1668512

Checked By: SMM

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE C-8A

SAMPLE IDENTIFICATION

Project Number	1668512(1000)	Sample Number	TO 20
Borehole Number	89UP-03	Sample Depth, m	24.39-25.00

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	7/27/2017		
Date Completed	8/15/2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m^3	18.46
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	13.67
Area, cm^2	31.57	Specific Gravity, measured	2.73
Volume, cm^3	80.00	Solids Height, cm	1.294
Water Content, %	35.04	Volume of Solids, cm^3	40.85
Wet Mass, g	150.60	Volume of Voids, cm^3	39.15
Dry Mass, g	111.52	Degree of Saturation, %	99.8

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.534	0.958	2.534				
5.90	2.534	0.958	2.534				
10.75	2.534	0.958	2.534				
20.53	2.532	0.957	2.532	60	2.27E-02	7.67E-05	1.70E-07
40.00	2.525	0.952	2.525	145	9.32E-03	1.36E-04	1.24E-07
78.82	2.507	0.937	2.507	211	6.31E-03	1.89E-04	1.17E-07
156.26	2.485	0.920	2.485	140	9.35E-03	1.13E-04	1.04E-07
226.14	2.465	0.905	2.465	118	1.09E-02	1.12E-04	1.20E-07
78.82	2.482	0.918	2.482				
40.00	2.488	0.923	2.488				
78.82	2.479	0.916	2.479	109	1.20E-02	9.15E-05	1.07E-07
226.11	2.461	0.902	2.461	113	1.14E-02	4.85E-05	5.40E-08
312.48	2.441	0.886	2.441	231	5.47E-03	9.05E-05	4.85E-08
441.12	2.388	0.845	2.388	409	2.96E-03	1.62E-04	4.70E-08
620.45	2.292	0.772	2.292	2196	5.07E-04	2.11E-04	1.05E-08
1241.20	2.109	0.630	2.109	778	1.21E-03	1.17E-04	1.39E-08
2481.08	1.981	0.531	1.981	470	1.77E-03	4.06E-05	7.05E-09
441.14	2.026	0.566	2.026				
224.46	2.052	0.586	2.052				
78.82	2.096	0.620	2.096				
20.53	2.149	0.660	2.149				
5.90	2.184	0.688	2.184				

Note:

Consolidation loading and unloading schedule assigned by the client.

cv and k are approximate only based on t90 estimated from Square Root of Time Method (ASTMD2435/2435M)

Specimen taken 48-56cm from top of the tube.

Specimen swelled under 10.75 kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.18	Unit Weight, kN/m^3	20.11
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	15.86
Area, cm^2	31.57	Specific Gravity, measured	2.73
Volume, cm^3	68.94	Solids Height, cm	1.294
Water Content, %	26.78	Volume of Solids, cm ^{3}	40.85
Wet Mass, g	141.39	Volume of Voids, cm ^{3}	28.09
Dry Mass, g	111.52		

Prepared By: LH

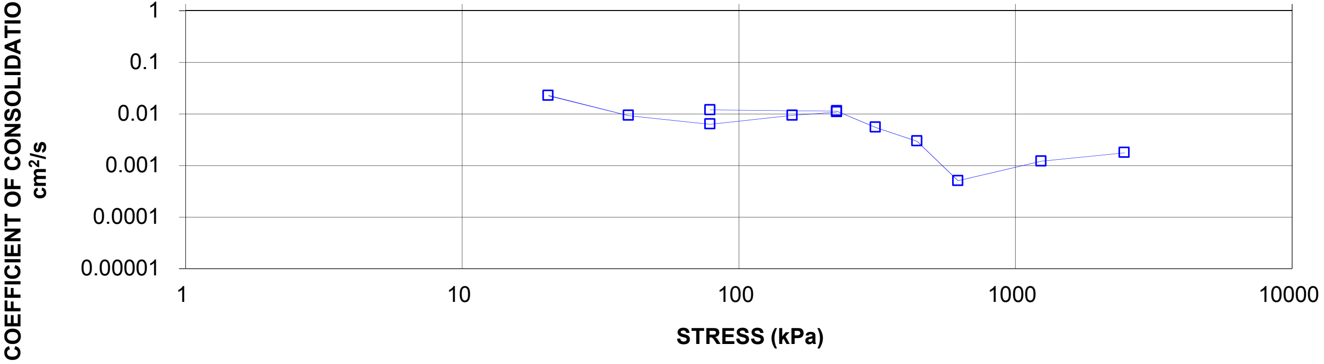
Golder Associates

Checked By: TZ

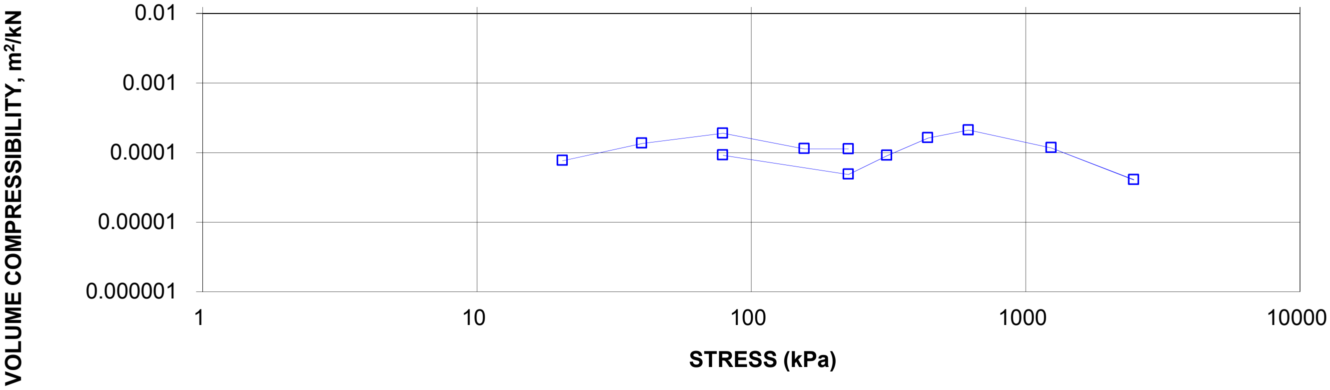
CONSOLIDATION TEST SUMMARY

FIGURE C-8B

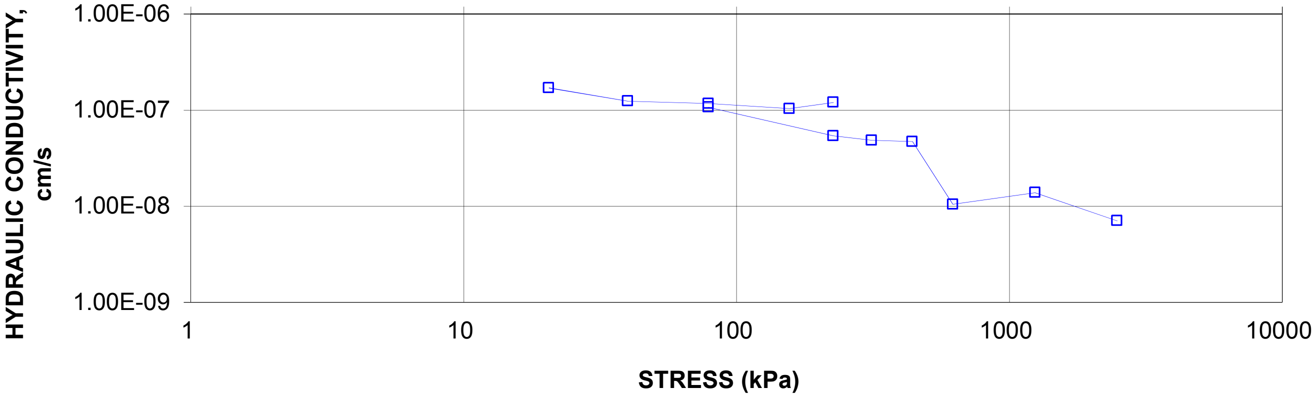
CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH 89UP-03 TO 20



CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH 89UP-03 TO 20

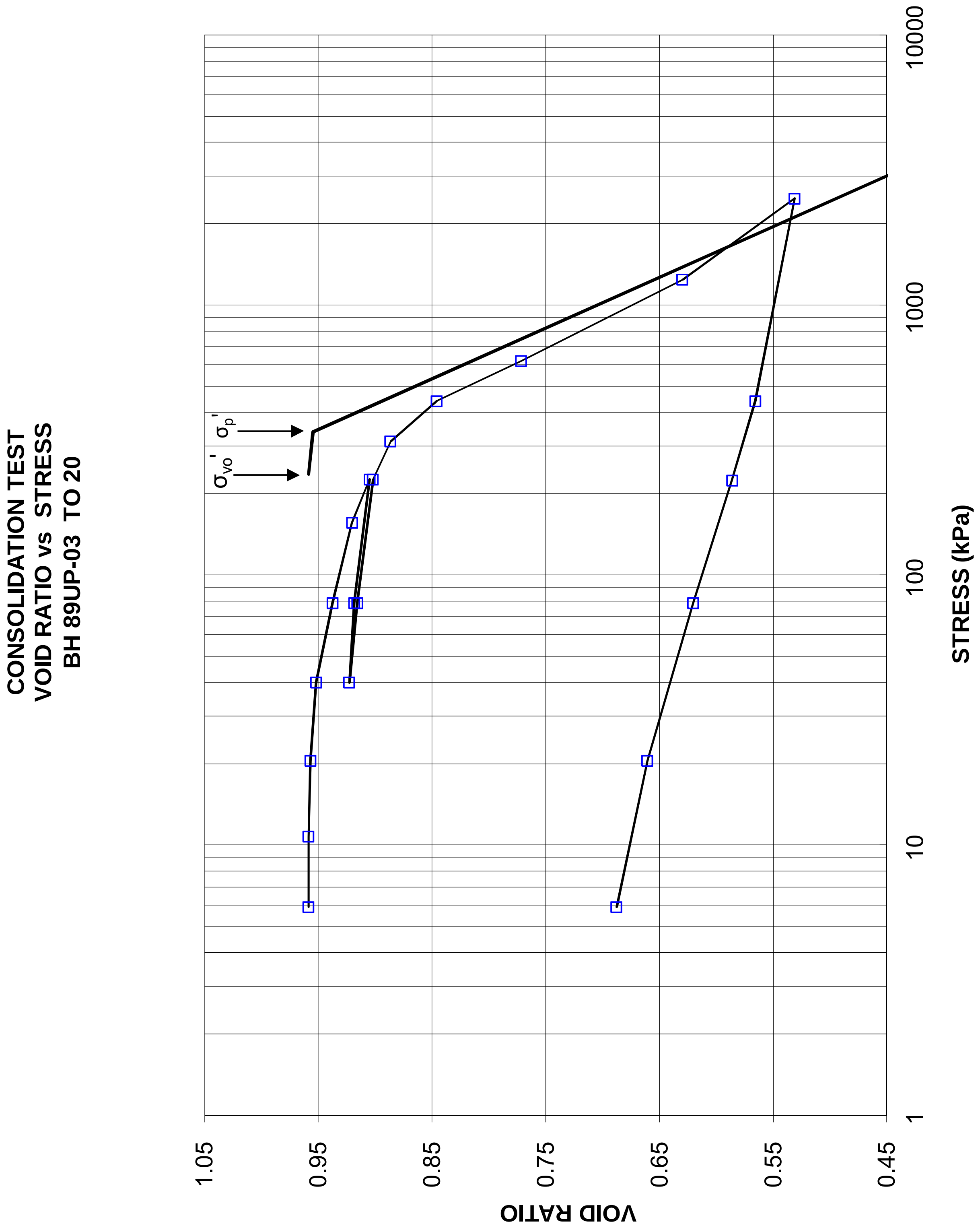


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 89UP-03 TO 20



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C-8C



Project No. 1668512(1000)

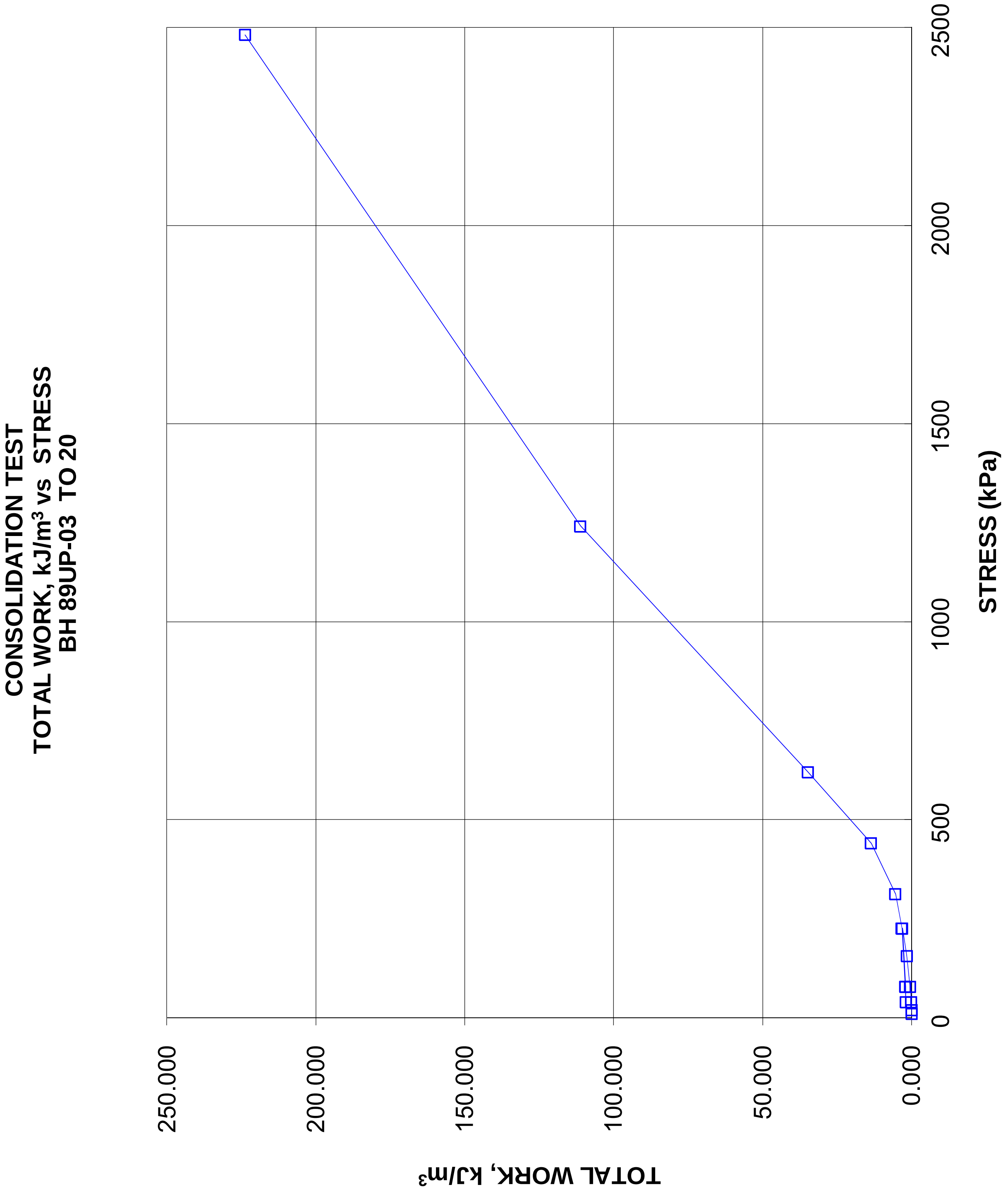
Prepared By: LH

Golder Associates

Checked By: TZ

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C-8D



Project No. 1668512(1000)

Golder Associates

Prepared By: LH

Checked By: TZ

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE C-9A

SAMPLE IDENTIFICATION

Project Number	1668512(1000)	Sample Number	24
Borehole Number	89UP-06	Sample Depth, m	36.93-36.88

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	8/23/2017		
Date Completed	9/13/2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	17.44
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	12.11
Area, cm^2	31.50	Specific Gravity, measured	2.71
Volume, cm^3	59.94	Solids Height, cm	0.867
Water Content, %	44.00	Volume of Solids, cm^3	27.32
Wet Mass, g	106.62	Volume of Voids, cm^3	32.62
Dry Mass, g	74.04	Degree of Saturation, %	99.9

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.903	1.194	1.903				
6.36	1.905	1.196	1.905				
11.22	1.907	1.199	1.907				
21.02	1.904	1.196	1.904	135	5.70E-03	1.37E-04	7.63E-08
40.53	1.892	1.182	1.892	392	1.94E-03	3.23E-04	6.14E-08
79.44	1.872	1.158	1.872	372	2.00E-03	2.82E-04	5.52E-08
157.05	1.851	1.134	1.851	296	2.45E-03	1.41E-04	3.40E-08
312.54	1.817	1.095	1.817	234	2.99E-03	1.12E-04	3.29E-08
410.46	1.793	1.068	1.793	265	2.57E-03	1.29E-04	3.26E-08
157.05	1.808	1.084	1.808				
40.53	1.840	1.121	1.840				
11.22	1.865	1.150	1.865				
40.48	1.857	1.141	1.857	360	2.03E-03	1.47E-04	2.93E-08
157.05	1.823	1.101	1.823	317	2.22E-03	1.55E-04	3.37E-08
410.34	1.781	1.054	1.781	267	2.52E-03	8.56E-05	2.11E-08
500.09	1.760	1.030	1.760	2579	2.55E-04	1.23E-04	3.07E-09
589.77	1.727	0.992	1.727	14789	4.28E-05	1.94E-04	8.12E-10
1197.85	1.564	0.803	1.564	1058	4.90E-04	1.41E-04	6.78E-09
2394.07	1.451	0.673	1.451	614	7.27E-04	4.97E-05	3.54E-09
589.78	1.490	0.717	1.490				
157.05	1.548	0.785	1.548				
40.53	1.615	0.862	1.615				
11.22	1.669	0.925	1.669				

Note:

Consolidation loading and unloading schedule assigned by the client.

cv and k are approximate only based on t90 estimated from Square Root of Time Method (ASTMD2435/2435M)

Specimen swelled under 11.22 kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.67	Unit Weight, kN/m^3	18.76
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	13.81
Area, cm^2	31.50	Specific Gravity, measured	2.71
Volume, cm^3	52.58	Solids Height, cm	0.867
Water Content, %	35.87	Volume of Solids, cm ^{3}	27.32
Wet Mass, g	100.60	Volume of Voids, cm ^{3}	25.26
Dry Mass, g	74.04		

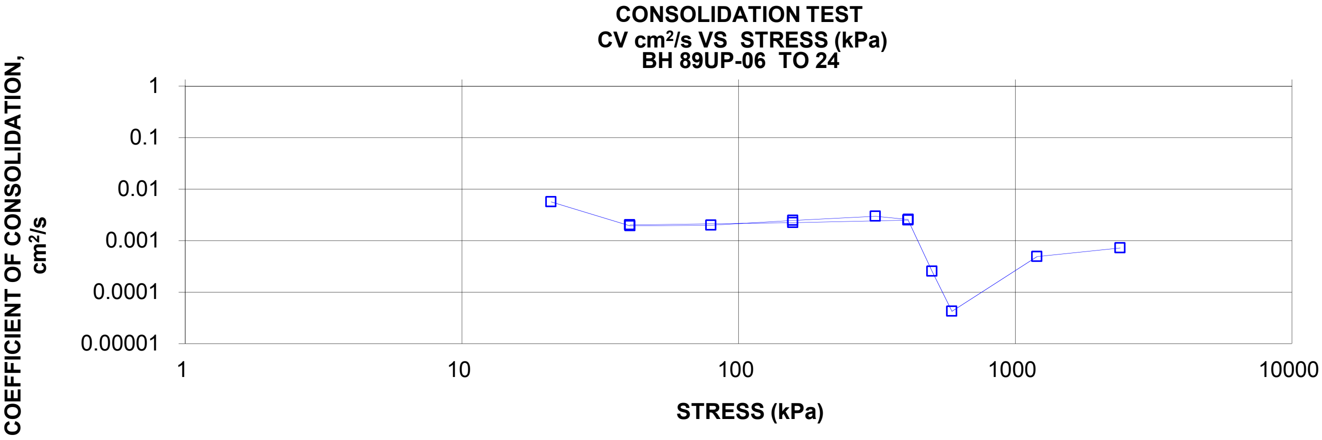
Prepared By: LH

Golder Associates

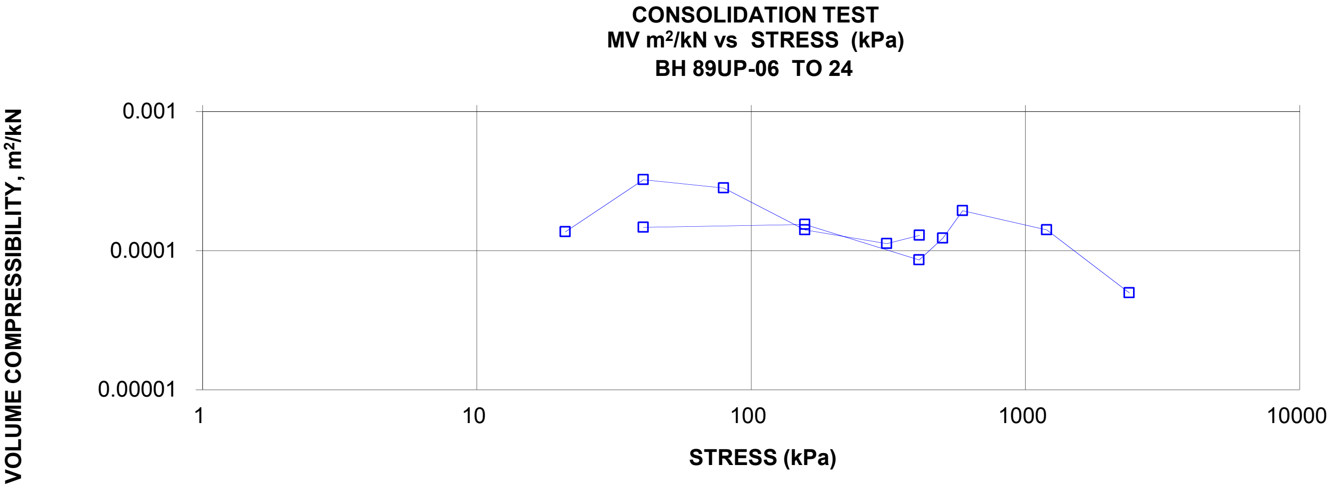
Checked By: TZ

CONSOLIDATION TEST SUMMARY

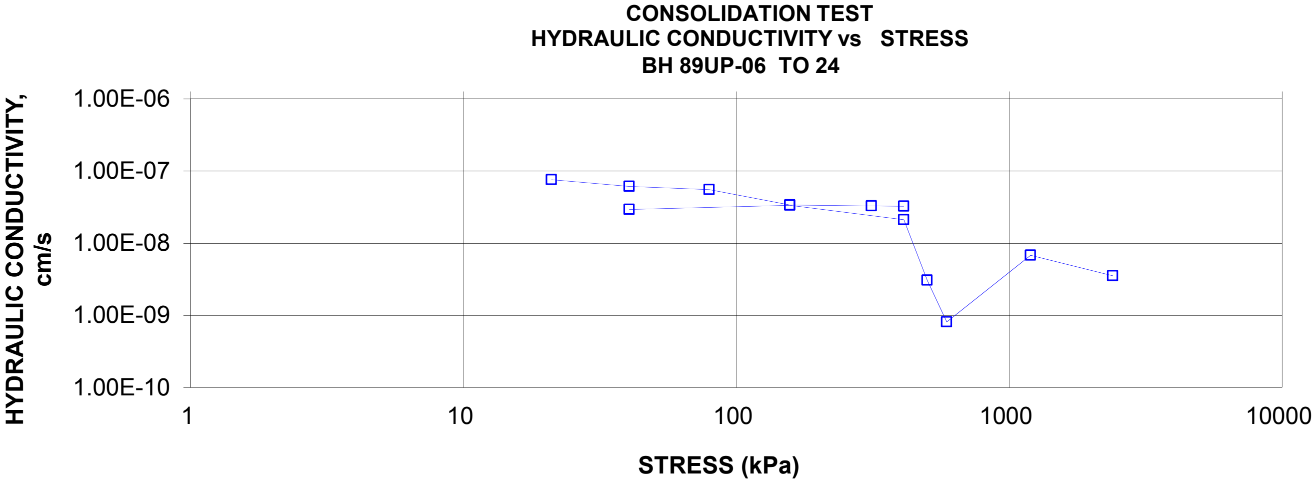
FIGURE C-9B



CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH 89UP-06 TO 24



CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH 89UP-06 TO 24



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 89UP-06 TO 24

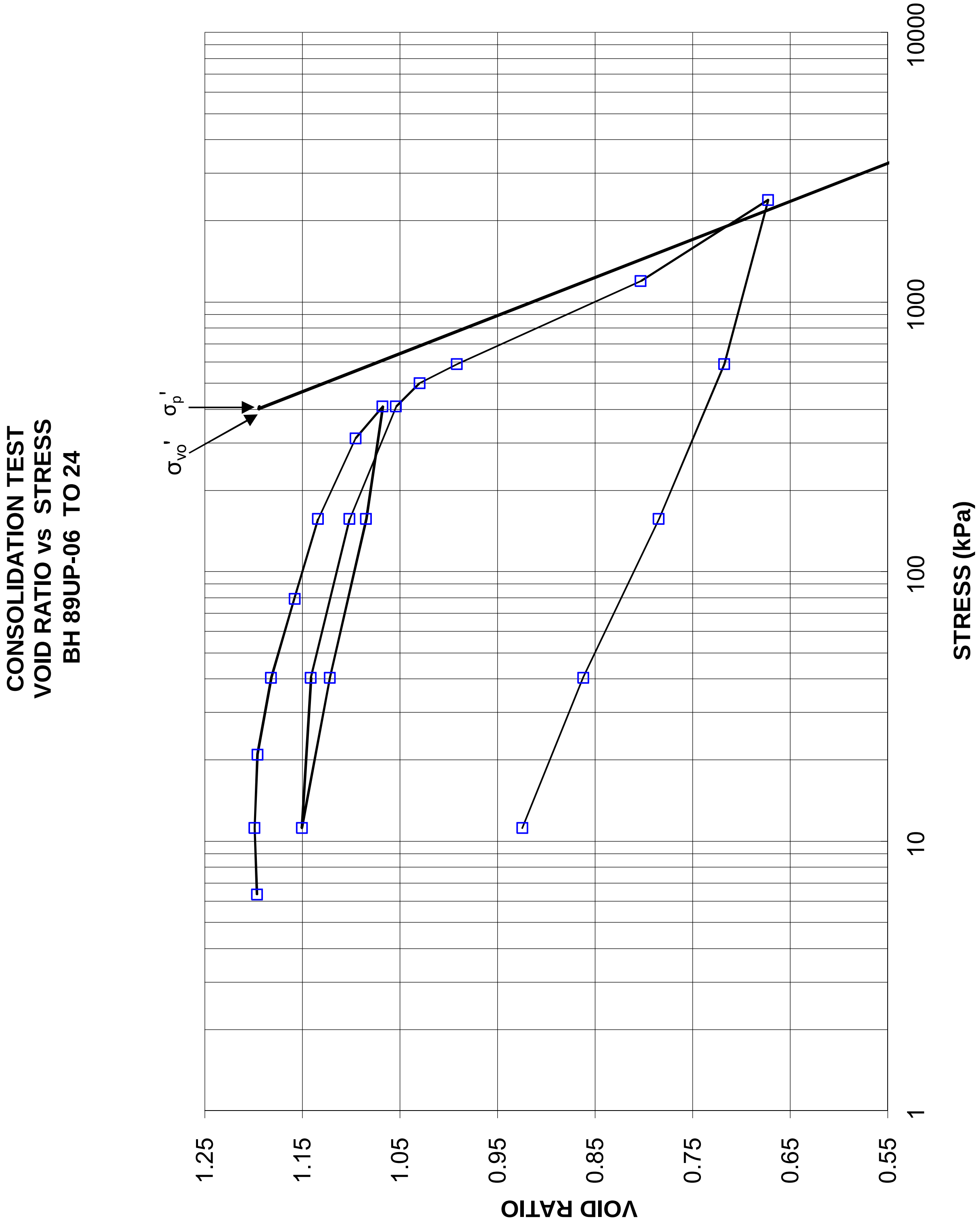
Project No. 1668512(1000)
Prepared By: LH

Golder Associates

Checked By: TZ

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C-9C



Project No. 1668512(1000)

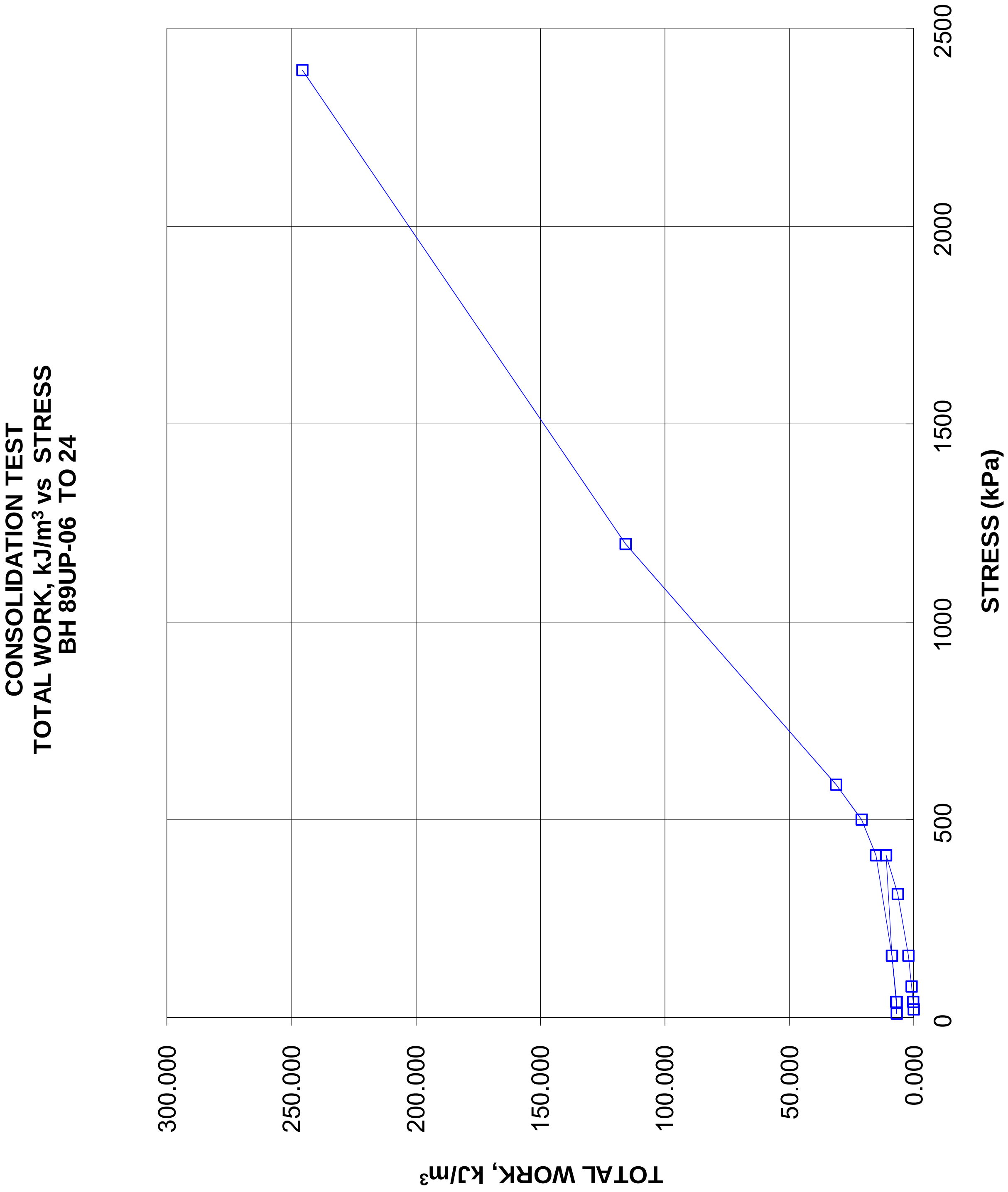
Prepared By: LH

Golder Associates

Checked By: TZ

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C-9D



Project No. 1668512(1000)

Prepared By: LH

Golder Associates

Checked By: TZ

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE C-10A

SAMPLE IDENTIFICATION

Project Number	1668512(1000)	Sample Number	21
Borehole Number	89UP-07	Sample Depth, m	25.91-26.52

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	8/10/2017		
Date Completed			

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.52	Unit Weight, kN/m^3	18.81
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.14
Area, cm^2	31.53	Specific Gravity, measured	2.75
Volume, cm^3	79.52	Solids Height, cm	1.322
Water Content, %	33.08	Volume of Solids, cm^3	41.68
Wet Mass, g	152.54	Volume of Voids, cm^3	37.84
Dry Mass, g	114.62	Degree of Saturation, %	100.2

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.522	0.908	2.522				
6.13	2.522	0.908	2.522				
10.99	2.522	0.908	2.522				
20.78	2.518	0.905	2.518	94	1.43E-02	1.46E-04	2.04E-07
40.27	2.508	0.898	2.508	83	1.61E-02	2.03E-04	3.20E-07
79.18	2.494	0.886	2.494	85	1.55E-02	1.49E-04	2.26E-07
156.72	2.471	0.869	2.471	144	8.99E-03	1.19E-04	1.04E-07
251.41	2.447	0.851	2.447	240	5.29E-03	1.01E-04	5.23E-08
79.18	2.462	0.863	2.462				
20.78	2.478	0.874	2.478				
79.18	2.467	0.866	2.467	126	1.02E-02	7.40E-05	7.43E-08
201.81	2.450	0.853	2.450	135	9.43E-03	5.43E-05	5.02E-08
401.46	2.419	0.830	2.419	154	8.06E-03	6.10E-05	4.81E-08
696.30	2.368	0.791	2.368	390	3.05E-03	6.93E-05	2.07E-08
1391.19	2.188	0.655	2.188	821	1.24E-03	1.03E-04	1.24E-08
2780.90	2.061	0.559	2.061	694	1.30E-03	3.61E-05	4.60E-09
695.70	2.093	0.583	2.093				
201.83	2.137	0.616	2.137				
79.18	2.169	0.641	2.169				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t90 estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen taken 18-22cm from bottom of the tube.
Specimen swelled under 10.99 kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.17	Unit Weight, kN/m^3	20.98
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.44
Area, cm^2	31.53	Specific Gravity, measured	2.75
Volume, cm^3	68.39	Solids Height, cm	1.322
Water Content, %	27.65	Volume of Solids, cm ^{3}	41.68
Wet Mass, g	146.31	Volume of Voids, cm ^{3}	26.71
Dry Mass, g	114.62		

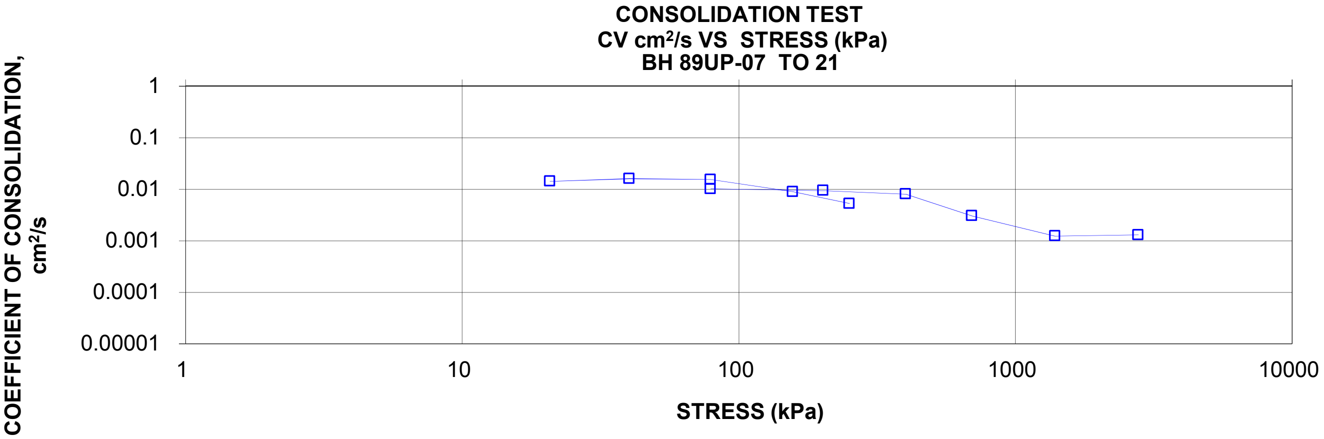
Prepared By: LH

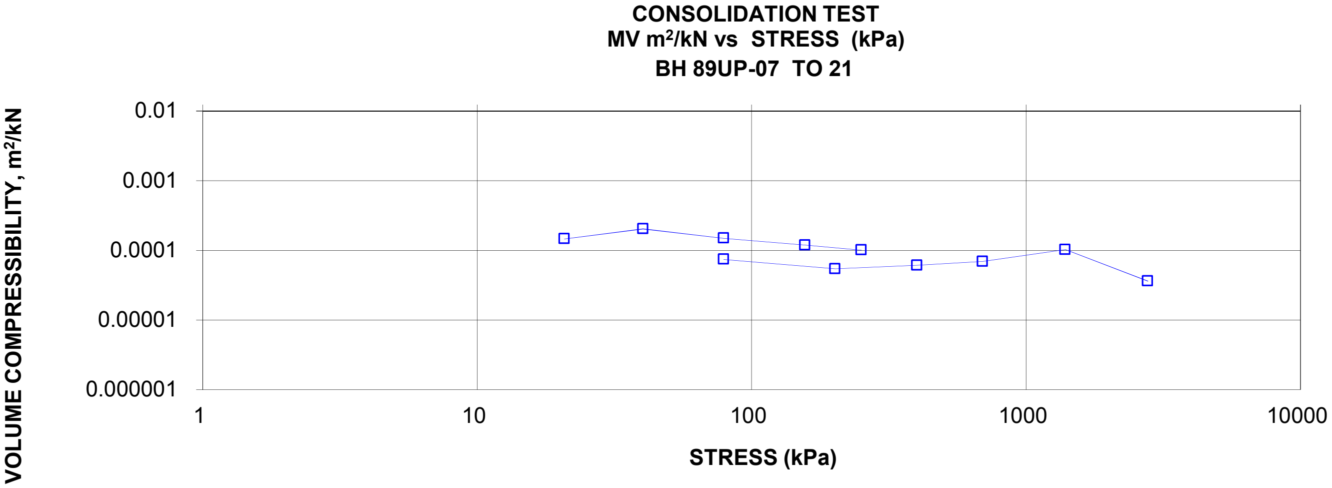
Golder Associates

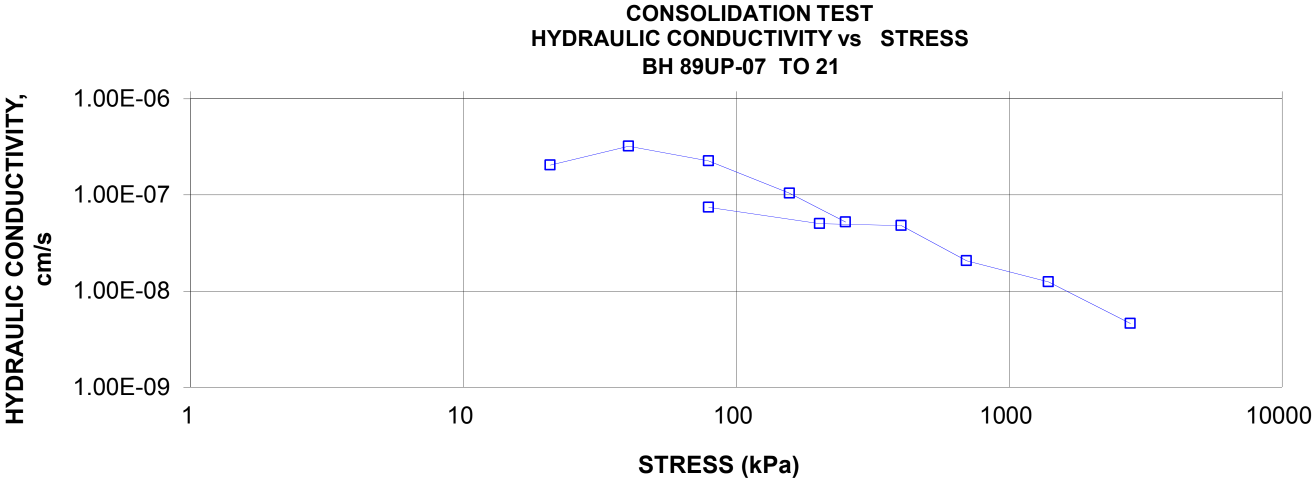
Checked By: TZ

CONSOLIDATION TEST SUMMARY

FIGURE C-10B







Project No. 1668512(1000)

Prepared By: LH

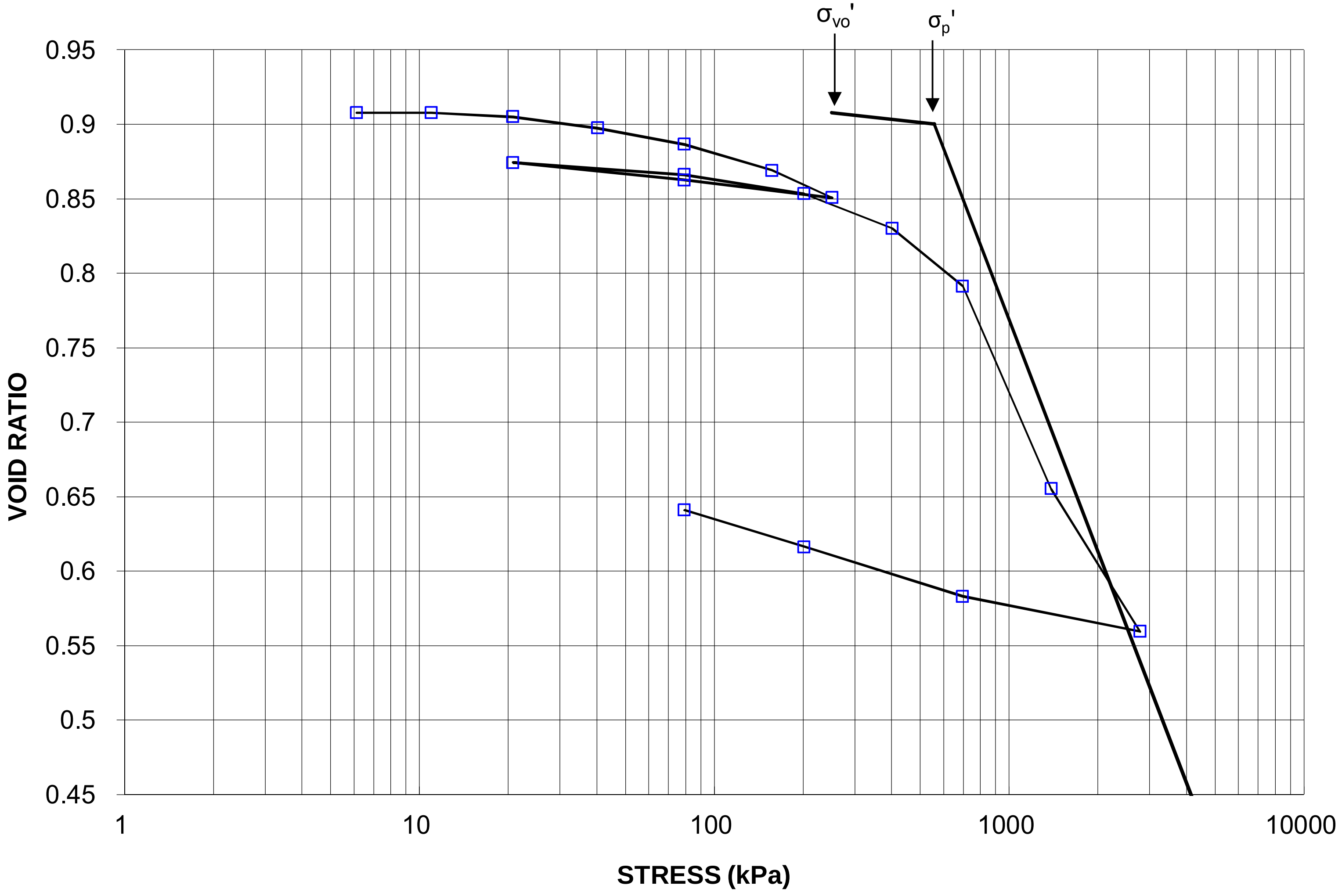
Golder Associates

Checked By: TZ

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C-10C

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 89UP-07 TO 21



Project No. 1668512(1000)

Prepared By: LH

Golder Associates

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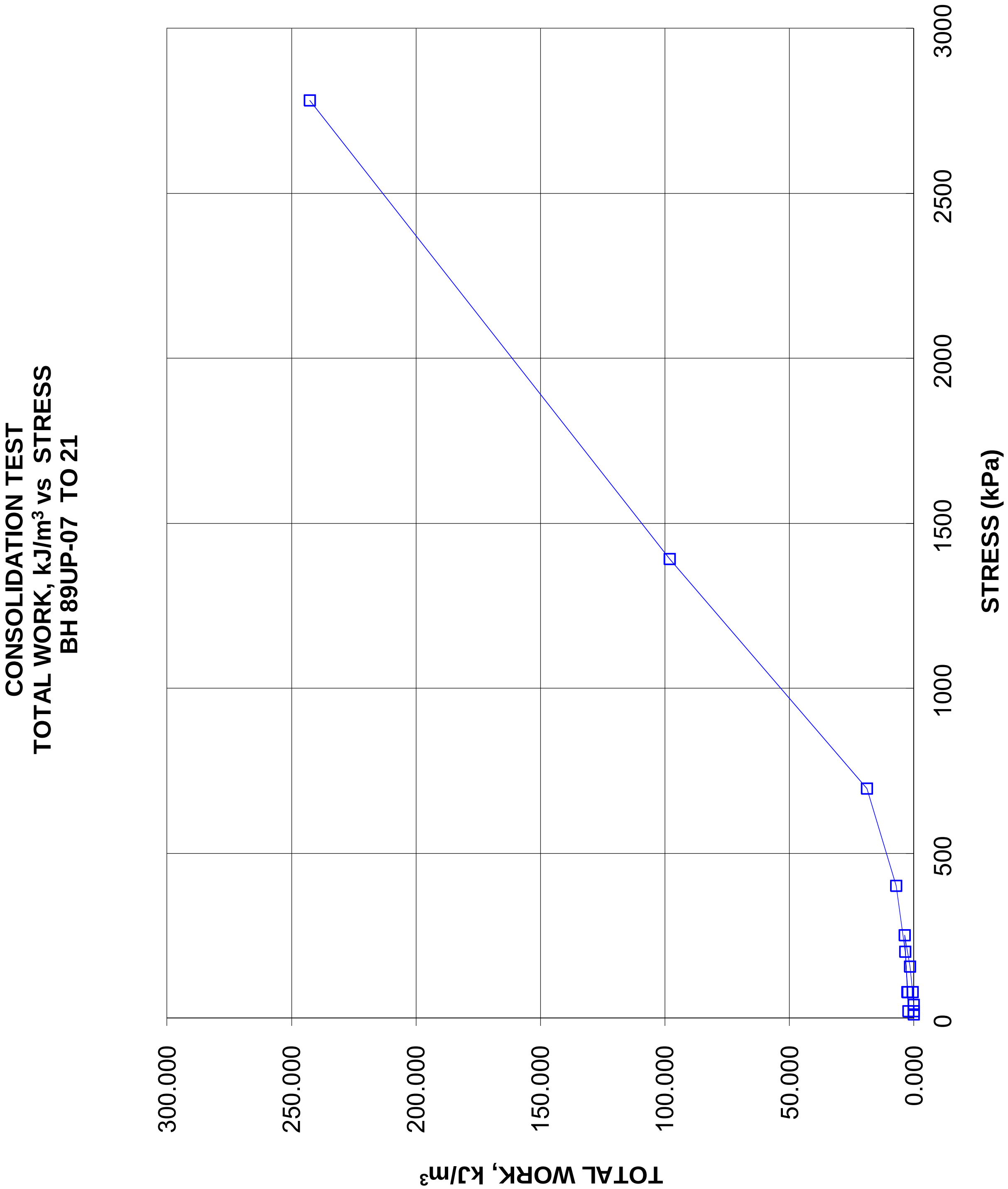
CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C-10D



Project No. 1668512(1000)

Golder Associates

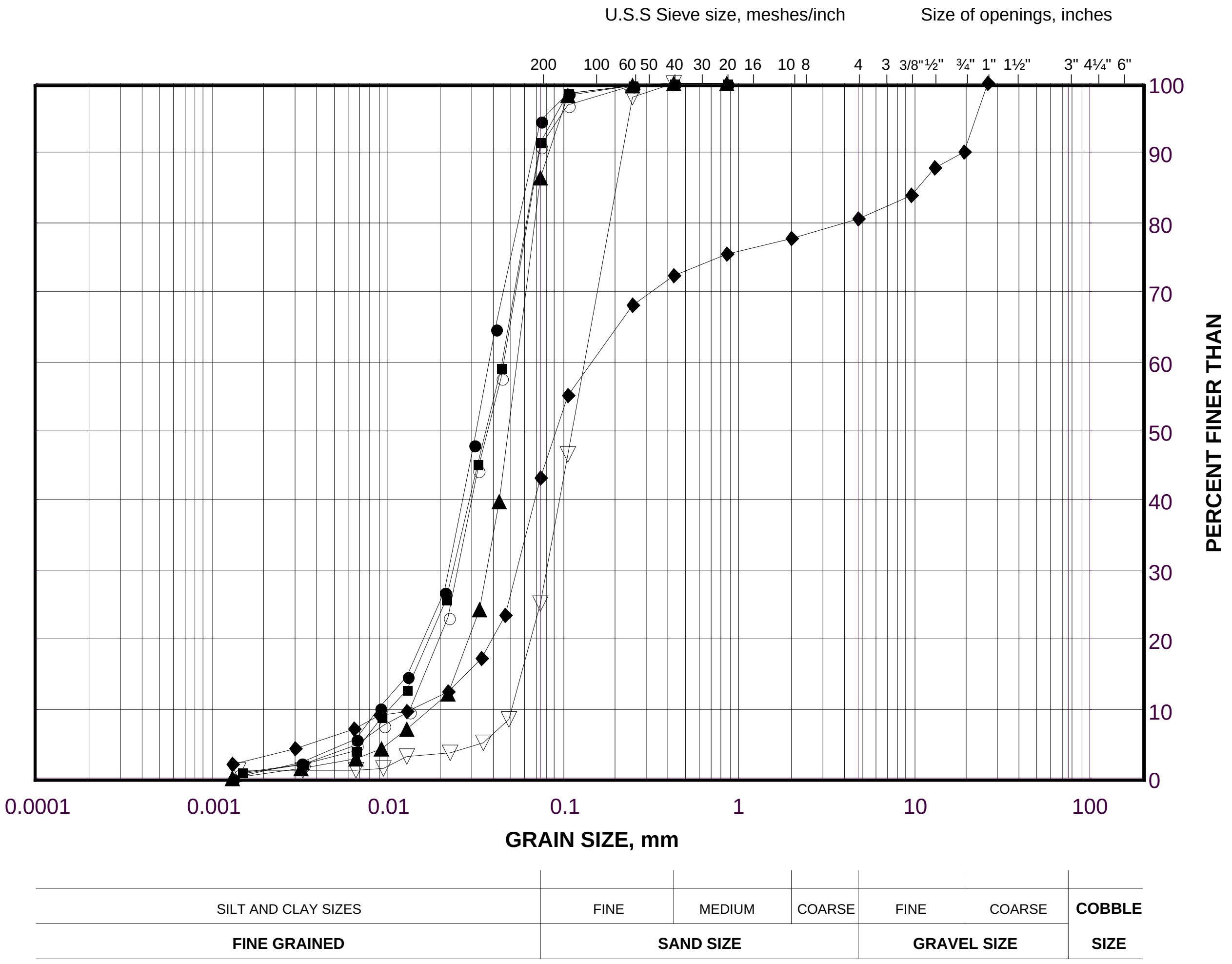
Prepared By: LH

Checked By: TZ

GRAIN SIZE DISTRIBUTION

Silt to Silt and Sand to Silty Sand (Lower Granular Deposit)

FIGURE C-11



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-04	21	190.9
■	89UP-05	21	193.8
◆	89UP-07	25	194.9
▲	89UP-03	26	193.6
▽	HF-02	27	192.1
❍	89UP-02	29	190.9

Project Number: 1668512

Checked By: SMM

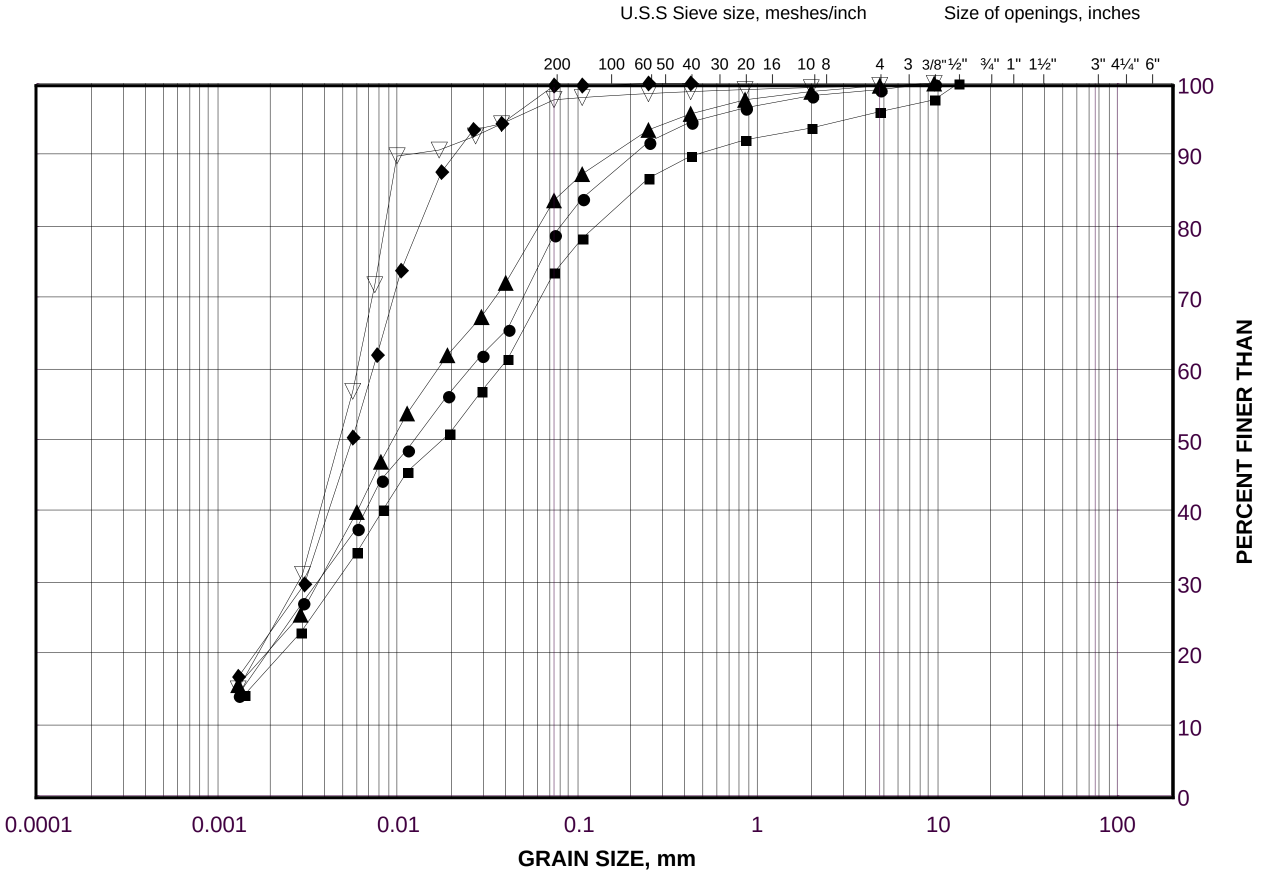
Golder Associates

Date: 05-Dec-17

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt (Lower Cohesive Deposit)

FIGURE C-12



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-04	22	187.8
■	89UP-07	27	188.8
◆	89UP-06	28	186.3
▲	89UP-03	29	184.4
▽	89UP-02	30	187.9

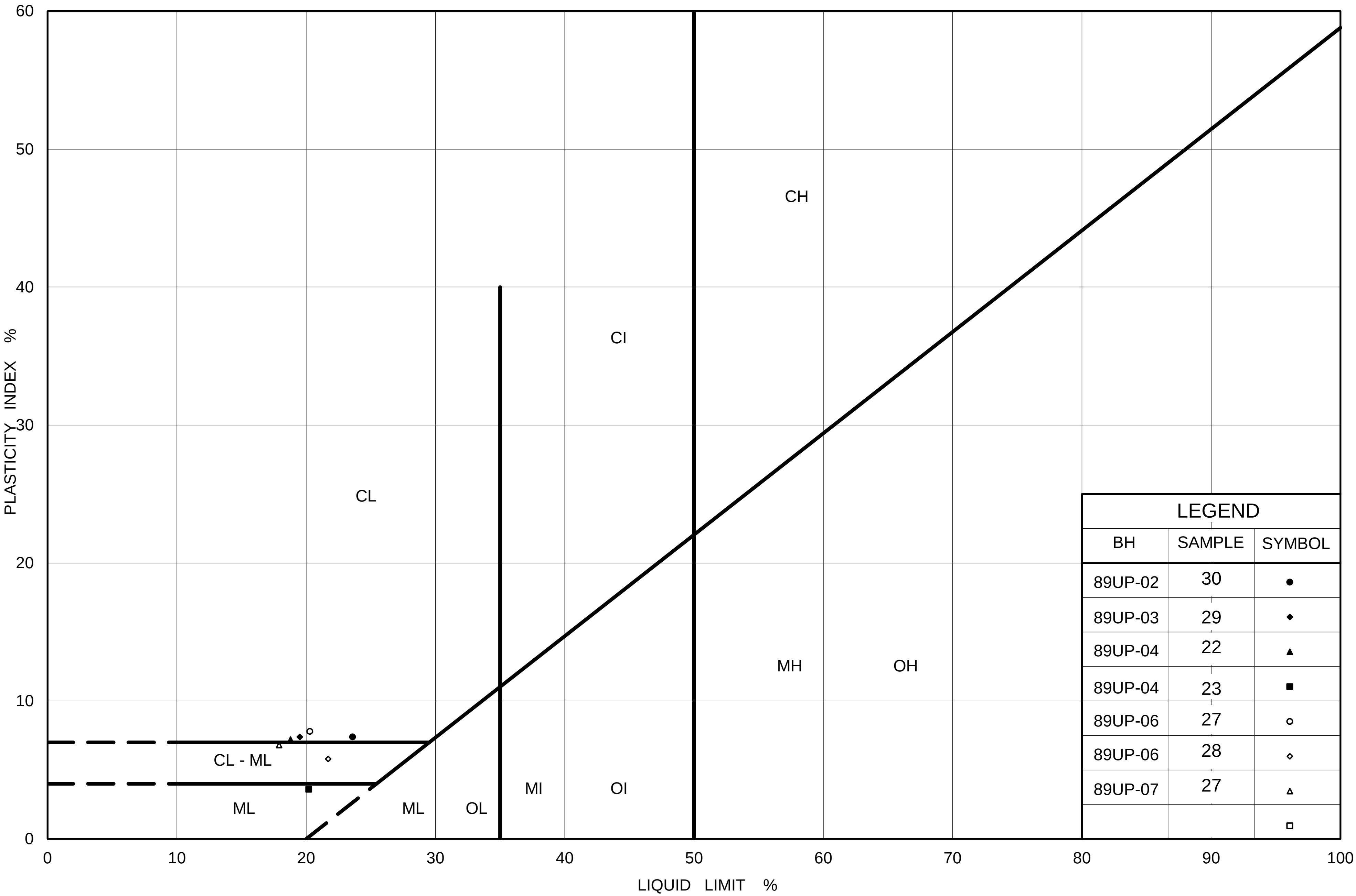
Project Number: 1668512

Checked By: SMM

Golder Associates

Date: 05-Dec-17

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
89UP-02	30	●
89UP-03	29	◆
89UP-04	22	▲
89UP-04	23	■
89UP-06	27	○
89UP-06	28	◇
89UP-07	27	△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART

Silt and Sandy Clayey Silt to Clayey Silt (Lower Cohesive Deposit)

Figure No. C-13

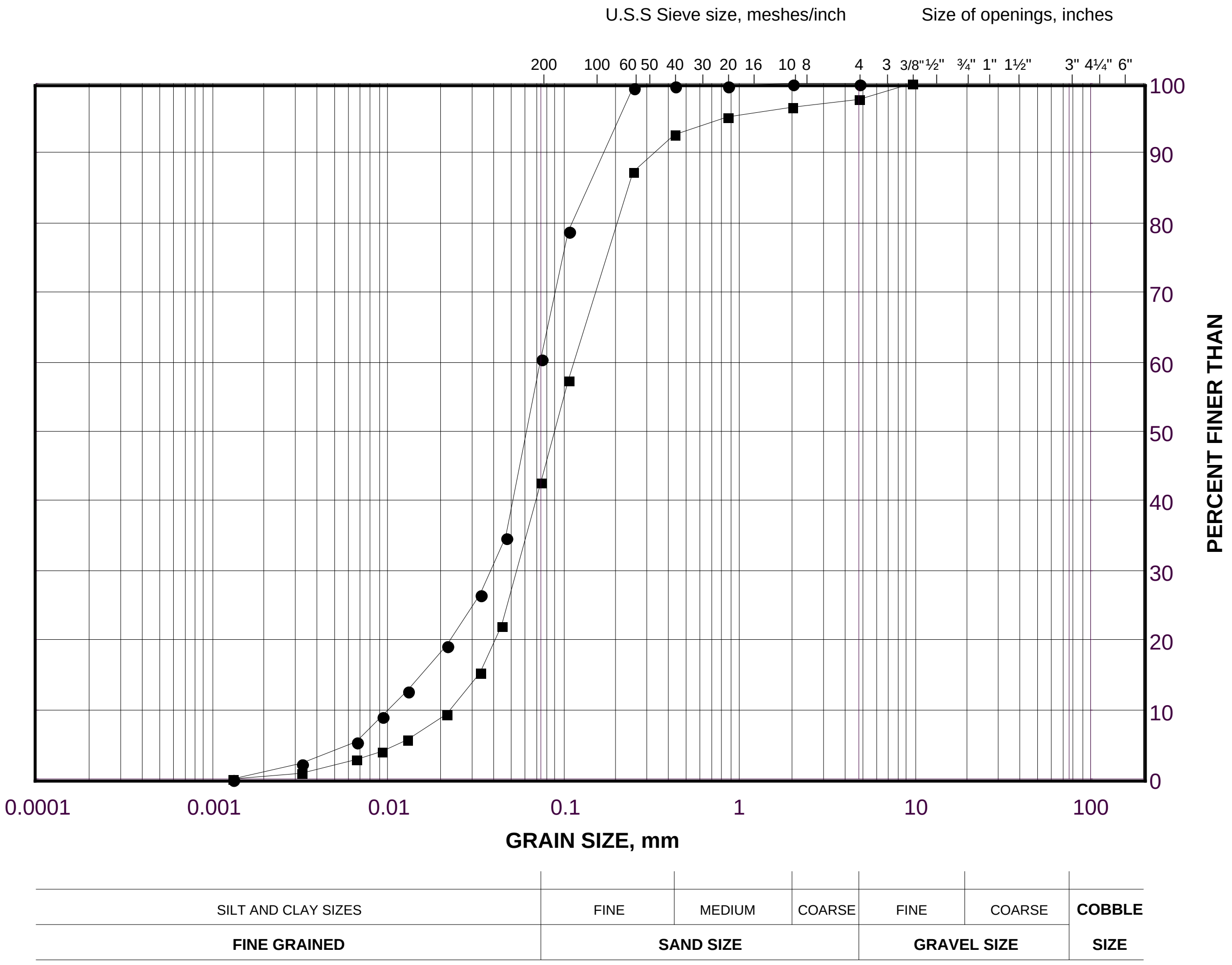
Project No. 1668512

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Silt and Sand (Till)

FIGURE C-14



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-07	29	182.9
■	89UP-03	31	178.3

Project Number: 1668512

Checked By: SMM

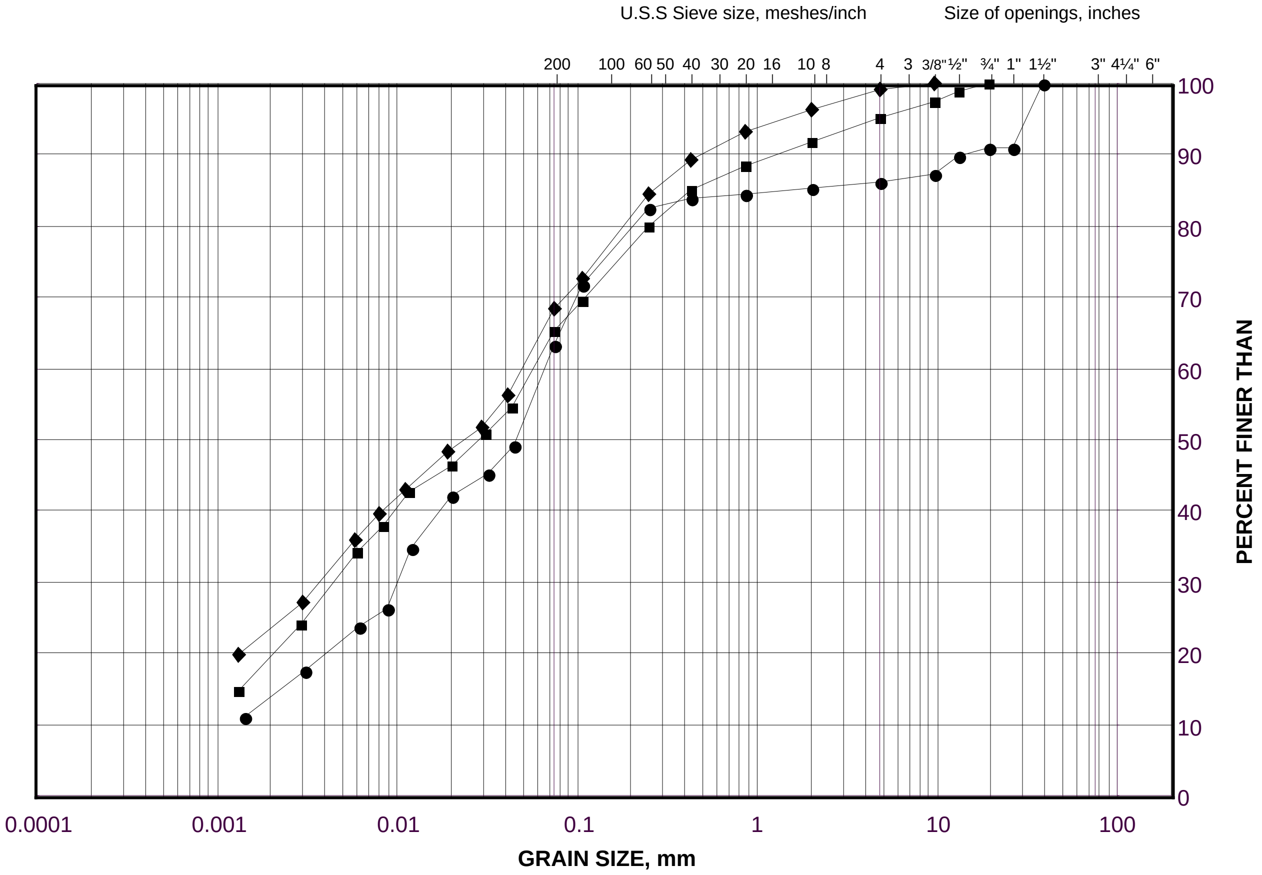
Golder Associates

Date: 05-Dec-17

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt with Sand (Till)

FIGURE C-15



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	89UP-05	24	184.7
■	89UP-04	25	178.9
◆	89UP-07	31	176.8

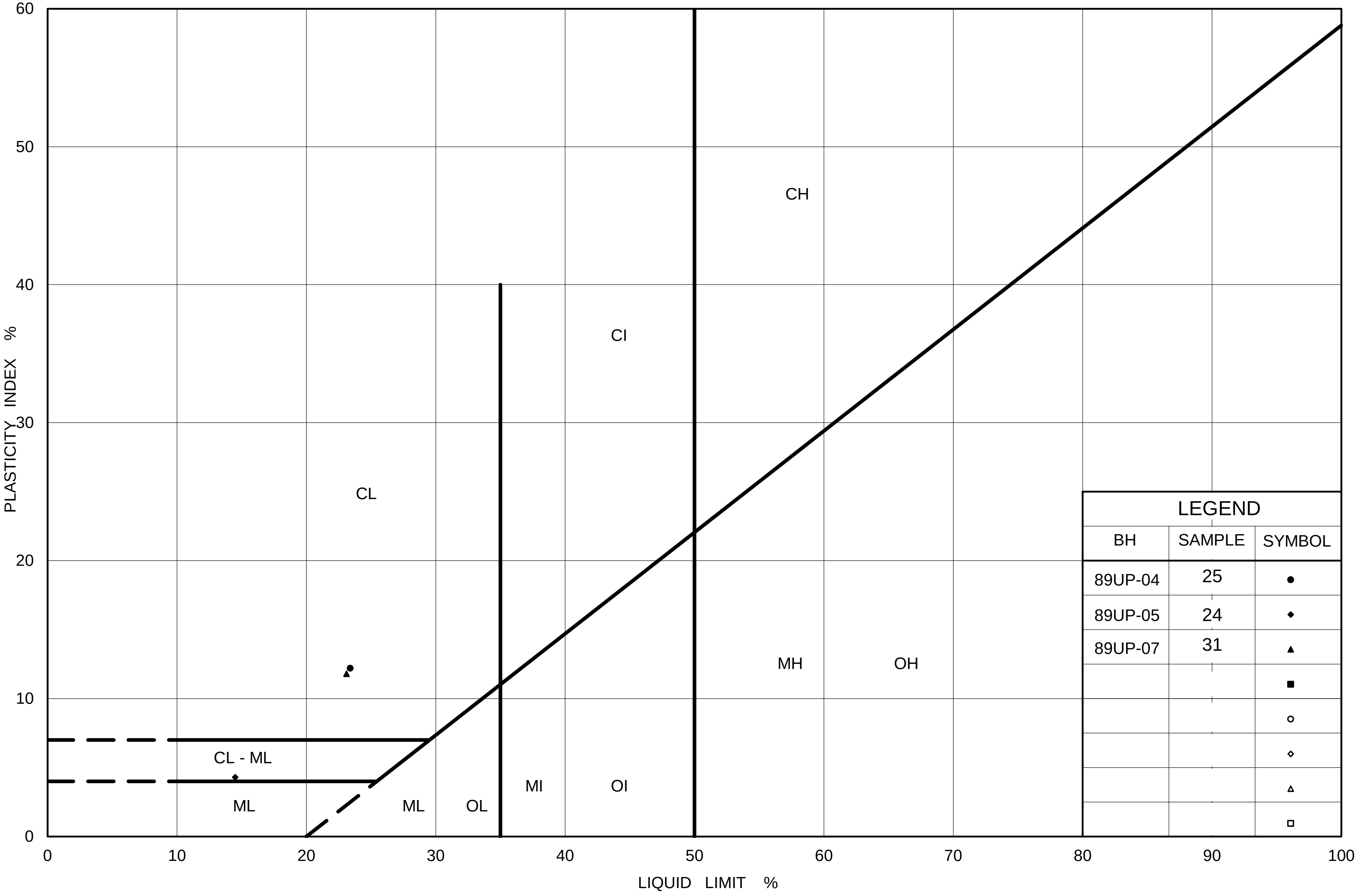
Project Number: 1668512

Checked By: SMM

Golder Associates

Date: 05-Dec-17

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
89UP-04	25	●
89UP-05	24	◆
89UP-07	31	▲
		■
		○
		◇
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		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Sandy Clayey Silt to Clayey Silt with Sand (Till)

Figure No. C-16

Project No. 1668512

Checked By: SMM

APPENDIX D

Analytical Laboratory Test Results

Your P.O. #: 1668512-1000
Your Project #: 1668512
Site Location: HWY 400/89
Your C.O.C. #: 30775

Attention:David Marmor

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/08/02
Report #: R4628079
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7F9592
Received: 2017/07/26, 17:23

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	N/A	2017/07/31	CAM SOP-00463	EPA 325.2 m
Conductivity	3	N/A	2017/08/01	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	3	2017/07/28	2017/07/28	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2017/07/27	2017/08/02	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	3	N/A	2017/07/31	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Your P.O. #: 1668512-1000
Your Project #: 1668512
Site Location: HWY 400/89
Your C.O.C. #: 30775

Attention:David Marmor

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/08/02
Report #: R4628079
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7F9592
Received: 2017/07/26, 17:23

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

RESULTS OF ANALYSES OF SOIL

Maxxam ID		EVB552	EVB553		EVB554		
Sampling Date		2017/06/11 12:00	2017/06/20 12:00		2017/06/26 12:00		
COC Number		30775	30775		30775		
	UNITS	**89UP-02 SA 1S**	**89UP-06 SA 23**	**RDL**	**89UP-05 SA 1**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	1200	2600		330		5094584
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	480	<20	20	1700	80	5098102
Conductivity	umho/cm	820	387	2	3060	2	5099515
Available (CaCl2) pH	pH	8.10	8.12		7.54		5096042
Soluble (20:1) Sulphate (SO4)	ug/g	35	220	20	<20	20	5098103

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

TEST SUMMARY

Maxxam ID: EVB552
Sample ID: 89UP-02 SA 1S
Matrix: Soil

Collected: 2017/06/11
Shipped:
Received: 2017/07/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5098102	N/A	2017/07/31	Deonarine Ramnarine
Conductivity	AT	5099515	N/A	2017/08/01	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5096042	2017/07/28	2017/07/28	Tahir Anwar
Resistivity of Soil		5094584	2017/08/02	2017/08/02	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5098103	N/A	2017/07/31	Deonarine Ramnarine

Maxxam ID: EVB553
Sample ID: 89UP-06 SA 23
Matrix: Soil

Collected: 2017/06/20
Shipped:
Received: 2017/07/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5098102	N/A	2017/07/31	Deonarine Ramnarine
Conductivity	AT	5099515	N/A	2017/08/01	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5096042	2017/07/28	2017/07/28	Tahir Anwar
Resistivity of Soil		5094584	2017/08/02	2017/08/02	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5098103	N/A	2017/07/31	Deonarine Ramnarine

Maxxam ID: EVB554
Sample ID: 89UP-05 SA 1
Matrix: Soil

Collected: 2017/06/26
Shipped:
Received: 2017/07/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5098102	N/A	2017/07/31	Deonarine Ramnarine
Conductivity	AT	5099515	N/A	2017/08/01	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5096042	2017/07/28	2017/07/28	Tahir Anwar
Resistivity of Soil		5094584	2017/08/02	2017/08/02	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5098103	N/A	2017/07/31	Deonarine Ramnarine

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	11.0°C

Samples have been received and analyzed past the recommended hold time.

CONDUCT-S: Analysis was performed past sample holding time. This may increase the variability associated with these results.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
5096042	Available (CaCl2) pH	2017/07/28			99	97 - 103			0.11	N/A
5098102	Soluble (20:1) Chloride (Cl)	2017/07/31	NC	70 - 130	103	70 - 130	<20	ug/g	0.50	35
5098103	Soluble (20:1) Sulphate (SO4)	2017/07/31	NC	70 - 130	109	70 - 130	<20	ug/g	0.93	35
5099515	Conductivity	2017/08/01			100	90 - 110	<2	umho/cm	0.77	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
Eva Pranjic
CHEMIST

Ewa Pranjic, M.Sc., C.Chem, Scientific Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

IMMEDIATE

6740 Campobello Road, Mississauga, Ontario L5N 2L8 www.maxxam.ca
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

CHAIN OF CUSTODY RECORD

30775

Page ____ of ____

INVOICE INFORMATION	REPORT INFORMATION (if differs from invoice)	PROJECT INFORMATION
Company Name: GOLDER ASSOCIATES	Company Name:	Quotation #:
Contact Name: DAVID MRMOR	Contact Name:	P.O. #: 1668512 - 1000
Address: 6925 ~~50~~ CENTURY	Address:	Project #: 1668512
AVE. SUITE #1001 MISSISSAUGA		Site Location: Hwy 400/89
Phone: 905-567-4444 Fax: 905 567 6561	Phone: Fax:	Site #:
Email: david_marmor@golder.com	Email:	Sampled By: Derek Franceschini

TURNAROUND TIME (TAT) REQUIRED

☒ Regular TAT (5-7 days)

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Applicable Surcharge)

☐ 1 Day (100%)
☐ 2 Days (50%)
☐ 3-4 Days (25%)

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

REGULATION 153 (2011)			OTHER REGULATIONS	
☐ Table 1	☐ Res/Park	☐ Med/Fine	☐ CCME	☐ Sanitary Sewer Bylaw
☐ Table 2	☐ Ind/Comm	☐ Coarse	☐ MISA	☐ Storm Sewer Bylaw
☐ Table 3	☐ Agri/Other		☐ PWQO	Municipality:
☐ Table ___			☐ Other (Specify):	
FOR RSC (PLEASE CIRCLE) YES / NO			☐ REG 558 (MINIMUM 3 DAY TAT REQUIRED)	

Include Criteria on Certificate of Analysis (Y/N)? ____

Rush Confirmation #:

Date Required:

LABORATORY USE ONLY

CUSTODY SEAL (Y/N)		Temperature (°C) on Receipt
Present	N	11/9/13
Intact	N	
COOLING MEDIA PRESENT (Y / N)	Y	

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	MATRIX	# OF CONT.	FIELD FILTERED (PLEASE CIRCLE) Metals / Hg / CrVI	Corrosivity Package (pH, sulphate, chloride, resistivity, electrical conductivity)											COMMENTS / TAT COMMENTS
1	89UP-02 SA 15	June 14/17	12am	S	2		X											standard corrosivity only
2	89UP-06 SA 23	June 20/17	12am	S	2		X											" "
3	89UP-05 SA 1	June 26/17	12am	S.	2		X											" "
4																		
5																		
6																		
7																		
8																		
9																		
10																		

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	# JARS U AND N SUBMIT
David Marmor	2017/07/26	5pm	MSG	2017/07/26	17:23	

26-Jul-17 17:23
Ema Gitej
B7F9592
HG2 ENV-757

COC-1004 (11/13) - ENV. ENG.

Maxxam Analytics International Corporation o/a Maxxam Analytics

APPENDIX E

Non-Standard Special Provisions, Notice to Contractor and Monitoring Drawings

CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

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

Submission and Design Requirements

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

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

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

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

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

Material

Corrugated steel pipe

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

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

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

Sand Fill

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

Table 1 – Sand Fill Gradation Requirements

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

Construction

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

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

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

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.
The CSP at each pile shall be constructed to the following tolerances:

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

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

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

Basis of Payment

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

END OF SECTION

DEEP FOUNDATIONS – Item No.

Special Provision

Amendment to OPSS.PROV 903, April 2016

903.01 SCOPE

Section 903.01 of OPSS.PROV 903 is amended by the addition of the following:

Under the above tender items, the Contractor shall:

a) Supply and install H-Piles

b) Provide 20 mm plywood sheet to cover CSP.

c) Coordinate with the Contractor Administrator or an independent testing company retained by the Contract Administrator for Pile Dynamic Analyzer (PDA) testing.

All as shown on the Contract Drawings.

903.02 REFERENCES

Section 903.02 of OPSS.PROV 903 is amended by the addition of the following:

The subsurface conditions at the site are described in the following Foundation Investigation Report:

> Highway 400/89 Underpass Replacement Structure Site No. 30-256, Reconstruction of Highway 400/89 Interchange, Town Of Innisfil, Simcoe County Ministry Of Transportation, Ontario, G.W.P. 2438-13-00

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS.PROV 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at end of drive (EOD) as performed by Contract Administrator or an independent testing company retain by the Contract Administrator. If the specified ultimate resistance is not achieved, retap / restrike should be conducted after sufficient tie has passed to allow soil setup. The requirements for soil setup are as specified in the Contract Documents.

903.07.02.07.04.02 High-Strain Dynamic Testing

High-strain dynamic testing shall be performed by the Contract Administrator or an independent testing company retained by the Contract Administrator using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for information purposes. The final piles to be tested will be decided by the Contact Administrator.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles, or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles, or as specified in the Contract Documents.

Restrike testing shall be carried out no sooner than 48 hours after installation of the individual pile and at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.07.02.07.06 Retapping Tests on Piles

Section 903.07.02.06 is deleted in its entirety and replaced by the following:

In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles, shall be retapped no sooner than 48 hours after installation of the individual pile to confirm that the ultimate axial geotechnical resistance has been achieved and/or sustained.

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT – Item No.

Special Provision

1.0 SCOPE

The Contractor shall retain a Foundation Engineering consultant registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical Specialty – Medium Complexity", to undertake the supply and installation of geotechnical settlement monitoring instrumentation (settlement plates, settlement rods and temporary benchmarks) and for providing appropriate recommendations based on the measurement readings.

The Contractor shall retain a Foundation Engineering consultant registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical Specialty – High Complexity", to undertake the supply and installation of geotechnical monitoring instrumentation (vibrating wire piezometers and vibrating wire inline extensometers) and for providing appropriate recommendations based on the measurement readings.

The Contractor shall also implement an embankment monitoring program to take, record and distribute all appropriate and timely measurements and transmit recommendations from the Foundation Engineering consultants to the Contract Administrator.

1.1 General Scope

This general special provision and the other item-specific special provisions contain the requirements for the supply and installation of the following geotechnical monitoring instrumentation:

- Settlement Plates (SP);
- Deep Settlement Rods (DSR);
- Vibrating Wire Piezometers (VWP); and,
- Vibrating Wire Inline Extensometer (VWIX).

This general special provision also contains the requirements for the supply and installation of temporary survey Benchmarks related to the geotechnical monitoring instrumentation.

1.2 Purpose

The purpose of these instruments and equipment is to monitor the progress of the settlement at the abutments during the preload period and to monitor the progress of settlement in the foundation soils under and adjacent to the high fill embankments along the W-S Ramp and the combined embankment for the W-N Ramp and the S-E/W Ramp where it crosses the existing gas main. The purpose of the temporary survey Benchmarks is to provide non-settling references for the surveying of the monitoring instruments.

The duration of the preloading period prior to driving the steel H-piles at the abutments will be controlled by the instrumentation readings, as specified elsewhere in the Contract Documents. The completed, preloaded embankments at the abutments shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of compression of the foundation soil has been achieved.

2.0 REFERENCES

2.1 General

When the Contract Documents indicate that provincial oriented specifications are to be used and there is a provincial oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be considered to be the OPSS listed, unless use of a municipal oriented specification is specified in the Contract Documents.

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 905 Steel Reinforcement for Concrete

Ontario Provincial Standards Specifications, Material

OPSS1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production

OPSS 1250 Clay Seal

OPSS 1301 Cementing Materials

OPSS 1801 Corrugated Steel Pile (CSP) Products

Ontario Water Resources Act RRO 1990:

Regulation 903 Wells

2.2 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Reports for this Contract.

> Foundation Investigation Report – High Fill Embankment, Highway 400/89 Reconstruction, Town of Innisfil, Simcoe County, Ministry of Transportation, Ontario. G.W.P. 2438-13-00
>
> Foundation Investigation Report – Highway 400/89 Underpass Replacement, Structure Site No. 30-256, Reconstruction of Highway 400/89 Interchange, Town of Innisfil, Simcoe County, G.W.P. 2483-13-00
>
> Foundation Investigation Report – Trenchless Installation of Proposed Culvert C-21, Highway 400/89 Interchange Reconstruction, Town of Innisfil, Simcoe County, G.W.P. 2483-13-00

3.0 DEFINITIONS

Contractor means the Contractor and his Geotechnical Consultant.

Equal shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

Geotechnical Engineering Consultant means a consultant with MTO classification of “Geotechnical (Structures and Embankments) - High Complexity”, to undertake the supply and installation of geotechnical instruments.

Monitoring Program means the monitoring readings conducted by others as part of the Contract Administration Assignment.

Settlement Plate means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

Settlement Plate means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

Temporary Survey Benchmark means a non-yielding, deep-seated survey reference point.

Vibrating Wire Inline Extensometer means a series of displacement transducers installed in a sampled borehole for the purposes of measuring settlement corresponding to depth

Vibrating Wire Piezometer means a sensor attached to a cable installed in a borehole for the purposes of measuring pore pressure response.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submission Requirements

4.1.1 Notification

The Contract Administrator shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of instruments.

4.1.2 Installation Methods

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system(s), monitoring enclosure(s), temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

5.0 MATERIALS

5.1 Materials for Temporary Benchmarks (TBM)

The Contractor shall supply all materials and equipment required for the installation of the Benchmarks.

5.1.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 7.2.2.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.1.2 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.1.3 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.1.4 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.1.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.2 Materials for Vibrating Wire Piezometer

The Contractor shall supply all materials and equipment required for the installation of the Vibrating Wire Piezometer.

5.2.1 Vibrating Wire Piezometer

The vibrating wire piezometer sensors shall be:

- Slope Indicator model 52611020 (-5 to 50 psi); or
- RST model VW2100-0.35; or
- Equal.

The VWPs shall be compatible with the Slope Indicator VW Minilogger, model 52613310, or equal. All VWPs shall be of the same make/supplier.

All VWPs shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

5.2.2 Signal Cable

The signal cable shall be:

- Slope Indicator model 50613524 cable; or
- RST model EL380004 cable; or
- Equal.

The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is sufficient length of signal cable for each piezometer to provide enough slack in the borehole and along the trenches until each cable is out of the embankment footprint area where they shall be protected from earthmoving equipment and extended to the monitoring station.

5.2.3 Bentonite

Bentonite to form borehole plugs as required shall be in accordance with OPSS 1205 in pellet form in sufficient quantity.

5.2.4 Filter Sand

Sand for filters around VWP sensors shall be clean washed sand, such as “Sakcrete” washed general-purpose sand; or similar.

5.2.5 Protective Surround

Protective casing as recommended by the manufacturer shall be provided over the length of the piezometers through the rock fill. Sand for additional protection around the casing shall be clean washed sand, such as “Sakcrete” washed general-purpose sand; or similar.

5.2.6 Grout

Grout shall be cement-bentonite mix consisting of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU - OPSS 1301).

5.2.7 Trench Burial and Conduit

The signal cable for each VWP shall be buried in a shallow trench at the locations indicated in Table 1C and taken out of the embankment footprint area if possible and/or to an area that will not be impacted by construction operations. Conduits to protect the signal cables in the trenches and above ground surface shall consist of Schedule 40 – 75 mm - 3" - steel pipe. A minimum 300 mm protective surround consisting of OPSS.PROV 1010 Granular ‘A’ in accordance OPSS.PROV 1010 shall be placed around the conduit. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

5.2.8 Data Acquisition System (Data Logger)

The signal cables from the vibrating wire piezometers shall be connected to the nearest data-logger. A minimum of one (1) data-loggers shall be installed at each abutment. The data acquisition systems shall be from the same supplier as the VWPs and shall consists of:

- Slope Indicator Model 56701000 (CR1000); or
- RST Model ELGL1200; or
- Equal.

The data-logger shall consist of the following:

- ENC 16/18 Water-proof Enclosure Model 56705020, Model ELF0638, or equal;
- SC32A Serial Interface (with RS232 transfer cable) Model 56704010, Model CS-SC32A, or equal;
- VW Interface Model 56701510 or 56701500, Model CS-AVW200, or equal;
- AM16/32 Multiplexer Model 56702110, Model ELGL2042, or equal;
- A suitable power supply which shall be able to last for 1 year);
- Cellular Modem
- LoggerNet Software Model 56708020, Model CS-Loggernet, or equal.

The data-loggers shall be programmed according to the following:

- Recording Software: VWP data shall be recorded six (6) times a day (i.e. one (1) reading every 4 hours); and,
- Test Software: once this program is transferred to the data-logger, the system shall be able to be tested to confirm readings can be gathered manually at the site and remotely by use of the cellular network.

The real-time data shall be retrieved remotely by cellular network. The Contractor shall be responsible for obtaining the cellular plan to allow for retrieval of the data by the cellular network for the duration of the construction.

5.2.9 Wooden Posts

Wooden posts for the support of the data acquisition system enclosures shall be:

- 100 mm x 100 mm (4"x4"), minimum 3 m (10') long pressured treated lumber.

5.3 Materials for Vibrating Wire Inline Extensometer (VWIX)

The Contractor shall supply all materials and equipment required for the installation of the Vibrating Wire Inline Extensometer.

5.3.1 Vibrating Wire Inline Extensometer Sensors

The vibrating wire inline extensometer sensors shall be:

- RST EXINLINE-1100; or
- Equal.

The VWIXs shall be compatible with the RST RSTAR L900 compatible data loggers, model CR300, or equal. The sensors shall be capable of measuring a total displacement of up to 150 mm. All VWIXs shall be of the same make/supplier.

All VWIXs shall be calibrated prior to installation and the calibration data for each instrument shall be provided to the Contract Administrator.

5.3.2 Anchors

At each abutment a minimum of seven (7) anchors shall be installed at each extensometer as follows:

- Six (6) Hydraulic Borros Anchors Model EXHY13000
- One (1) Groutable Anchors with Spring Legs Model EXIL12000

The locations of the extensometers and depths of the anchors are specified in Table 1D. Prior to the installation of the bottom anchor, soil samples shall be obtained as specified in Section 7.4.2 to confirm the bottom anchor is installed at least 3 m into the "100-blow" hard clayey silt till.

5.3.3 Signal Cable

The signal cable shall be:

- RST Model EL380004 cable; or
- Equal.

The length of cable for each extensometer shall be carefully estimated from the construction drawings to ensure that there is sufficient length of signal cable for each extensometer to provide enough slack in the borehole and along the trenches until each cable is out of the embankment footprint area where they shall be protected from earthmoving equipment and extended to the monitoring station.

5.3.4 Bentonite

Bentonite to form borehole plugs as required shall be in accordance with OPSS 1205 in pellet form in sufficient quantity.

5.3.5 Protective Surround

Protective casing as recommended by the manufacturer shall be provided over the length of the extensometer through the embankment fill. Sand for additional protection around the casing shall be clean washed sand, such as "Sakcrete" washed general-purpose sand; or similar.

5.3.6 Grout

Grout shall be of similar strength/consistency of the surrounding soils. Grout shall be cement-bentonite mix consisting of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU - OPSS 1301). The grout at the bottom anchor shall be density of $2 g/cm^3$.

5.3.7 Trench Burial and Conduit

The signal cable for each VWIX shall be buried in a shallow trench at the locations indicated in Table 1D, and taken out of the embankment footprint area if possible and/or to an area that will not be impacted by construction operations. Conduits to protect the signal cables in the trenches and above ground surface shall consist of Schedule 40 – 75 mm - 3" - steel pipe. A minimum 300 mm protective surround consisting of OPSS.PROV 1010 Granular 'A' shall be placed around the conduit. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

5.3.8 Data Acquisition System (Data Logger)

The signal cables from the VWIXs shall be connected to the nearest data logger. Two (2) dataloggers shall be installed; one at each abutment. The data acquisition systems shall be from the same supplier as the VWIXs and shall consists of:

- RST FlexDAQ (CR300); or
- Equal.

The data logger shall consist of the following:

- RST L900 RTU, or equal
  - RST Node Model DT2055
  - RST Data Logger Model CR300
  - Cellular Modem
- RST FlexDAQ weatherproof enclosure, or equal;
- A suitable power supply which shall be able to last for 1 year);
- LoggerNet Software Version 4.4.2, or equal.

The data-loggers shall be programmed according to the following:

- Recording Software: VWIX data shall be recorded six (6) times a day (i.e. one (1) reading every 4 hours); and,
- Test Software: once this program is transferred to the data-logger, the system shall be able to be tested to confirm readings can be gathered manually at the site and remotely by use of the cellular network.

The real-time data shall be retrieved remotely by cellular network. The contractor shall be responsible for obtaining the cellular plan to allow for retrieval of the data by the cellular network for the duration of the construction.

5.3.10 Wooden Posts

Wooden posts for the support of the data acquisition system enclosures shall be:

- 100 mm x 100 mm (4"x4"), minimum 3 m (10') long pressured treated lumber.

6.0 EQUIPMENT

6.1 Monitoring Equipment Operation and Weather Conditions

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring will be conducted potentially year-round by the Contract Administrator.

6.2 Data Logger

The Contractor shall submit a detailed proposal on the setup of the data-logging system (i.e. numbers and locations of the data-logging unit(s)) to the Contract Administrator for review, prior to ordering the data-logger(s).

7.0 CONSTRUCTION

7.1 Monitoring Instrument Installations

7.1.1 Drawings

Reference shall be made to the following drawings that are contained elsewhere in the Contract Documents:

- Monitoring Instrumentation Plans;
- Typical Monitoring Sections; and
- Typical Instrument Installation Details.

7.1.2 Quantities and Locations of Instruments

The quantities and approximate location of instruments are presented in Table 1A and are shown on the Contract Drawings. The final locations shall be "field fit" by the Contractor to take account of any utilities that may be present, construction operations, and safe access conditions.

Table 1A – Instrument Quantities and Locations

Monitoring Section[1]	**Quantities**			
	SP	**DSR**	**VWP**	**VWIX**
West Abutment Approach Embankment	3	--	3	1
East Abutment Approach Embankment	3	--	3	1
Gas main - W-S Ramp (Station 10+202)	2	2	--	--
Gas main - W-N Ramp and the S-E/W Ramp (Station 10+290)	2	2	--	--
TOTAL:	**10**	**4**	**6**	**2**

7.1.3 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted.

7.1.4 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

7.1.5 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

7.1.6 Marking and Labelling

The location of any above-ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls, if and where applicable.

Instruments shall be clearly labelled in the field, with each instrument having a unique identifier as contained in the other item-specific special provisions. The labelling shall remain legible for the entire duration of monitoring.

7.1.7 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced by the Contractor at no cost to the Owner or Contract Administrator.

7.1.8 Survey Personnel

Surveying to establish the benchmarks and other elevations shall be carried out by a registered surveyor with appropriate equipment. The surveyor shall be retained by the Contractor.

7.1.9 Accuracy of Surveying for Elevations

Elevations shall be surveyed to an accuracy of ± 2 mm or better.

7.1.10 Boreholes

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled for the installation of monitoring instruments. In situ or laboratory geotechnical testing is not required.

Boreholes shall be advanced using conventional drilling methods, where applicable, and shall be as straight and vertical as practicable.

7.1.11 Installation Program

The instruments shall be installed prior to the commencement of the embankment construction Table 1B gives a summary of the installation schedule requirements.

Table 1B – Instrument Installation Program

Instrument Type	Instrument Location	Start Installation	Finish Installation
SP	At ground surface after stripping, along alignment of gas main and beneath EPS embankment.	After stripping and prior to start of EPS embankment construction.	At completion of EPS embankment construction.
SP	Approach Embankment at the East and West Abutment.	After stripping and prior to start of approach embankment construction.	At completion of approach embankment construction.

Instrument Type	Instrument Location	Start Installation	Finish Installation
DSR	Offset from gas main and at the gas main invert depth.	Prior to start of EPS embankment construction.	At completion of EPS embankment construction.
VWP	Approach Embankment at the East and West Abutment.	Before start of embankment construction.	At completion of approach embankment construction.
VWIX	Approach Embankment at the East and West Abutment	Before start of embankment construction	At completion of approach embankment construction.

7.2 Benchmark Installation

7.2.1 Number and Locations

The minimum number and approximate locations of the Benchmarks are to be determined by the Contractor and his Foundation Engineering consultant in conjunction with the Contract Administrator, the Foundation Monitoring Consultant, and Surveyors. For bidding purposes assume that 6 benchmarks are required: 3 benchmarks anchored at 6 m depth and 3 benchmarks anchored at 15 m depth (through the existing embankments). The number and locations of Benchmarks shall be determined in the field to satisfy the following conditions:

- Direct sighting is possible from all instruments to at least one Benchmark.
- Each Benchmark is located in an area that will not experience a change in loading (due to grade raise or excavation) that could induce settlement or heave in the ground in which the Benchmark is installed (i.e. non-settling benchmark).
- Each Benchmark is located in such a way to minimize interference with and damage by construction activities.
- The rod anchor elevation shall be adjusted in the field to extend approximately 1 m into soils having Standard Penetration Test ‘N’ values of greater than 25 blows per 0.3 m of penetration. Reference shall be made to the Foundation Investigation Reports for information in order to determine the anchor elevation for each Benchmark location selected.

Intermediate tie-in points may be required as deemed necessary by the surveyor, and shall be tied into the temporary benchmarks during each reading.

7.2.2 Installation

The Contractor shall install Benchmarks in accordance with the following:

7.2.3 Borehole

The borehole shall be advanced to rod anchor elevations controlled by the Standard Penetration test “N” values given above, using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

7.2.4 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.2.5 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

7.2.6 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

7.2.7 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

7.3 Vibrating Wire Piezometer (VWP) Installation

7.3.1 Number and Locations

The locations of the VWP are shown in the Contract Documents and in Tables 1A to 1C. The VWP shall be installed in boreholes immediately prior to construction of the approach embankment at the abutments. The VWPs shall be installed to a tip elevation given in Table 1C. Installation of the VWPs shall be as per the manufacturer's recommendations in addition to what is stated or emphasised below.

The VWIX signal cables shall be extended to the data-logger enclosure areas through a metal or plastic conduit buried in trenches with protective surround, as specified in Section 5.2.5. The final location of the monitoring enclosure should be determined on-site prior to ordering instruments to ensure there is sufficient cable length(s). Due to the restricted working area, the location of the monitoring enclosure should be determined to avoid construction traffic.

Table 1C – Vibrating Wire Piezometer Locations and Elevations

Monitoring Location	Hwy 89 Station	Offset from Highway 89 Centreline	Approximate Elevation of Existing Ground Surface (m)	Tip Elevation (m)
Approach Embankment at	9+958	13 m LT	227.5	203.0
				199.0

Monitoring Location	Hwy 89 Station	Offset from Highway 89 Centreline	Approximate Elevation of Existing Ground Surface (m)	Tip Elevation (m)
East Abutment				186.0
Approach Embankment at West Abutment	10+042	13 m RT	227.2	202.0
				199.0
				188.0

7.3.2 Borehole Installation

The borehole at each VWP location shall be advanced to 300 mm below the lowest tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris. A split spoon sample shall be taken at the proposed installation depth to confirm the soil stratum at the VWP tip elevation. The borehole shall be filled with water prior to installation of the VWP tip.

7.3.3 Protective Enclosures for Data Loggers

The data-logger shall be installed in a protective enclosure near each approach embankment to prevent vandalism and prolonged wear-out of the data-loggers against extreme weather. The protective enclosure shall be lockable and weather proofed. The Contractor shall submit a detailed proposal of the protective enclosure (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction/installation.

The Contractor shall ensure access to the protective enclosure at all times, including but not limited to snow clearing in the winter.

7.3.4 Completion of Installation

It is known that the process of installing VWPs can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VWP shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures, for the period noted below until the value has stabilized as determined by the Contract Administrator. Stabilization shall be deemed to have occurred:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10 per cent of the anticipated hydrostatic value; and,
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5 per cent of the anticipated hydrostatic value.

The Contractor should be prepared to wait for a period of 10 days to 15 days after completion of installation of the instruments for the baseline readings to stabilize.

7.4 Vibrating Wire Inline Extensometers

7.4.1 General

The locations of the VWIX are as shown in Table 1D. Installation of the extensometers shall be as per the manufacturer's recommendations in addition to what is stated or emphasized below. The extensometers shall be installed in boreholes immediately prior to construction of the approach embankment at the abutments. The contractor shall ensure that appropriate care is taken while lowering the system in the borehole, by attaching it to a steel pipe or a wireline attached to the bottom anchor. Provisions shall be made to prevent applying excessive extension/pressure on the sensors during the installation process, to avoid breaking the pin that sets extension/compression.

The VWIX signal cables shall be extended to the data-logger enclosure areas through a metal or plastic conduit buried in trenches with protective surround, as specified in Section 5.2.7. The final location of the monitoring enclosure shall be determined on-site prior to ordering instruments to ensure there is sufficient cable length(s). Due to the restricted working area, the location of the monitoring enclosure shall be determined to avoid construction traffic.

Table 1D – Vibrating Wire Inline Extensometer Locations and Elevations

Monitoring Location	Hwy 89 Station	Offset from Centreline	Approximate Elevation of Existing Ground Surface (m)	Anchor Type[1,2]	Anchor Elevation (m)
Approach Embankment at East Abutment	9+958	centerline	227.5	HBA	227.0
				HBA	223.0
				HBA	217.0
				HBA	210.0
				HBA	203.0
				HBA	199.0
				GASL	178.0
Approach Embankment at West Abutment	10+042	centerline	227.2	HBA	227.0
				HBA	224.0
				HBA	217.0
				HBA	210.0
				HBA	202.0
				HBA	198.0
				GASL	175.0

Note: 1. HBA – Hydraulic Borros Anchor
2. GASL – Groutable Anchors with Spring Legs

7.4.2 Borehole Installation

The borehole at each VWIX location shall be advanced to 300 mm below the lowest anchor elevation using suitable drilling techniques. At each extensometer, a split-spoon sample shall be taken to confirm the appropriate soil stratum, as directed by the Contract Administrator's Foundation Consultant. Equipment to complete the sampling (e.g. Drill rig, 50 mm split-spoon sampler, hydraulic piston sampler) shall be provided. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris prior to sampling, testing and installation of the VWIXs.

7.4.3 Protective Enclosures for Data Loggers

The data-logger shall be installed in a protective enclosure near each approach embankment to prevent vandalism and prolonged wear-out of the data-loggers against extreme weather. The protective enclosure shall be lockable and weather proofed. The Contractor shall submit a detailed proposal of the protective enclosure (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction/installation.

The Contractor shall ensure access to the protective enclosure at all times, including but not limited to snow clearing in the winter.

7.5 Monitoring Program

7.5.1 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days after the completion of installation of Benchmarks, Settlement Plates, Deep Settlement Rods, Vibrating Wire Piezometers and Vibrating Wire Inline Extensometers.

7.5.2 Reporting

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within three (3) days of completion of installation of each instrument.

7.5.2.1 Temporary Survey Benchmarks

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- TBM Northing and Easting in MTM NAD 83 coordinates;
- Elevation of the rod anchor bottom, rod anchor length, and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions it encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

7.5.2.2 Settlement Plates and Deep Settlement Rods

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP and DSR Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP or DSR rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

7.5.2.3 Vibrating Wire Piezometers

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWP Northings and Eastings in MTM NAD 83 coordinates;
- Elevations of VW sensors in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Installation notes / sketches;
- Model, make and serial numbers of VW sensors, readout unit and signal cable; and,
- Calibration details of VW sensors.

7.5.2.4 Vibrating Wire Inline Extensometers

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWIX Northings and Eastings in MTM NAD 83 coordinates;
- Elevations of anchors and sensors in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Installation notes / sketches;
- Model, make and serial numbers of VWIX sensors, readout unit and signal cable; and,
- Calibration details of VWIX sensors.

7.5.3 Monitoring

The Contractor shall meet with the Contract Administrator and staff responsible for the ongoing monitoring immediately after installation of the instruments and before the start of embankment construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in the item-specific special provisions.

Monitoring by the Contract Administrator’s representative for the baseline readings shall commence within seven working days after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the construction of the embankments, and for up to approximately 6 weeks to 3 months following the completion of construction to the preload grade.

7.5.4 Decommissioning of Instruments

At the end of the monitoring period, the Contractor shall decommission all the temporary survey Benchmarks and tie-in points by removing the rod and friction-reducing sleeve to at least 1.5 m below grade by excavating and backfilling with compacted granular fill in accordance with the specifications for fill placement.

At the end of the monitoring period, the Contractor shall decommission all Settlement Plates, Deep Settlement Rods, Vibrating Wire Piezometers and Vibrating Wire Inline Extensometers, unless otherwise advised by the Contract Administrator. Decommissioning of instrumentation shall be carried out per the item-specific special provisions and according to the Ontario Water Resources Act, Regulation 903 (as amended).

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.1 Supply and Installation of Embankment Monitoring Equipment - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including the supply, installation and decommissioning of survey benchmarks, Vibrating Wire Piezometers and Vibrating Wire Inline Piezometers, as well as performing all required monitoring and reporting.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003 March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling – Structures, is amended as follows:

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 10 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, utilities, and structures, within a distance of 250 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

SETTLEMENT PLATES – Item No.

Special Provision

1.0 SCOPE

1.1 General Scope

This special provision contains the requirements for the supply and installation of Settlement Plates (SPs).

The purpose of the SPs is to monitor settlements at approximately the existing ground surface elevation above the alignment of the existing gas main prior to construction of the Expanded Polystyrene (EPS) embankment for the W-S Ramp and the combined embankment for the W-N Ramp and the S-E/W Ramp.

In addition, Settlement Plates are also required to be installed prior to the construction of the approach embankments which are to be preloaded prior to driving of the steel H-piles at the abutments. The completed, preloaded embankments at the abutments shall remain undisturbed until such time as the monitoring indicates that a sufficient degree of compression of the foundation soil has been achieved.

Settlement is measured by survey of the top of the rod with reference to stable, non-settling Benchmarks. The settlement readings are intended to confirm that the settlements at the gas main level are maintained at less than 15 mm.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

The Contractor shall supply all materials and equipment required for the installation of the SPs.

5.1 Plate

The Contractor shall supply a steel plate with a thickness of at least 6.35 mm. The plate shall be at least 0.5 m wide by 0.5 m long.

5.2 Rod

The SP rod shall be fixed to the centre of the plate and perpendicular to the plate. The coupling of the rods shall be such that all sections have the same axis and that no separation or contraction will occur at the couplings.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.3 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.4 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.6 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.7 Extension of Rod

The SP rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

5.7 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and friction-reducing sleeve within the backfill/embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the friction-reducing sleeve shall be filled with medium to coarse sand.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.1 Installation

7.1.1 General Procedure

As embankment construction proceeds, the rods shall be extended above the new top of embankment. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

7.1.2 Location

The Contractor shall install SPs at the locations shown on the Contract Drawings and given in Table 1. The instrument locations should be field fit to avoid the Contractor’s operations, but to be as close to the intended locations as practicable.

Table 1 – Settlement Plate Locations

Monitoring Section[1]	Point ID	Approx. Station/Offset	Approx. Elevation of Ground Surface[2] (m)
Gas main - W-S Ramp	SP1	10+197 RSH	227.0
Gas main - W-S Ramp	SP2	10+197 LSH	227.0
Gas main - W-N Ramp	SP3	10+130 LSH	228.5
Gas main - S-E/W Ramp	SP4	10+295 RSH	228.5
TOTAL:	**4**		

1. Station referenced to Ramp centreline.
2. Ground surface elevation estimated following completion of topsoil removal and prior to embankment construction.

The Contractor shall install SPs as shown on the Contract Drawings and the Typical Installation Detail, in addition to what is stated below.

7.1.3 Plate

The settlement plate shall be installed horizontally on the undisturbed native soil or existing embankment fill just below the existing ground surface.

7.1.4 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.1.5 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the SP rod shall extend 25 mm above the top of the friction sleeve at all times.

7.1.6 Extension of Rod

The SP rods shall be extended upwards as the embankment widening is constructed so that the top of the rod is always at least 0.3 m, but not more than 2 m above the surrounding fill.

7.1.7 Protective Surround

The CSP, friction-reducing sleeve and sand surround shall be extended concurrent with the rods, where applicable. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

7.1.8 Miscellaneous Installation Details

The elevation, northing and easting of the top of the rod shall be surveyed by the Contractor.

The total distance from the rod anchor to the top of the rod shall be measured and recorded by the Contractor to an accuracy of ± 2 mm or better.

The Contractor is responsible for preventing damage to the settlement plate during the embankment construction. If the rod is damaged during fill placement, the rods, friction-reducing sleeve and protective surround shall be replaced before resuming the fill placement.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement for payment will be made on the basis of the number of units of SPs installed, including extension through the embankment construction.

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including all appurtenances and extension through the embankment construction.

SETTLEMENT RODS – Item No.

Special Provision

1.0 SCOPE

1.1 General Scope

This special provision contains the requirements for the supply and installation of deep Settlement Rods (DSRs).

The purpose of the DSRs is to monitor settlements at approximately the invert elevation of the existing gas main prior to construction of the EPS embankment for the W-S Ramp and the combined embankment for the W-N Ramp and the S-E/W Ramp.

Settlement is measured by survey of the top of the rod with reference to stable, non-settling Benchmarks. The settlement readings are intended to confirm that the settlements at the gas main are maintained at less than 15 mm.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

The Contractor shall supply all materials and equipment required for the installation of the DSRs.

5.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 1.3.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.2 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.3 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.4 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.6 Extension of Rod

The DSR rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

5.7 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and friction-reducing sleeve within the backfill/embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the friction-reducing sleeve shall be filled with medium to coarse sand.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.1 Installation

7.1.1 General Procedure

As embankment construction proceeds, the rods shall be extended above the new top of embankment. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

7.1.2 Location

The Contractor shall install DSRs at the locations shown on the Contract Drawings and given in Table 1. The instrument locations should be field fit to avoid the Contractor's operations, but to be as close to the intended locations as practicable.

Table 1 – Settlement Rod Locations

Monitoring Section[1]	Point ID	Approx. Station/Offset	Approx. Elevation of Ground Surface[2] (m)	Estimated Elevation of Rod Anchor[3] (m)
Gas main - W-S Ramp	DSR1	10+197 RSH	227.0	224.4
Gas main - W-S Ramp	DSR2	10+197 LSH	227.0	224.4
Gas main - W-N Ramp	DSR3	13+130 LSH	228.5	224.4

Monitoring Section[1]	Point ID	Approx. Station/Offset	Approx. Elevation of Ground Surface[2] (m)	Estimated Elevation of Rod Anchor[3] (m)
Gas main - S-E/W Ramp	DSR4	10+295 RSH	228.5	224.4
TOTAL:	**4**			

1. Station referenced to Ramp centreline.
2. Ground surface elevation estimated following completion of subexcavation and backfill operation, prior to embankment construction.
3. Based on invert elevation of gas main from As-Laid drawings, to be confirmed by hydrovaccing.

The Contractor shall install DSRs as shown on the Contract Drawings and the Typical Installation Detail, in addition to what is stated below.

7.1.3 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.1.4 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

7.1.5 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

7.1.6 Extension of Rod

The SP rods shall be extended upwards as the embankment widening is constructed so that the top of the rod is always at least 0.3 m, but not more than 2 m above the surrounding fill.

7.1.7 Protective Surround

The CSP, friction-reducing sleeve and sand surround shall be extended concurrent with the rods, where applicable. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

7.1.7 Miscellaneous Installation Details

The elevation, northing and easting of the top of the rod shall be surveyed by the Contractor.

The total distance from the rod anchor to the top of the rod shall be measured and recorded by the Contractor to an accuracy of ± 2 mm or better.

The Contractor is responsible for preventing damage to the settlement rod during the fill placement process and wall construction. If the rod is damaged during fill placement, the rods, friction-reducing sleeve and protective surround shall be replaced before resuming the fill placement.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement for payment will be made on the basis of the number of units of DSRs installed, including extension through the embankment construction.

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including all appurtenances and extension through the embankment costruction.

PROTECTION SYSTEM – Item No.

Special Provision

Amendment to OPSS.PROV 539, November 2014

593.07.02 Removal of Protection Systems

Subsection 539.07.02 of OPSS.PROV 539 is deleted in its entirety and replaced with the following:

Protection systems shall be removed from the right-of-way unless it is specified in the Contract Documents that the protection system may be left in place.

Where piles are left in place, the top shall be removed to at least 1.5 m below the finished grade or ground level.

The method and sequence of removal shall be such that there shall be no damage to the new work, existing work and facility being protected.

All disturbed areas shall be restored to an equivalent or better condition than existing prior to the commencement of construction.

<u>VIBRATION MONITORING</u> – Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

This special provision describes requirements for vibration monitoring during the installation of deep foundations associated with the Highway 400/89 underpass, installation of temporary protection systems at Culvert Structure Nos. 30-399/C and 30-568/C and at proposed Culvert C-21.

2.0 REFERENCES

The subsurface conditions at the site are described in the following Foundation Investigation Reports:

Culvert Extensions (Structure Site Nos. 30-568/C and 30-399/C) for Reconstruction of Highway 400/89 Interchange
Town Of Innisfil, Simcoe County
Ministry Of Transportation, Ontario
G.W.P. 2438-13-00

Highway 400/89 Underpass Replacement Structure Site No. 30-256, Reconstruction of Highway 400/89 Interchange
Town Of Innisfil, Simcoe County
Ministry Of Transportation, Ontario
G.W.P. 2438-13-00

High Fill Embankment Highway 400/89 Interchange Reconstruction Town of Innisfil, Simcoe County
Ministry Of Transportation, Ontario
G.W.P. 2438-13-00

Proposed Culvert C-21, Highway 400/89 Interchange Reconstruction Town of Innisfil, Simcoe County
Ministry Of Transportation, Ontario
G.W.P. 2438-13-00

3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Contractor's Engineer means an Engineer with a minimum of five (5) years' experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the Contract. The Contractor's Engineer shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.
d) Proposed location of instruments adjacent to the on the residences, utilities, wells, or other potentially vibration-sensitive structures within a 350 m radius from Highway 89 underpass, culverts and protection system(s), as applicable.
e) Action plan to be taken to adjust protection system installation methods if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within 350 m of the Highway 400/89 underpass, culverts and for each protection system location.

7.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure/well within a 350 m radius of the Highway 400/89 underpass, culverts and each protection system on the project, shall be completed a minimum of two (2) weeks prior to commencement of installation of the protection system(s). Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of temporary protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

a) Type of structure, including type of construction and if possible, the date when built.
b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

7.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey at each structure within a 350 m radius of the bridge, culvert or protection system(s), is required within two (2) months of completion of the installation of deep foundations for the Highway 400/89 underpass, and protection systems on this contract.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations and protection systems.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface in the vicinity of each foundation element or protection system, and on the ground surface at radial distances of 25 m, 50 m, and 100 m from the foundation element or protection system locations at the bridge site(s) or culvert site(s). The Contractor shall take readings continuously during deep foundation installation and during installation of temporary protection systems, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on private structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities, if applicable, shall not exceed 10 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

7.3 Records

The Contractor/Contractor’s Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

a) The time/duration of each reading.
b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.
c) Details of exceedances and modifications to operations.
d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material required to do the work.

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab for the pile cap at the pier, Culvert Structure Nos. 30-399/C and 30-568/C and proposed Culvert C-21 crossing under Highway 89 west of Highway 400 underpass.

2.0 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction
OPSS 902 Excavating and Backfilling - Structures

3.0 Definitions - Not Used

4.0 Design and Submission Requirements - Not Used

5.0 Materials

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement for Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01 July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: 44.204167	Longitude: -79.654167

Temporary Flow Passage Systems

Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m^3/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
30-399C – extension (3 barrel Culvert 44 – Innisfil Creek)	2	14.5	-	26.4	31.6	Yes
30-568C – replacement (Culvert 14 – under ramp N-E/W)	2	1.2	1.6	2.0	3.2	Yes
Culvert C-21, Highway 89 Station 9+946	2	1.6	2.2	2.7	3.4	Yes

Dewatering Systems

Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)	Design Engineer Requirements (Note 1)
30-399C – extension (3 barrel Culvert 44 – Innisfil Creek)	100	Yes
30-568C – replacement (Culvert 14 – under ramp N-E/W)	100	Yes
Culvert C-21, Highway 89 Station 9+946	100	Yes

Note:

1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. “N/A” indicates a preconstruction survey is not required.

STABILITY OF EXCAVATION BASE – Item No.

Notice to Contractor

The Contactor shall be alerted to the groundwater elevation at site. The subsurface conditions at the site are described in the following reports:

- Foundation Investigation Report – High Fill Embankment, Highway 400/89 Interchange Reconstruction, Town of Innisfil, Simcoe County, G.W.P. 2483-13-00
- Foundation Investigation Report – Highway 400/89 Underpass Replacement, Structure Site No. 30-256, Reconstruction of Highway 400/89 Interchange, Town of Innisfil, Simcoe County, G.W.P. 2483-13-00
- Foundation Investigation Report – Culvert Extensions (Structure Site Nos. 30-399/C And 30-568/C), Reconstruction of Highway 400/89 Interchange, Town of Innisfil, Simcoe County, G.W.P. 2483-13-00
- Foundation Investigation Report – Proposed Culvert C-21, Highway 400/89 Interchange Reconstruction, Town of Innisfil, Simcoe County, G.W.P. 2483-13-00

The fill and native silt to silt and sand deposits present below the groundwater level will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. Further, the bedding from adjacent utilities may act as a conduit for subsurface water flow. The Contractor is to design and install an appropriate excavation protection and dewatering system to enable construction of the culverts and pile cap at the pier foundation unit to prevent disturbance to the founding soils. Lowering of the groundwater level to 1 m below the underside the base of the excavation for the culverts and for the pile cap for the pier foundation unit will be required prior to placing of heavy machinery inside the excavation. Alternatively, a tremie plug may be constructed, or other suitable method of stabilizing the base of the excavation.

The dewatering system design shall be completed by a design Engineer and design-checking Engineer, both of whom shall have a minimum 5 years experience in designing systems of similar nature and scope to the required work.

OBSTRUCTIONS

Notice to Contractor

The Contactor shall be alerted to the potential presence of cobbles and boulders within the fill and native soils and to the potential presence of cobbles, boulders, brick and asphalt fragments within the fill along the alignment of Culvert C-2.

Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for open cut excavations, and installation of temporary protection systems.

PRELOAD PERIOD – Approach Embankment at East and West Abutments for Underpass

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Notice to Contractor

The Contractor shall schedule his operation to include preloading of the full embankment height at the east and west abutments for the underpass at Highway 400/89. The area to be preloaded must extend a distance of 20 m behind the abutments and the side slopes are to be inclined at two horizontal to one vertical (2H:1V). The full height embankment shall remain in place for a minimum period of two months, prior to driving the piles at the abutments.

The Contractor shall not proceed with removal of the preloaded full height embankment and driving of the piles until approval has been given by the Contract Administrator.

PAVING OF REALIGNED HIGHWAY 89 AND RAMPS– Item No.

Notice to Contractor

Following completion of re-aligned Highway 89 and ramps and prior to placement of the pavement structure granular base material and paving, the Contractor shall conduct a survey to determine the elevation of the top of the granular sub-base material, and shall place additional granular sub-base material as and where required to achieve the pavement design sub-base elevation.

The Contractor shall not proceed with final granular placement and paving until approval has been given by the Contract Administrator.

MINISTRY OF TRANSPORTATION, ONTARIO

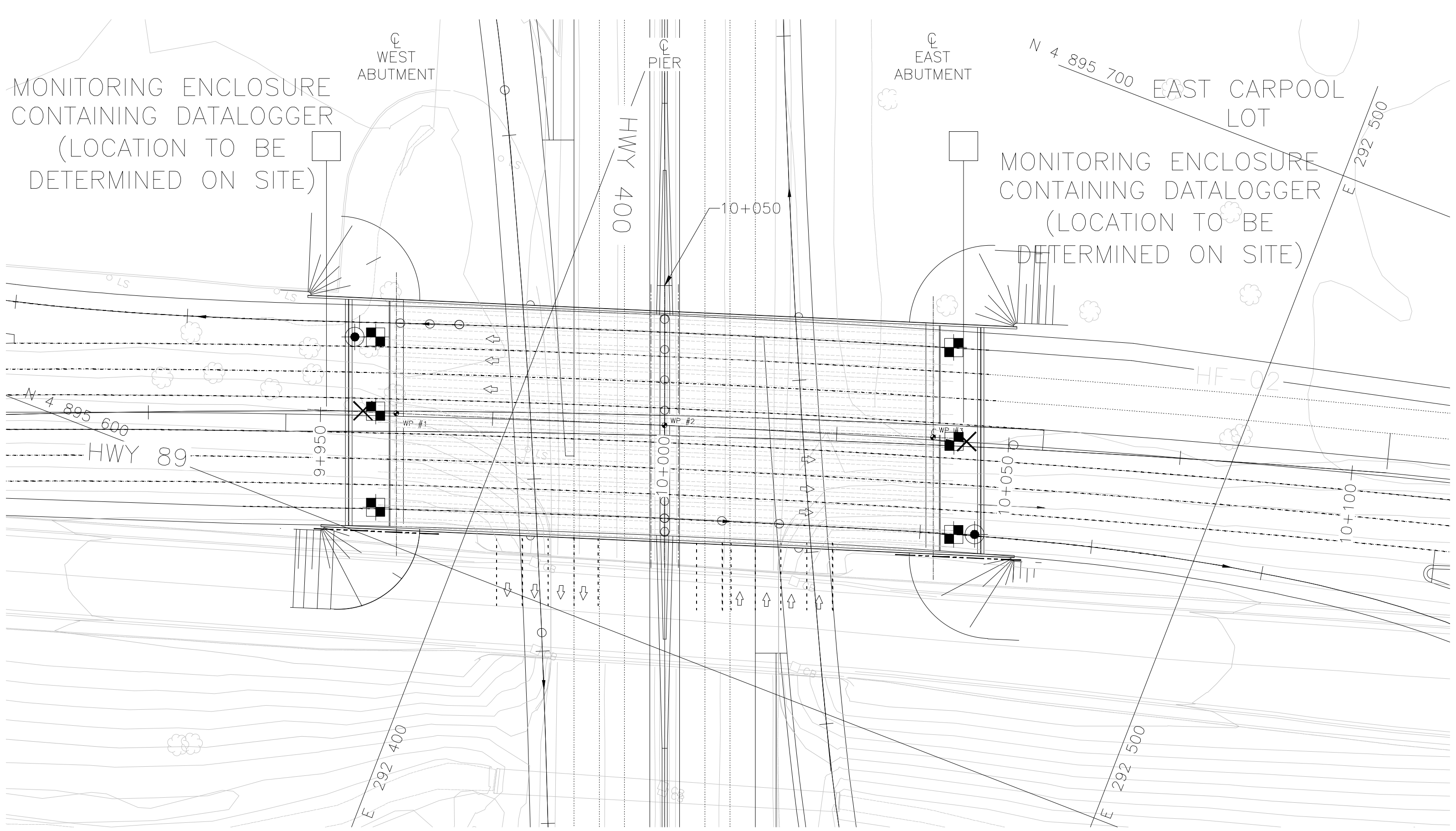
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2018–2024
GWP No. 2438–13–00

HIGHWAY 400 AND HIGHWAY 89 INTERCHANGE
EAST AND WEST ABUTMENT
MONITORING INSTRUMENTATION
PLAN

SHEET

Golder Associates

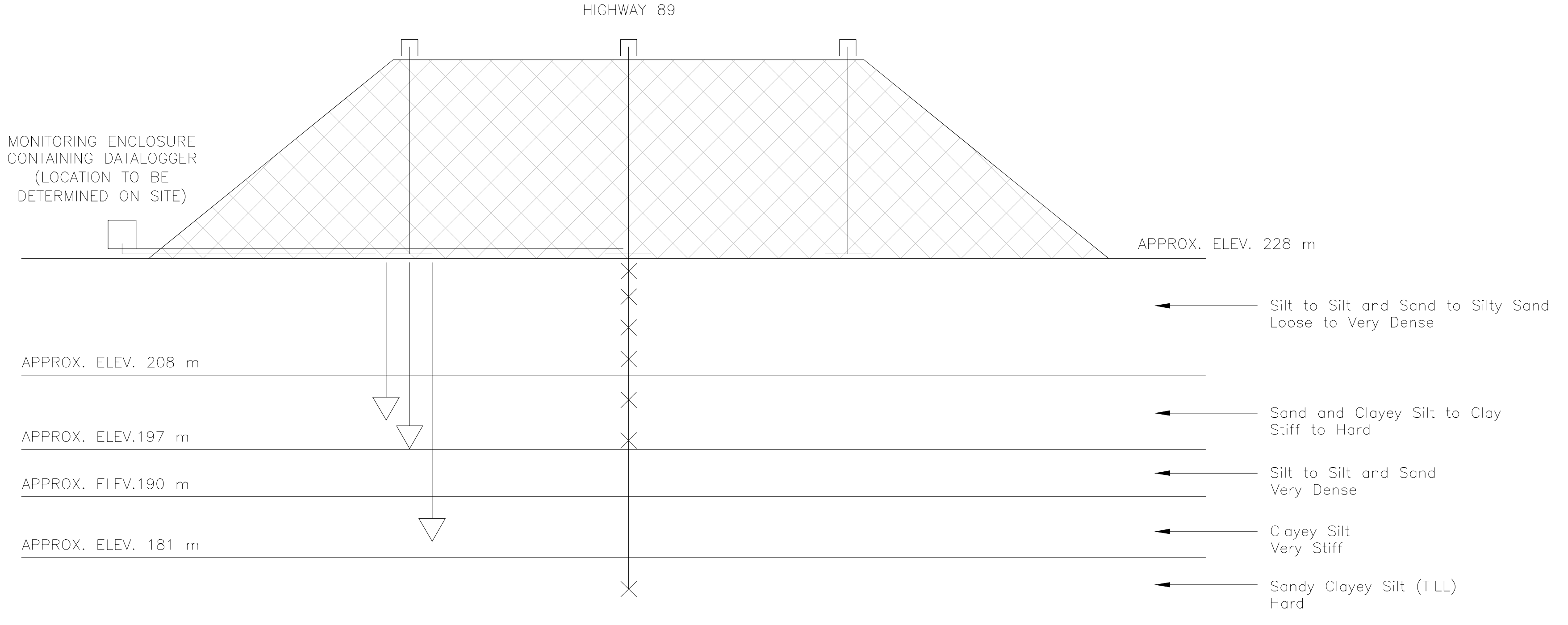


PLAN
SCALE

KEY PLAN
SCALE

LEGEND

▚	Settlement Plate (SP) (PLAN)
⊙	Vibrating Wire Inline Piezometer (VWP) (PLAN)
×	Vibrating Wire Inline Extensometer (VWIX) (PLAN)
⊓	Settlement Plate (SP) (SECTION)
▽	Vibrating Wire Inline Piezometer (VWP) (SECTION)
×	Vibrating Wire Inline Extensometer (VWIX) (SECTION)
▨	Preloading



TYPICAL MONITORING SECTION
HWY 89 EAST AND WEST ABUTMENT
NOT TO SCALE

L.C. COYNE
Nov. 22, 2018

S. McGaghran
Nov. 22, 2018

NOTES

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

See specification for location, depth and number of instruments.

Subsurface strata shown for illustration purposes only. For detailed subsurface information see Foundation Investigation Report.

REFERENCE

Base plans provided in digital format by Morrison Hershfield, received May 26, 2017.

PDR Allignment provided in digital format by Morrison Hershfield, drawing file "PDR_Allignment.dwg", received May 31, 2017.

Bridge GA provided by Morrison Hershfield, drawing file "1170121–01.dwg", received Sept. 19, 2017.

NO.	DATE	BY	REVISION

Geocres No. 31D–702

HWY. 400/89		PROJECT NO. 1668512	DIST. CENTRAL
SUBM'D. SMM	CHKD. SMM	DATE: 11/22/2018	SITE:
DRAWN: SMD/DD	CHKD. SMM	APPD. LCC	DWG. E–1

PLOT DATE: November 22, 2018
FILENAME: S:\Clients\MTO\Hwy_400_and_Hwy_89_Interchange\99_PROJ\1668512_Detailed_Design\40_PROD\0003_Underpass\1668512–0003–BG–0003.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

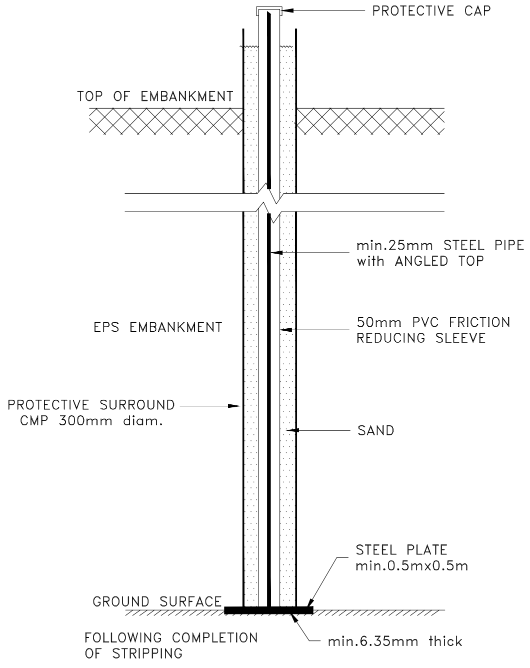
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2108–2024
GWP No. 2438–13–00

HWY 400 AND HWY 89 INTERCHANGE
HIGH FILL EMBANKMENTS AND APPROACH EMBANKMENT
TYPICAL INSTRUMENT INSTALLATION
DETAILS

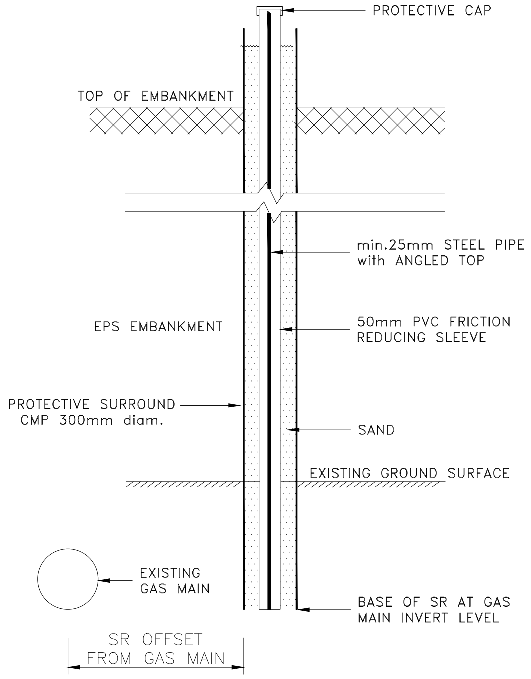
SHEET

Golder Associates



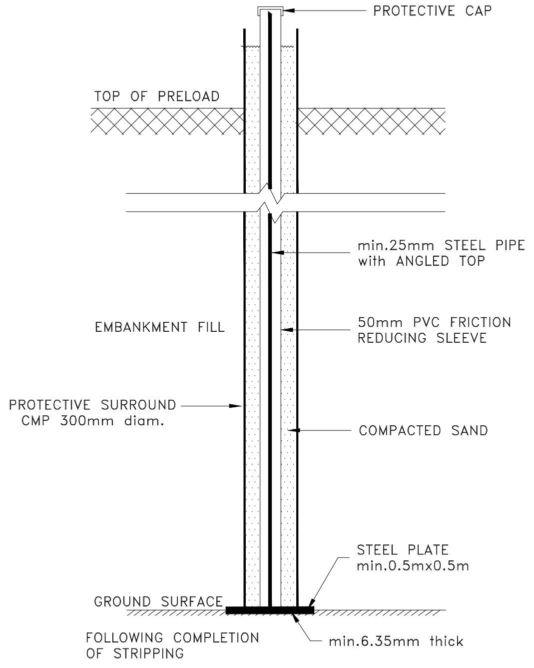
SETTLEMENT PLATE (SP)
RAMP W–S,
RAMP S–E/W and W–N

NOT TO SCALE



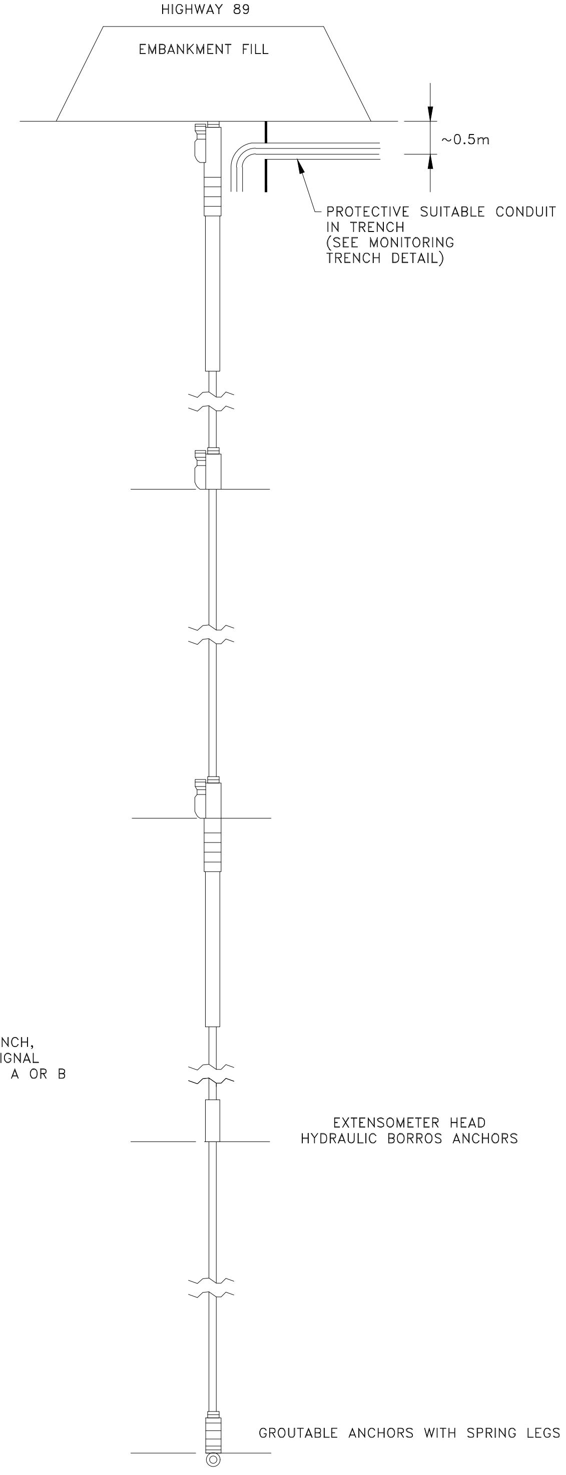
SETTLEMENT ROD (SR)
RAMP W–S,
RAMP S–E/W and W–N

NOT TO SCALE



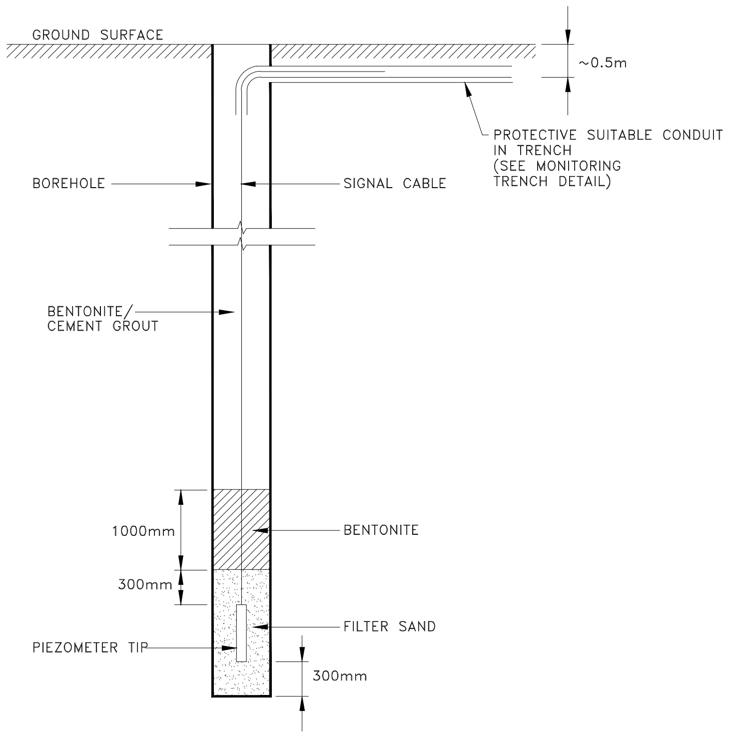
SETTLEMENT PLATE (SP)
EAST AND WEST
APPROACH EMBANKMENT

NOT TO SCALE



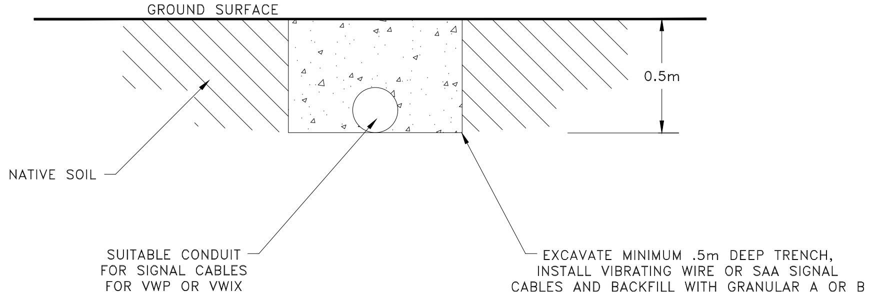
VIBRATING WIRE INLINE EXTENSOMETER (VWIX)
EAST AND WEST APPROACH EMBANKMENT

NOT TO SCALE



VIBRATING WIRE PIEZOMETER (VWP)

NOT TO SCALE



MONITORING TRENCH

NOT TO SCALE

L.C. COYNE
Nov. 22, 2018

S. McGaghran
Nov. 22, 2018

NOTES

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

NO.	DATE	BY	REVISION

Geocres No. 31D–702

HWY. 400/89		PROJECT NO. 1668512	DIST. CENTRAL
SUBM'D. SMM	CHKD. SMM	DATE: 11/21/2018	SITE:
DRAWN: SMD/DD	CHKD. SMM	APPD. LCC	DWG. E–2

PLOT DATE: November 22, 2018
FILENAME: S:\Clients\MTO\Hwy_400_and_Hwy_89_Interchange\99_PROJ\1668512_Detailed_Design\40_PROD\0003_Underpass\1668512-0003-BG-0004.dwg

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