October 20, 2016

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

ANNE STREET UNDERPASS, SITE NO. 30-347
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
W.O. 06-20016

Submitted to:
AECOM
30 Leek Crescent, 4th Floor
Richmond Hill, Ontario
L4B 4N4

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REPORT

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (now AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the preliminary design for the replacement of the Anne Street Underpass in the City of Barrie. The proposed work is part of the preliminary and design-build ready design associated with the Highway 400 widening from 1 km south of Highway 89 to the junction of Highway 11 in Simcoe County, Ontario.

This report addresses the proposed replacement of the Anne Street Underpass (MTO Structure Site No. 30-347) and the associated approach embankments only.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2013. Golder's scope of work for foundation engineering services associated with the Anne Street Underpass replacement is contained in Section 5.8 of AECOM's (previously URS Canada) Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated January 20, 2014.

2.0 SITE DESCRIPTION

The Highway 400/Anne Street Underpass is located in the city of Barrie, Ontario and the existing bridge structure is a two-span concrete rigid frame supported on driven H-piles. The total length of the bridge is approximately 36 m measured along the centerline of Anne Street between abutments, and the total deck width is 17 m measured between fasciae.

The overall surface topography in the vicinity of the site is relatively flat and consists of both residential and commercial areas to the east and west of Highway 400. Anne Street has been constructed in fill with approach embankments up to about 7 m high at an existing grade between about Elevations 240.7 m and 242.5 m adjacent to the east and west abutments, respectively. The Highway 400 grade at Anne Street is at about Elevation 236 m, rising toward the north.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

A subsurface investigation was carried out at this site for the Department of Highways, Ontario (DHO) in June and July, 1957, by Universal Geotechnique Limited (GEOCRES No. 31D-182). At that time, a total of six boreholes were advanced in the vicinity of the abutments and pier for the then-proposed structure. Boreholes 1 and 2 were located in the vicinity of the east abutment, Boreholes 3 and 4 were drilled near the west abutment, and Boreholes 5 and 6 were advanced at the approximate location of the central pier. The boreholes were advanced to depths ranging between about 7.6 m and 18.7 m. All of the boreholes were advanced from approximately the Highway 400 grade and the locations are shown on Drawing 1.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open borehole were observed during and following the drilling operations. There are no laboratory test results provided with the 1957 investigation report.

The borehole locations in MTM NAD83 northing and easting coordinates have been estimated from the plotted locations on the Digital Terrain Model base plan, and, together with the ground surface elevations referenced to Geodetic datum and drilled depths are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
1	4,916,194.1	288,232.1	235.3	7.6
2	4,916,212.5	288,238.4	235.5	18.7
3	4,916,225.7	288,212.6	235.8	15.2
4	4,916,241.8	288,219.9	236.0	7.8
5	4,916,209.2	288,222.6	235.1	7.6
6	4,916,226.7	288,229.7	235.5	7.8

3.2 Current Borehole Investigation

The field work at the site of the Anne Street Underpass was carried out on March 29 and April 20 and 21, 2016 during which time two boreholes were advanced to supplement the existing subsurface information. The Record of Borehole sheets are presented in Appendix A. The locations of these boreholes are shown in plan on Drawing 1 and in profile / cross section on Drawings 1 and 2.

The borehole investigation was carried out using a Diedrich D-90 truck-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 210 mm outside diameter hollow stem augers. Soil samples were generally obtained at intervals of depth about 0.75 m and 1.5 m, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions and water level in the open boreholes were observed during and immediately following the completion of drilling operations. The boreholes were backfilled upon completion of drilling in accordance with Ontario Regulation 903 (as amended), and the pavement was reinstated using dry mix concrete and cold patch asphalt.

The field work was observed by members of Golder's engineering 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 and cared for 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 laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected soil samples. The results of the laboratory testing are included in Appendix B.

The as-drilled borehole locations were measured relative to the existing on-site features shown on the Digital Terrain Model (DTM) for the site, and the ground surface elevations were interpolated from the topographic data provided by AECOM. The borehole locations provided on the borehole records and shown in plan on Drawing 1 and in profile / cross section on Drawing 2 are given using MTM NAD83 northing and easting coordinates, and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
AS1-1	4,916,217.8	288,209.9	236.0	18.8
AS1-2	4,916,185.3	288,241.7	240.5	18.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*[1], this section of Highway 400 from 6 km south of Highway 89 to the junction of Highway 11 traverses, generally in a south–north direction, the following physiographic regions: the Peterborough Drumlin Field; the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Peterborough Drumlin Field is present from the southern limit of the project site to south of Line 13 of the Township of Bradford West Gwillimbury, as well as between about 1 km north of Highway 89 to about Essa Road. The Simcoe Lowlands covers the area from south of Line 13 to approximately 1 km north of Highway 89 and from about Essa Road to just north of Anne Street. The Simcoe Uplands extends from just north of Anne Street to beyond the northern limit of this project site.

The surficial soils in the Peterborough Drumlin Field, consist primarily of gravelly sand till or sand and gravel deposits. Deposits of silt, clay or peat may also be found in the low-lying areas between drumlins and eskers.

Along Highway 400, the Simcoe Lowlands include: the Holland River valley; the lowlands of the Lake Simcoe basin to the east; the lowlands of the Nottawasaga basin to the west, which includes Innisfil Creek and the Nottawasaga River to the south and west of the project limits, respectively. The Lake Simcoe and Nottawasaga basins are connected by a flat floored valley through Barrie which extends from the shores of Kempenfelt Bay west generally along Highway 90. The Simcoe Lowlands are generally characterized by deep deposits of deltaic or lacustrine silts, sands and clays associated with glacial Lake Algonquin.

The Simcoe Uplands consist of till plains and ancient shorelines. The till deposits range from clayey to silty and generally become more sandy and containing more boulders in the north. The low-lying areas of this region may also contain shallow deposits of sand and gravel associated with former glacial lake shorelines.

4.2 Subsurface Conditions

The subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation, together with the results of in situ and laboratory testing, are presented on the Record of Borehole sheets and laboratory test summary figures provided in Appendices A and B, respectively. The Record of Borehole sheets from the previous investigation are presented in Appendix C. The interpreted stratigraphic profile and cross-sections are shown on Drawings 1 and 2.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross-sections are inferred from observations of drilling progress and non-continuous sampling and, therefore,

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

represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of a layer of asphalt (at boreholes drilled from the road platform) and non-cohesive fill material associated with the existing Highway 400 approach embankments, underlain by a deposit of sand, in places interlayered by silt, silty clay and sand and gravel layers / pockets.

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

4.2.1 Asphalt

An approximately 200 mm thick layer of asphalt was encountered at ground surface in Boreholes AS1-2.

4.2.2 Gravelly Sand to Sand and Gravel Fill

A 1.9 m and 3.9 m thick deposit of fill comprised of gravelly sand, trace to some clay containing wood fragment, to sand and gravel was encountered at ground surface at about Elevation 236.0 in Borehole AS1-1 and below the asphalt in Borehole AS1-2 at about Elevation 240.3 m.

The measured Standard Penetration Test (SPT) ‘N’-values measured within the fill deposit range from 4 blows to 31 blows per 0.3 m of penetration, indicating a loose to dense relative density.

The natural water content measured on two samples of the fill measured about 3 per cent and 8 per cent.

The grain size distributions of a sample of the gravelly sand portion of the fill material is shown on Figure B1 in Appendix B.

4.2.3 Silty Sand to Sand Fill

A 4.2 m thick deposit of fill comprised of silty sand, trace to some gravel, trace clay, to sand, some silt, trace gravel and containing organic silt layers was encountered below the sand and gravel fill in Borehole AS1-2 at about Elevation 236.4 m. The 1957 boreholes encountered between 2 m and 2.5 m of fill at ground surface comprised of sand containing gravel, clay, organics and wood fragments.

The SPT ‘N’-values measured within the fill deposit from the current investigation range from 12 blows to 29 blows per 0.3 m of penetration, indicating a compact relative density. The SPT ‘N’-values in the previous investigation range from 9 blows to 29 blows per 0.3 m of penetration, indicating that the fill has a loose to compact relative density.

The natural water content measured on a sample of the fill measured about 14 per cent and a moisture content measured on a sample of the organic silt measured about 77 per cent.

4.2.4 Silt to Sandy Silt to Sand

A non-cohesive deposit comprised primarily of sand silt to sand containing trace to some gravel, trace clay was encountered below the fill at all borehole locations between about Elevation 232.2 m and 236.0 m.. Pockets of silt clay were encountered within the sand deposit in Borehole AS1-1 as well as in Borehole 2 and 3. In addition, pockets or interlayers of silt as well as of sand and gravel were encountered within the sand deposit in Borehole 2 and 4.

The SPT 'N'-values measured within the non-cohesive deposit generally range from 11 blows to 130 blows per 0.3 m of penetration, indicating a compact to very dense relative density. It should be noted that one SPT 'N'-value measured 1 blow per 0.3 m of penetration measured in the sand deposit in Borehole AS1-1 and was likely caused by disturbed material as a result of the drilling operation near the groundwater level and is not considered a representative SPT 'N'-value of the deposit.

The natural water content measured on samples of this deposit taken during the current investigation ranges from about 14 per cent to 23 per cent and a natural water content measured on the silty clay layer measured about 25 per cent.

The grain size distributions of samples of the sand deposit and the silt to sandy silt interlayers from the current investigation are shown on Figures B2-1 and B2-2, respectively in Appendix B.

An Atterberg limits test carried out on a sample of the silty clay pocket measured a liquid limit of about 40 per cent, a plastic limit of about 15 per cent and a corresponding plastic index of about 25 per cent. The result of the Atterberg limits test, presented on Figure B3, indicates that the material is classified as a silty clay of intermediate plasticity.

4.3 Groundwater Conditions

The water level encountered during drilling and observed in Borehole AS1-2 upon completion of drilling for the current investigation is at about Elevation 231.9 m. The water levels observed in the open boreholes following completion of the 1957 investigation were measured at between Elevation 233.5 m and 234.5 m.

In should be noted that the water level observed in the open boreholes during and/or on completion of drilling may not represent the longer-term, stabilized groundwater level at the site. In addition, the water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This report was prepared by Mr. Billy Murphy, B.Eng., a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
C.C. NG
100086170
20/10/2016
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J. M. A. COSTA
Oct. 20/2016
PROVINCE OF ONTARIO

Christopher Ng, P.Eng.
Senior Geotechnical Engineer, Associate

Jorge M. A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant

BM/CN/JMAC/cn/mck

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
ANNE STREET UNDERPASS – SITE NO. 30-347
HIGHWAY 400 WIDENING
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6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Highway 400-Anne Street Underpass (MTO Structure Site No. 30-347). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the structure foundations, as may be required. The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the 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

Golder Associates Ltd. (Golder) has been retained by AECOM (formerly URS Canada Inc.) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspect for the preliminary design of the Highway 400-Anne Street Underpass in the City of Barrie. It is understood that the Anne Street Underpass will consist of a two-span, pre-cast girder bridge with 45 m span lengths. Further, a grade raise of up to 1.6 m is proposed at the approach embankments adjacent to the abutments.

Based on the General Arrangement (GA) Drawing provided by AECOM on June 23, 2016, the grade of the proposed Underpass is about Elevation 242.3 m and 243.9 m at the east and west abutments, respectively. In comparison, the proposed grade for Highway 400 is at about Elevation 236 m.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code (2014 CHBDC) and its Commentary, the proposed underpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical consequence level" associated with exceeding limits states design. In addition, given the limited level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, 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 "low degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and 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, as indicated in Sections 6.4 to 6.8.

6.3 Foundation Options

As part of the future widening of Highway 400 in Simcoe County, the existing Anne Street Underpass will require replacement. According to the available information, the existing two-span structure is supported on steel H-piles with the underside of the abutments and pile cap at approximately Elevation 232.5 m. Highway 400 is proposed to be widened by approximately 31 m to the west and 22 m to the east of the existing alignment. Based on the proposed underpass geometry and the subsurface conditions at this site, both shallow foundation and deep foundation options have been considered for support of the abutments and pier for the proposed structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the foundation alternative for a replacement structure is presented in Table 1.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the compact to very dense sand deposit or perched within the embankment fill, are feasible for support of the new abutments and centre pier, although this foundation type will preclude the use of integral abutments.
- **Deep foundations – driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and centre pier, and would permit design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only).
- **Deep foundations – drilled shaft (caissons):** Drilled shafts (caissons) are considered feasible for the support of the abutments and centre pier; however this option would preclude integral abutment design. This option would be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would be required. If caissons are adopted for support of the abutments temporary liners may be required during construction to control potential ground losses and/or disturbance of the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments and pier, although steel H-pile foundations are preferred from a foundations perspective for all foundation elements.

6.4 Shallow Foundations

6.4.1 Founding Elevation

For the support of the new abutments spread/strip footings should be founded on the compact to very dense sand deposit, or on compacted granular pads. Where spread/strip footings are to be founded on the native sand deposit, the highest founding elevations recommended for preliminary design of footings are:

Foundation Element	Highest Founding Elevation (m)	Founding Soil
West Abutment	234.0	Compact to Dense Sand (Inferred)
Centre Pier	234.0	Compact to Dense Sand
East Abutment	232.0	Compact to Dense Sand

6.4.2 Factored Geotechnical Axial Resistances

The following factored ultimate and serviceability geotechnical resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for preliminary design of spread/strip footing founded on the properly prepared sand, or on a compacted Granular 'A' pad having a minimum thickness of 1 m:

Foundation Alternative	Factored Ultimate Geotechnical Axial Resistance[1] (at ULS) (kPa)	Factored Serviceability Geotechnical Resistance[1] (at SLS) for 25 mm of Settlement (kPa)
Footing on properly prepared compact to very dense sand	700	150
Footing on minimum 1 m thick compacted Granular 'A' pad	750	175

Note: 1. The factored geotechnical resistances given above are estimated for a 3 m wide spread/strip footing.

The preliminary factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *CHBDC (2014)*.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the founding soils should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The following presents the coefficient of friction, tan φ', for the interface between the concrete footing and sand deposit or Granular 'A' pad:

Founding Material	Coefficient of Friction (tan φ')
Cast-in-place concrete footing on native compact to very dense sand	0.45
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

6.4.4 Frost Protection

All footings 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.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.5 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.5.1 Founding Elevation

The abutments for the replacement structure may be supported on steel H-piles or pipe piles driven to found within the dense to very dense sand deposit or very dense silt interlayer within the sand deposit.

Based on the GA Drawing, semi-integral abutments are proposed to be adopted for the design of the replacement structure with the abutments "perched" within the Anne Street embankments. The following pile tip elevations are recommended for preliminary design purposes:

Foundation Element	Proposed Underside of Pile Cap (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	236.9	218.0	Very Dense Silt (Inferred)
Centre Pier	233.0	218.0	Very Dense Silt
East Abutment	236.4	218.0	Very Dense Silt (or Inferred Sand)

Based on the above elevations, the proposed piles are estimated to be approximately 15.0 m to 18.9 m long at the west and east abutment.

6.5.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) for driven steel H-piles and closed-end, concrete-filled 324 mm (12-¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) are presented below.

Pile Type	Approximate Length of Driven Pile (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement [1] (kN)
HP 310x110	15.0 to 18.9	1,250 (Abutments) 1,150 (Centre Pier)	N/A
324 mm OD Pipe Pile	15.0 to 18.9	1,100 (Abutments) 1,000 (Centre Pier)	N/A

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

Pile installation should be in accordance with OPSS 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity.

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

6.5.3 Frost Protection

All pile caps 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 pile cap.

If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.6 Drilled Shaft (Caisson) Foundations

6.6.1 Founding Elevations

Drilled shafts (caissons) founded within the dense sand to very dense silt deposit may be considered for support of the abutments and centre pier for the proposed replacement structure. The following drilled shaft founding elevations may be used for preliminary design purposes:

Foundation Element	Proposed Underside of Pile Cap (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	236.9	219.0	Dense Sand / Very Dense Silt (Inferred)
Centre Pier	233.0	219.0	Dense Sand / Very Dense Silt
East Abutment	236.4	219.0	Dense Sand / Very Dense Silt (Inferred)

If drilled shaft foundations are adopted, a temporary liner will be required to support the overburden soils during construction to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.

6.6.2 Geotechnical Axial Resistance/Reaction

The following factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for design of drilled shaft (caisson) foundations:

Drilled Shaft Diameter (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement[1] (kN)
0.9	2,700 (Abutments) 2,650 (Centre Pier)	N/A
1.2	4,600 (Abutments) 4,500 (Centre Pier)	N/A

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

The preliminary factored geotechnical resistances provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of any additional subsurface investigation at the foundation elements.

6.6.3 Frost Protection

All pile caps 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 and perpendicular from the face of the abutment slope to the edge of the underside of the pile cap.

If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stem walls, and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, 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.

The following recommendations are made concerning the design of the abutment walls and associated retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. 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 sub-drains and frost taper should be in accordance with OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC (2014)* Section 6.12.3 and Figure 6.6. 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 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC 2014*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Soil Unit Weight	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

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). 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)*.

6.8 Approach Embankments

6.8.1 Subgrade Preparation and Embankment Construction

Based on the existing topographic information, the existing Anne Street embankment side slopes are inclined at about 2 horizontal to 1 vertical (2H:1V). For the proposed widening of the Highway 400 embankments, the new side slopes should also be constructed at a maximum inclination of 2H:1V. Where widening of the existing embankment occurs, benching the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes) to integrate the new fill into the existing slope fill.

It is understood that a 1.6 m grade raise of the existing/widened portion of the approach embankments is proposed. As indicated on OPSD 202.010 (Slope Flattening), a minimum 2 m wide bench should be incorporated into the approach embankment slopes where the slopes are equal to or greater than 8 m high, such that the uninterrupted slope height does not exceed 8 m.

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. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).

6.8.2 Embankment Stability and Factored Settlement

Limit equilibrium slope stability analyses for the embankment was carried out using the commercially available program Slide (version 6.0), developed by Rocscience Inc., employing the Morgenstern Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed for the critical embankment cross-section in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS 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, a target minimum FoS of 1.7 has been used for the design of the embankment slopes for temporary and permanent conditions, respectively, as per Table 6.2 of CHBDC (2014). The stability analyses assume that all organics and other deleterious materials are removed prior to constructing the approach embankments. Based on the results of the analysis for deep-seated global failure surfaces, the minimum FoS is equal to or greater than 1.7 and as such, stability issues are not anticipated within the limits of the approach embankment widening.

Settlement analyses were carried out using the commercially available program Settle3D (version 3.0), developed by Rocscience Inc. The factored settlement associated with the up to about 1.6 m of grade raise of the approach embankment is estimated to be about 65 mm. Given that the native subgrade deposit is primarily non-cohesive, it is expected that the majority of the settlement will occur during and shortly after reconstruction and raising of the embankment.

6.9 Construction Considerations

The following sections identify future construction considerations that may impact the future design and construction.

6.9.1 Open-Cut Excavations

The construction of new spread/strip footings and/or pile caps will require excavations to depths of up to about 8.5 m below the existing Anne Street grade and will be made through the existing embankment fill. The existing fill material is classified as a Type 3 soil above the water table, according to the Occupational Health and Safety Act (OHSA) and is considered a Type 4 soil below the water table. As such, temporary open-cut excavations should be made with side slopes no steeper than 1H:1V above the groundwater table and with side slopes no steeper than 3H:1V below the groundwater table.

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

6.9.2 Temporary Protection Systems

Temporary protection systems may be required to facilitate the removal of the existing bridge foundations and construction of the abutments and centre pier. 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.

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

6.9.3 Control of Groundwater

At the abutments, whether “perched” spread/strip footings or “perched” pile caps for deep foundations are adopted, the excavation bottom (i.e. founding level) should be maintained above the groundwater level. At the centre pier, however, excavations for the spread/strip footing or pile cap could extend about 1.5 m below the groundwater level at the site.

The soils at the base of the excavation at the pier location consist of a water-bearing, relatively permeable sand deposit, potentially containing silt/sand and gravel/silty clay pockets in places. At this preliminary stage, it is anticipated that an active dewatering system (beyond pumping from sumps within the excavation) will be required to lower the groundwater level. It is recommended that the groundwater level be lowered to not less than 0.5 m below the footing/pile cap founding level. An accurate estimate of the groundwater pumping volumes cannot be made at the preliminary design stage, as the flow rate would be dependent on whether the contractor includes an interlocking sheetpile cut-off wall and the duration for which the foundation excavation is open.

It is anticipated that the zone of influence for dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the silt to sand deposit. However, the potential for settlement impacts on the structure foundations and any adjacent utilities should be assessed at the detail design stage.

6.9.4 Ground and Groundwater Control for Drilled Shaft (Caisson) Construction

As noted in Section 6.6.1, running or flowing soil from the native non-cohesive deposits could occur during or after drilling the drilled shafts (caissons) and heave could occur at the caisson base. If drilled shaft foundations are adopted, temporary liners should be used to support the overburden soils. Balancing groundwater pressures during construction by utilizing a head of water or bentonite drilling slurry inside the temporary liner may be required, and should be assessed at the detail design stage. In addition, placement of concrete by tremie methods would be required.

6.10 Recommendations for Future Work During Detail Design

Given the variability of the strata encountered to the varying depths of the previous and current foundation investigation, it is recommended that, during detail design, additional site investigation and field testing be carried out at/within the footprint of the abutment and centre pier foundations to a sufficient depth below the ground surface. Such a foundation investigation will allow for a more specific assessment of the subsurface conditions at these locations and for design of deep foundations, both to a lower tip elevation and higher factored geotechnical resistances if required.

7.0 CLOSURE

This report was prepared by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

Christopher Ng, P.Eng.
Senior Geotechnical Engineer, Associate

Jorge M. A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant

BM/CN/JMAC/cn/mck

n:\active\2014\1111\14-1111-0002 urs - highway 400 tesr update - barrie\06 reports\8 - anne street underpass\final\14-1111-0002-8 rpt 16oct20 anne street underpass.docx

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Canadian Standards Association (CSA), 2014. *Canadian Highway Bridge Design Code* and *Commentary on CAN/CSA S6-14*. CSA Special Publication, S6.1-14.

Chapman, L. J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

Department of Highways, Ontario, 1957. *Report on Subsurface Exploration for Proposed Overpass at Anne Street and Highway 400, Barrie, Ontario*, GEOCRES No. 31D-182.

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Ministry of Transportation, Ontario, 2002. *Preliminary Foundation Investigation and Design Report Anne Street Underpass, Structure Site 30-347; Highway 400 Widening from 1 km South of Highway 89 to Highway 11, G.W.P. 30-95-00*, prepared by Golder Associates Ltd.

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Commercial Software:

Slide (Version 6.0) by Rocscience Inc.

Settle3D (Version 3.0) by Rocscience Inc.

Ministry of Transportation Ontario:

Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

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 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 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankments
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

TABLES

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	• Feasible for the support of new abutments and centre pier.	• Conventional excavation and construction techniques. • Lower cost compared to deep foundations. • Footings can be perched within the approach embankment fill so that subgrade excavation will not be required.	• Do not allow for integral abutment construction. • Likely requires temporary protection system to allow for excavation/footing construction at centre pier. • Groundwater control and temporary protection system likely required for construction of pier footing.	• Estimated cost is approximately $600/m^3 for construction of shallow foundations.	• Footing subgrade must be protected from frost penetration
Steel H-piles or pipe piles	• Feasible for the support of new abutments with pile cap “perched” within the approach embankments	• Conventional construction methods for H-pile or steel pipe pile foundations. • Steel H-piles allow for integral abutment configuration. • Pile cap can be constructed within the approach embankment fill so that subgrade excavation will be not be required.	• Piles may refuse above design tip elevation due to the very dense native overburden, especially pipe piles which have a larger displacement base. • Pipe piles not readily accepted for integral abutment construction; allow for semi-integral abutment configuration.	• Estimated cost is approximately $250/m length for pile installation and $600/m^3 for pile cap construction.	• Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving.
Drilled Shaft (Caissons)	• Feasible but not recommended for the support of abutments	• Abutment pile caps could be constructed at the underside of the bridge or maintained higher than spread footings, or H-pile caps, reducing depth of excavation and protection system requirements, or	• Temporary liners will be required, plus special measures such as tremie placement of concrete; likely not possible to inspect caisson base. • Precludes use of	• Estimated cost is approximately $1,000/m length for caisson installation and $600/m^3 for pile cap construction; the cost may be higher to account for the use of a temporary liner.	• Risk of loosening and leaving in place disturbing founding soils at base of caissons.

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
		caps can be constructed at level of underside of structure. • Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles.	integral abutments. • More expensive compared to shallow foundations.		

DRAWINGS

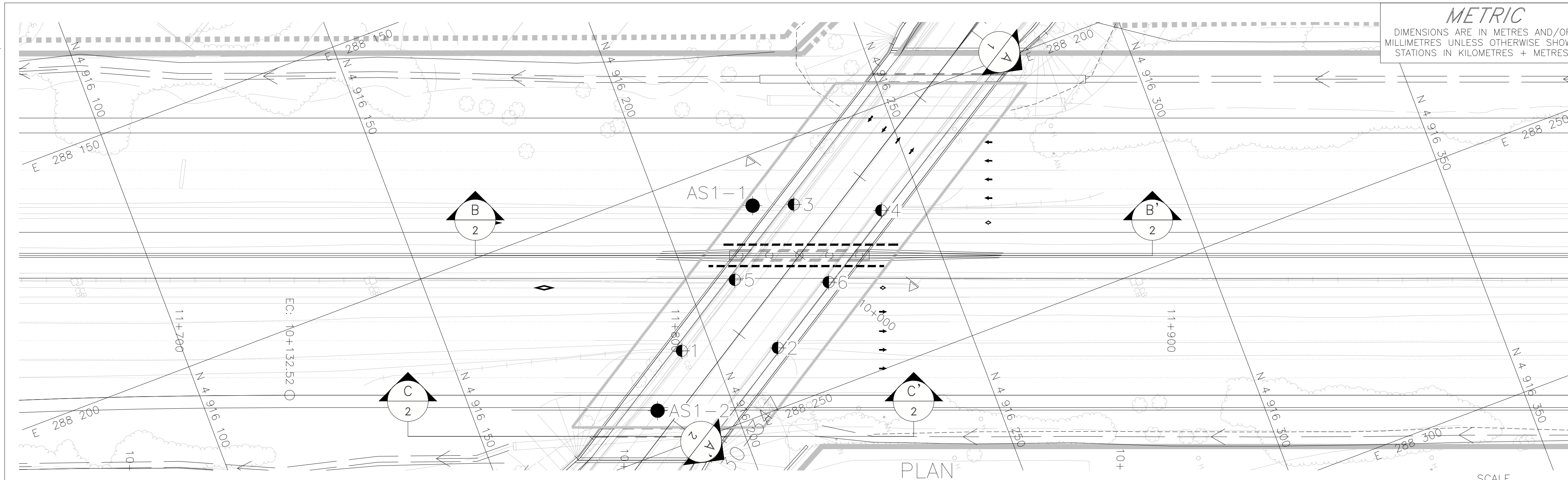
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

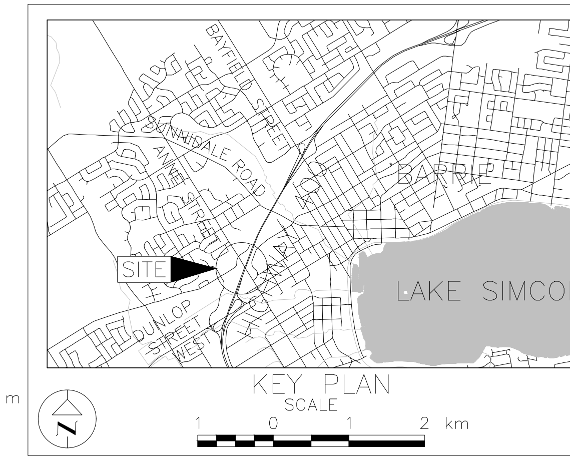
CONT No.
GWP No.06-20016

SHEET

ANNE STREET UNDERPASS
HIGHWAY 400 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA

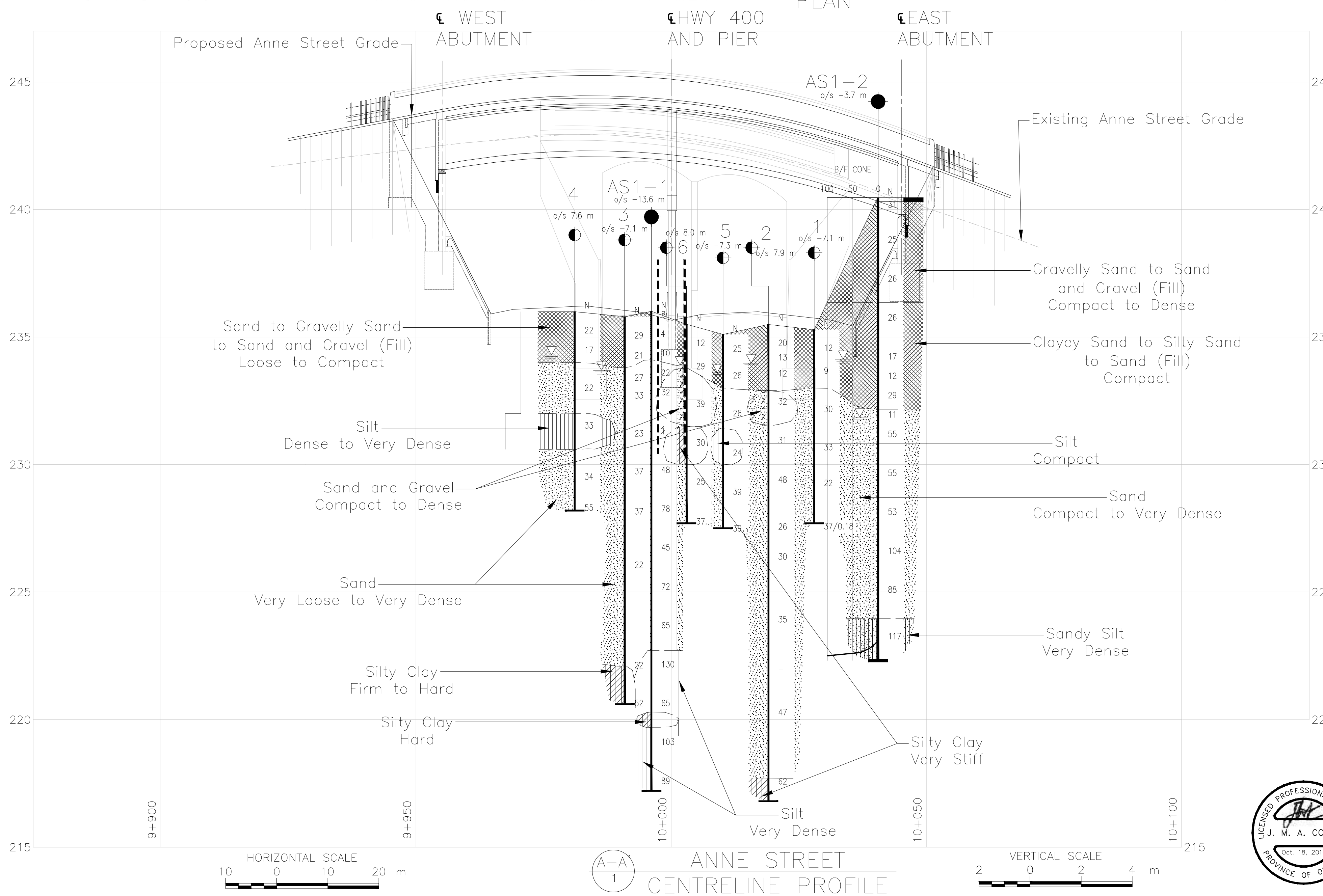
Golder Associates





LEGEND

●	Borehole – Current Investigation
◐	Borehole – Previous Investigation (Geocres No. 31D-182)
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling



A-A' 1 ANNE STREET CENTRELINE PROFILE

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
1	235.3	4916194.1	288232.1
2	235.5	4916212.5	288238.4
3	235.8	4916225.7	288212.6
4	236.0	4916241.8	288219.9
5	235.1	4916209.2	288222.6
6	235.5	4916226.7	288229.7
AS1-1	236.0	4916217.8	288209.9
AS1-2	240.5	4916185.3	288241.7

NOTES

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

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

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

REFERENCE

General arrangement, designs, base plans, profile and surface data provided in digital format by AECOM, drawing file nos. "01_Anne Street_GA(2).dwg", received June 23, 2016, "X-Base_All.dwg", received January 27, 2016, "X-Design_4th Line_Interim.dwg", received June 22, 2015, and "X-Surfaces.dwg", received April 14, 2015.

LICENSED PROFESSIONAL ENGINEER J. M. A. COSTA Oct. 18, 2016 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.C. NG 100086170 Oct. 18, 2016 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D-666

HWY. 400	PROJECT NO. 14-1111-0002		DIST.
SUBM'D. BM	CHKD. CN	DATE: 7/22/2016	SITE: 30-347
DRAWN: MR	CHKD. BM	APPD. JMAC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

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

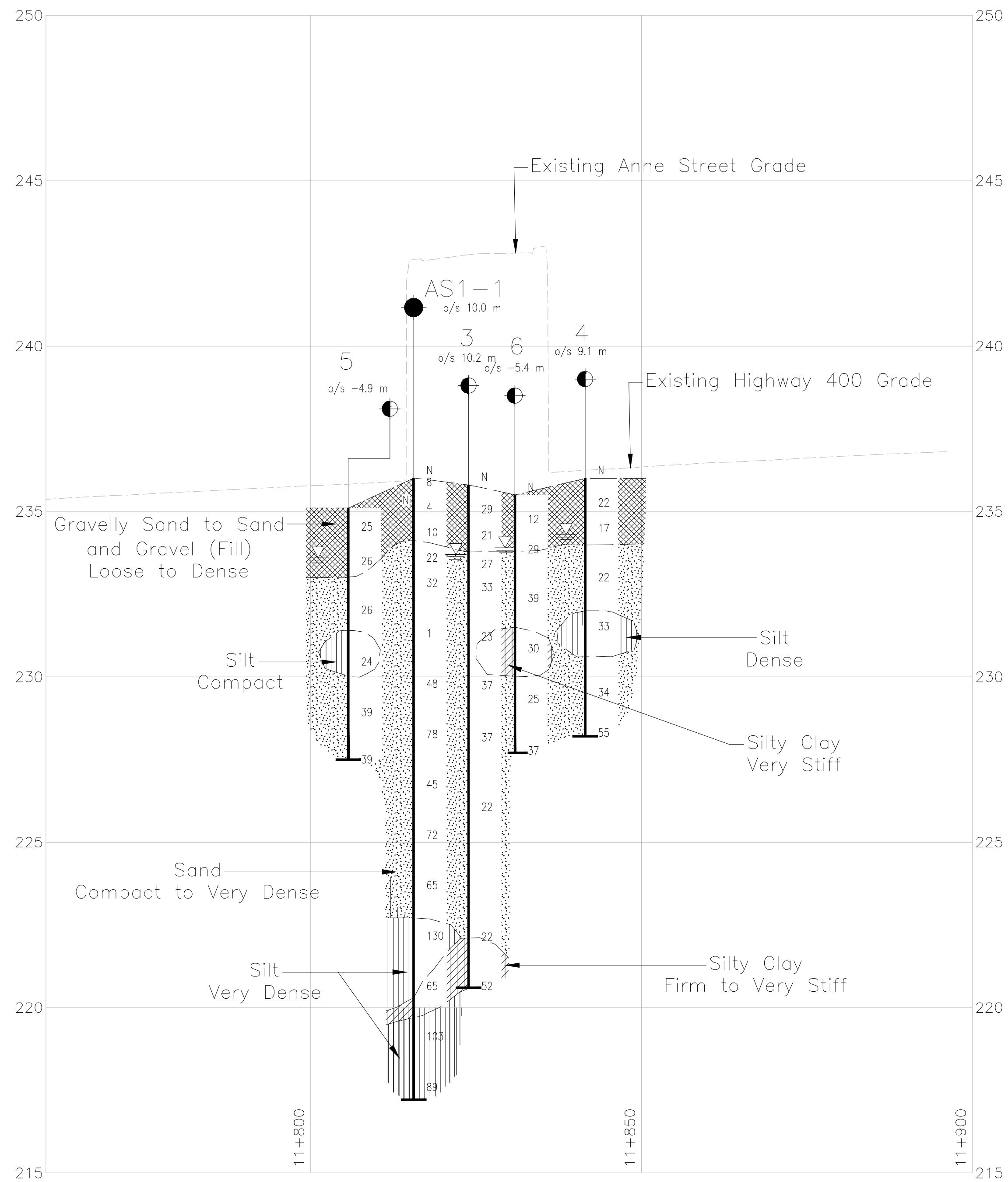
CONT No.
GWP No. 06–20016

ANNE STREET UNDERPASS
HIGHWAY 400 WIDENING

SOIL STRATA

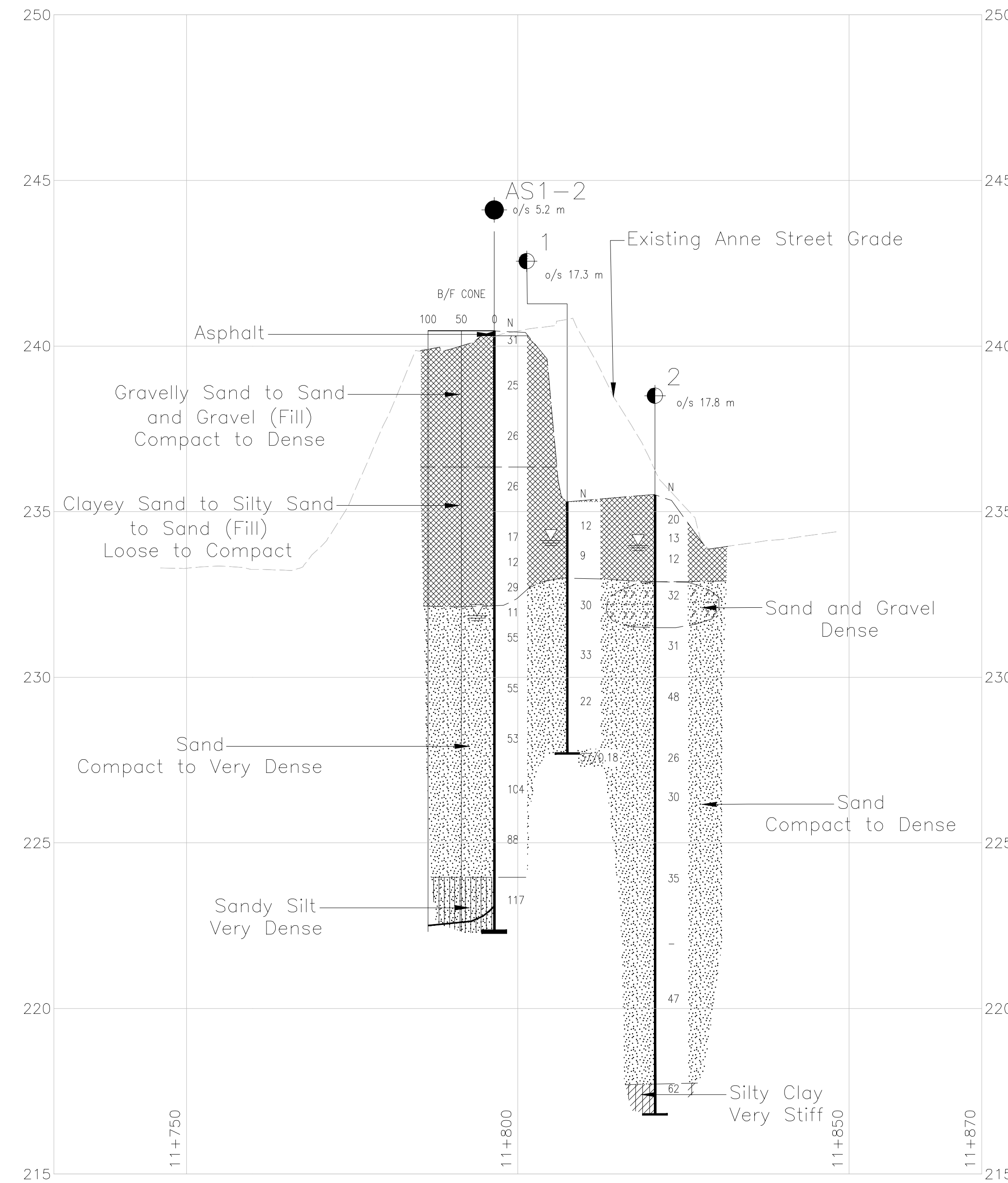
SHEET

Golder Associates



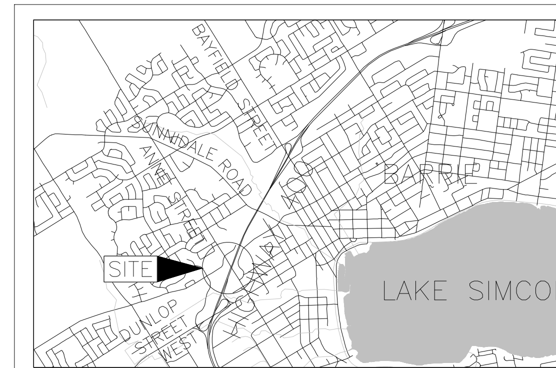
B–B 1 ANNE STREET CENTRE PIER CROSS–SECTION

HORIZONTAL SCALE
10 0 10 20 m



C–C 1 ANNE STREET EAST ABUTMENT AREA CROSS–SECTION

VERTICAL SCALE
2 0 2 4 m



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 31D–182)
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
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LICENSED PROFESSIONAL ENGINEER J. M. A. COSTA Oct. 18, 2016 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.C. NG 100086170 Oct. 18, 2016 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D–666

HWY. 400	PROJECT NO. 14–1111–0002		DIST.
SUBM'D. BM	CHKD. CN	DATE: 7/22/2016	SITE: 30–347
DRAWN: MR	CHKD. BM	APPD. JMAC	DWG. 2

PLOT DATE: October 18, 2016
FILENAME: S:\Clients\MTO\Hwy_400_Barrie\99_PROJ\1411110002_URS_Highway_400_TESR_Update\40_PROD\0008_TESR_AnneSt\1411110002–0008–BC–0002.dwg

APPENDIX A

Record of Boreholes – Golder 2016 Investigation

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 Soils

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

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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))	Sand and Gravel
	or With (cohesive)	Silty Clay with sand / Clayey Silt with sand

Golder Associates

RECORD OF BOREHOLE No AS1-1

SHEET 1 OF 2

METRIC

PROJECT 14-1111-0002
G.W.P. 06-20016
LOCATION N 4916217.8 ;E 288209.9
ORIGINATED BY ML
DIST Central HWY 400
BOREHOLE TYPE Truck - Mounted D-50 108 mm I.D., 194 mm O.D. Hollow Stem Auger
COMPILED BY MCK
DATUM Geodetic
DATE March 29, 2016
CHECKED BY CN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
236.0	GROUND SURFACE								
0.0	Gravelly sand, trace to some clay, containing wood fragments (FILL) Loose to compact Brown to grey Moist	1	SS	8					
		2	SS	4					
		3A / 3B	SS	10	○				
234.1 / 1.9	SAND, trace to some gravel, trace to some silt Very loose to very dense Brown to grey Wet	4	SS	22	○	16	74	8	2
		5	SS	32					
		6	SS	1	○				
		7	SS	48	○	4	75	19	2
		8	SS	78	○				
		9	SS	45					
		10	SS	72	○				
		11	SS	65					
222.7 / 13.3	SILT, trace to some sand, trace clay Very dense Brown to grey Wet	12	SS	130	○	0	6	90	4

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No AS1-1

SHEET 2 OF 2 **METRIC**

PROJECT 14-1111-0002
G.W.P. 06-20016 LOCATION N 4916217.8 ;E 288209.9 ORIGINATED BY ML
DIST Central HWY 400 BOREHOLE TYPE Truck - Mounted D-50 108 mm I.D., 194 mm O.D. Hollow Stem Auger COMPILED BY MCK
DATUM Geodetic DATE March 29, 2016 CHECKED BY CN

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	SILT, trace to some sand, trace clay, Very dense, Brown to grey, Wet	13A / 13B	SS	65								
220.3 / 15.7	SILTY CLAY, Hard, Grey, Wet					220						
219.7 / 16.3	SILT, trace to some sand, trace clay, Very dense, Brown to grey, Wet	14	SS	103		219			0	5	87	8
						218						
		15	SS	89								
217.2 / 18.8	END OF BOREHOLE											

NOTE:

1. Borehole caved to a depth of 1.2 m.

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

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No AS1-2

SHEET 1 OF 2 **METRIC**

PROJECT 14-1111-0002			
G.W.P. 06-20016	LOCATION N 4916185.3 ;E 288241.7		ORIGINATED BY ML
DIST Central HWY 400	BOREHOLE TYPE Truck - Mounted D-90, 108 mm I.D, 194 mm O.D. Hollow Stem Auger		COMPILED BY MK
DATUM Geodetic	DATE April 20 and 21, 2016		CHECKED BY CN

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
240.5	GROUND SURFACE
0.0	ASPHALT
0.2	Gravelly sand to sand and gravel, trace silt (FILL) Compact to dense Brown Moist
236.4 4.1	Silty sand, trace to some gravel, trace clay, containing black organic silt pockets (FILL) Compact Grey Moist
235.6 4.9	Sand, some silt, trace gravel to gravelly, trace organics (FILL) Compact Grey to brown Moist to wet - 130 mm organic silt pocket encountered at a depth of about 7.1 m - 80 mm clayey silt pocket encountered at a depth of about 7.3 m
232.2 8.3	SAND, trace gravel, trace to some silt Compact to very dense Brown Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	31	
2	SS	25	24 70 5 1
3	SS	26	
4A / 4B	SS	26	
5	SS	17	
6A / 6B	SS	12	
7	SS	29	
8	SS	11	
9	SS	55	
10	SS	55	
11	SS	53	0 83 17 0
12	SS	104	

GROUND WATER CONDITIONS: water level symbol at approx. elev. 232.

ELEVATION SCALE: 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226

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 (77 at sample 6)

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No AS1-2

SHEET 2 OF 2

METRIC

PROJECT 14-1111-0002

G.W.P. 06-20016

LOCATION N 4916185.3 ;E 288241.7

ORIGINATED BY ML

DIST Central HWY 400

BOREHOLE TYPE Truck - Mounted D-90, 108 mm I.D, 194 mm O.D. Hollow Stem Auger

COMPILED BY MK

DATUM Geodetic

DATE April 20 and 21, 2016

CHECKED BY CN

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND, trace gravel, trace to some silt Compact to very dense Brown Wet	13	SS	88	225	
224.0 16.5	Sandy SILT Very dense Brown Wet	14	SS	117	224 223	0 22 78 0
223.0 17.5	Start of Dynamic Cone Penetration Test (DCPT)					
222.4 18.1	END OF BOREHOLE NOTES: 1. Water level at a depth of about 8.6 m below ground surface (Elev. 231.9 m) upon completion of drilling. 2. Borehole caved to a depth of about 4.0 m.					

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

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

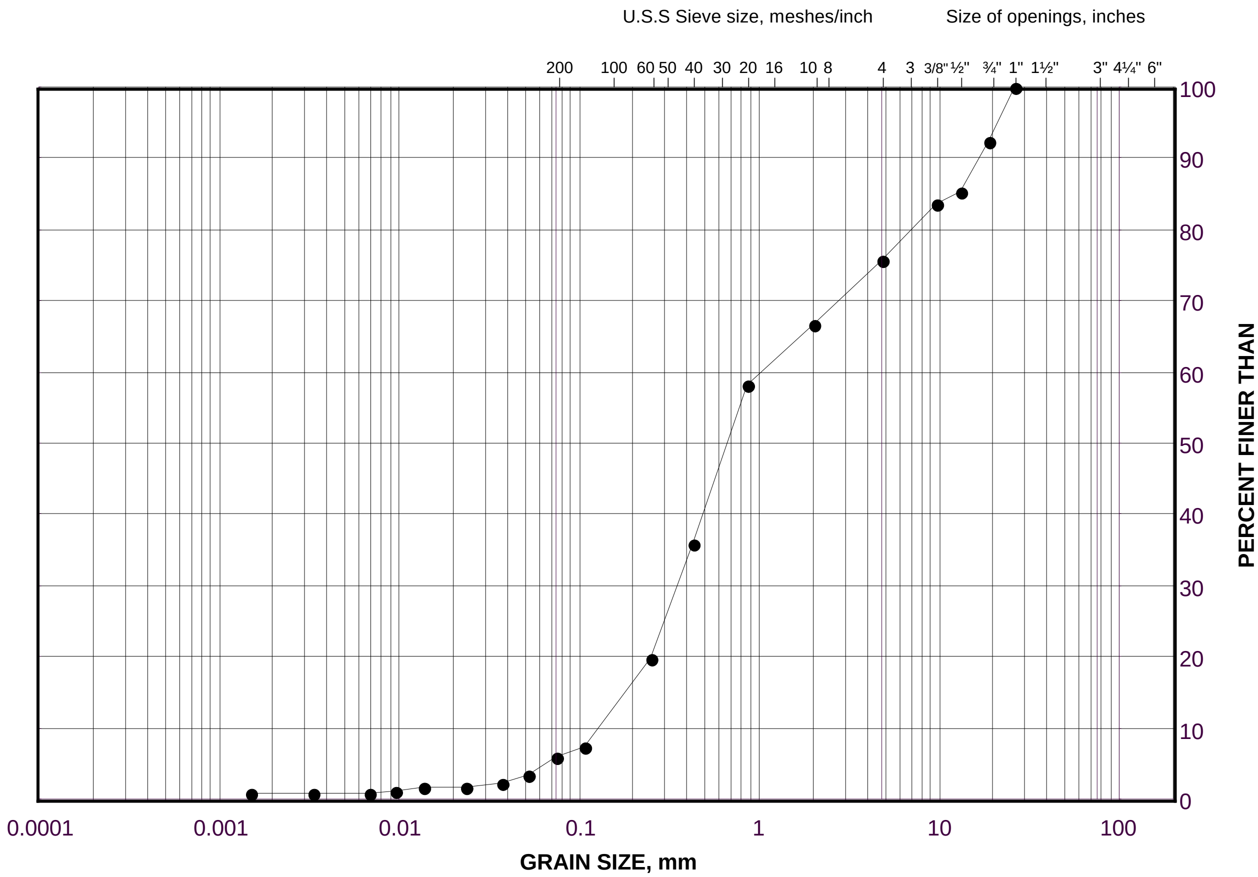
APPENDIX B

Laboratory Test Results – Golder 2016 Investigation

GRAIN SIZE DISTRIBUTION

Gravelly Sand (Fill)

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	AS1-2	2	238.7

Project Number: 14-1111-0002

Checked By: CN

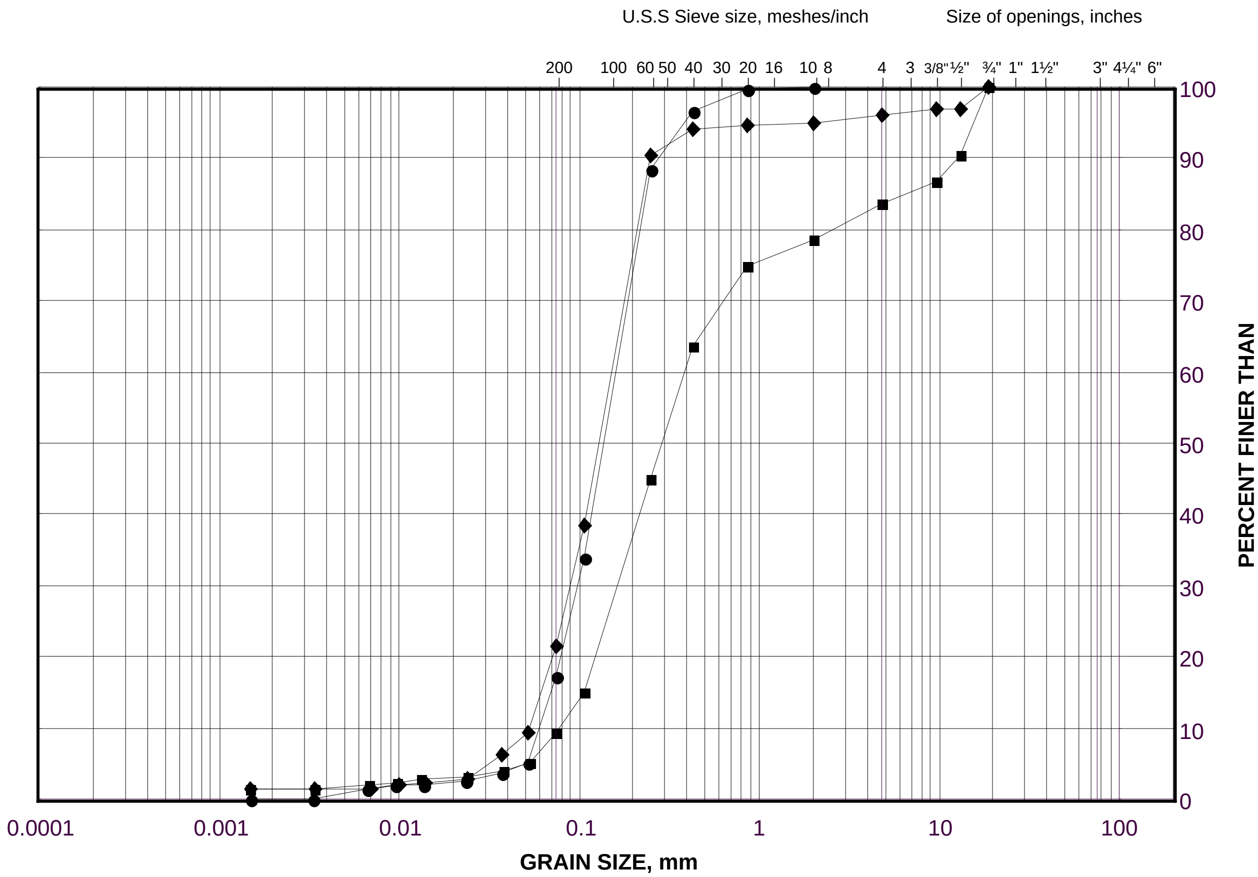
Golder Associates

Date: 21-Jul-16

GRAIN SIZE DISTRIBUTION

Sand

FIGURE B2-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AS1-2	11	228
■	AS1-1	4	233.4
◆	AS1-1	7	229.7

Project Number: 14-1111-0002

Checked By: CN

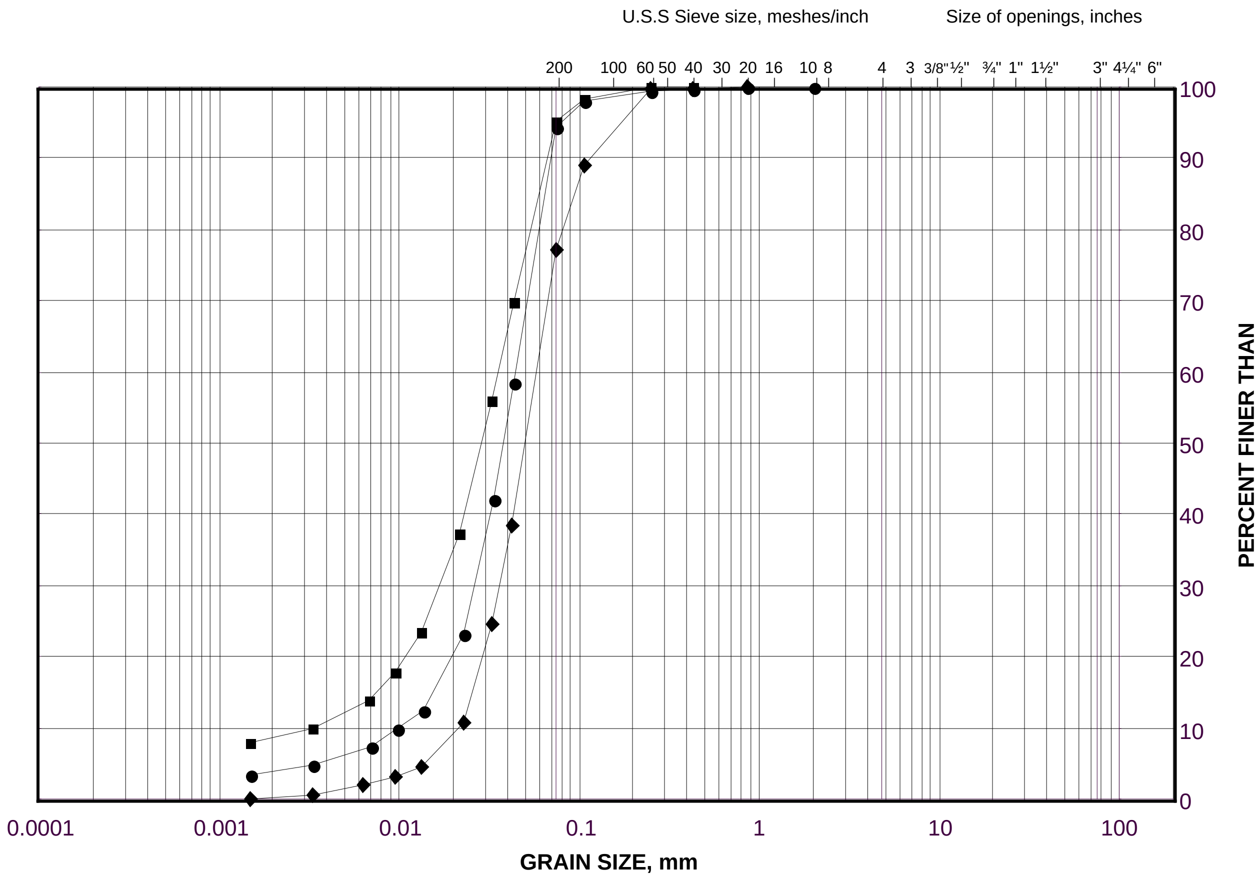
Golder Associates

Date: 22-Jul-16

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt

FIGURE B2-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AS1-1	12	222
■	AS1-1	14	219
◆	AS1-2	14	223.2

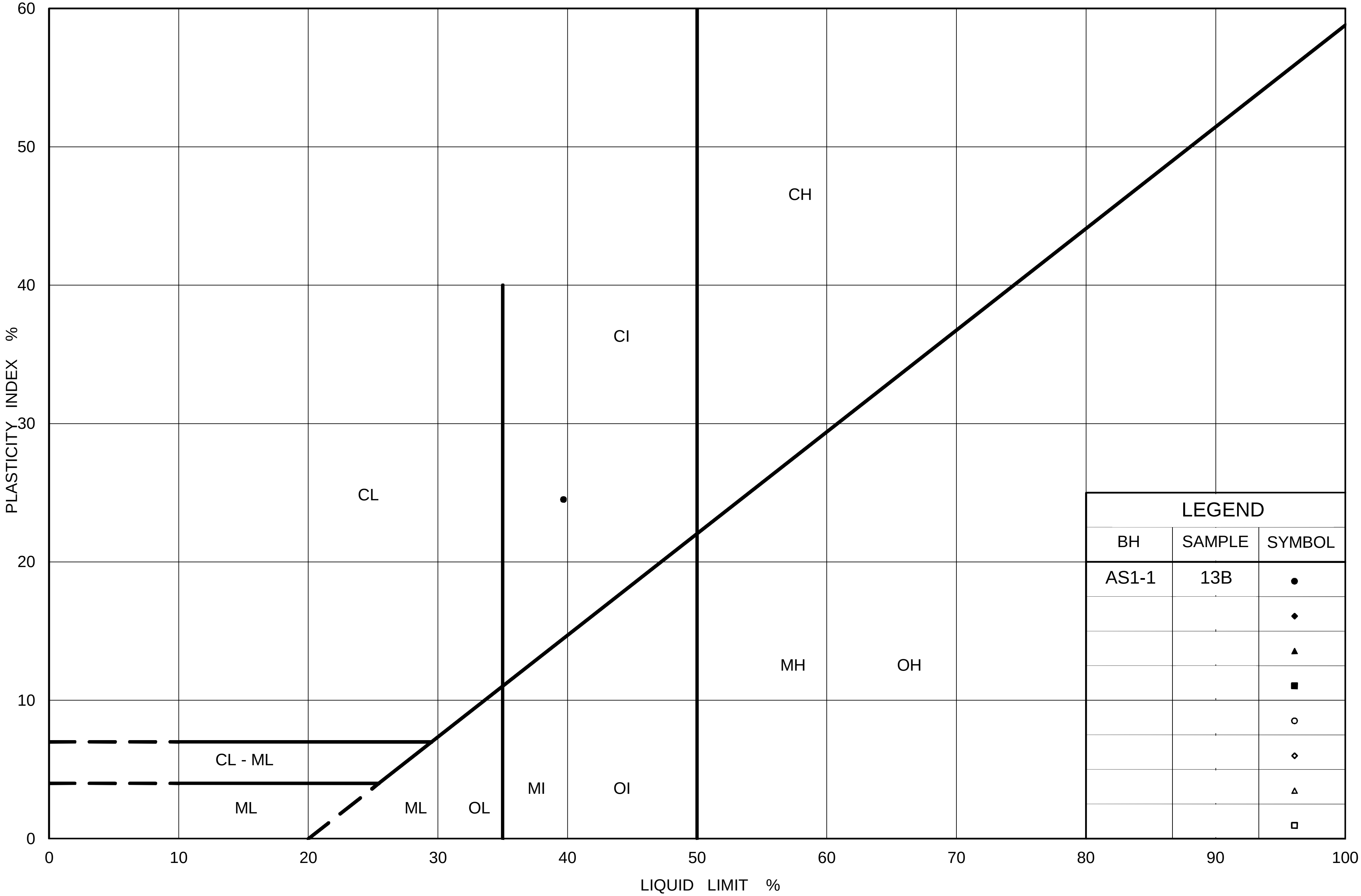
Project Number: 14-1111-0002

Checked By: CN

Golder Associates

Date: 22-Jul-16

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay

Figure No. B3

Project No. 14-1111-0002

Checked By: CN

APPENDIX C

Record of Boreholes – Previous Investigation (GEOCRES No. 31D-182)

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PRO Anne Street Overpass, Barrie, Ontario. ORDER NO. I.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.1 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ———

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	771.82 235.3m			Zero – 0.0m			
Firm brown to grey clayey sand with some organic matter and fine to medium gravel. Probably FILL.			● 1	Free Water		12	Moist Low to medium dry strength.
Loose brown to grey sand with fine to medium gravel and some organic matter. Probably FILL.			● 2			9	Moist Low dry strength.
Dense brown to grey fine to coarse SAND with generally subrounded fine to medium gravel.			● 3			30	do
Dense brown grey generally fine calcareous SAND with fine to medium subrounded gravel.			● 4			33	do
Firm do			● 5			22	do
Dense brown grey fine to medium calcareous SAND with fine to medium subrounded gravel.	227.7m		● 6 End of Borehole	25'-1" – 7.6m		37 (7")	do

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL GEOTECHNIQUE LIMITED
SOIL MECHANICS LABORATORY
BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario. ORDER NO. 1,227/37

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.2 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	772.65 225.5m			Zero	0.0m		
Firm brown grey sand, clayey concentrations. Black organic matter. Probably FILL.			● 1			20	Moist
Firm do			● 2	Free Water		15	do
do Iron staining			● 3			12	do
	232.9m			8'-6"	2.6m		
Dense grey brown fine to coarse calcareous SAND and fine to medium generally subrounded GRAVEL.			● 4			32	Moist Low dry strength.
	231.5m			13'-0"	4.0m		
Dense brown sandy SILT with lenses of fine to medium SAND. Traces of bedding.			● 5			31	Moist. Low to medium dry strength.
do Some iron staining			● 6			48	do
	228.5m			23'-0"	7.0m		
Firm brown grey fine to medium calcareous SAND. Lenses of fine subrounded to rounded gravel embedded in clay.			● 7			26	Moist Low dry strength.
Dense brown grey fine to medium calcareous SAND with fine to medium gravel, generally subrounded.			● 8			30	do
do			● 9			35	do
do			● 10			-	Wash Sample
	221.2m			47'-0"	14.3m		
Dense grey generally fine calcareous SAND.			● 11			47	Moist Low dry strength.

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT: Anne Street Overpass, Barrie, Ontario. ORDER No. T.227/57

CLIENT: Department of Highways, Ontario.

BOREHOLE No. BH.2 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

FORM G-1A 500-6-54 UNITED STATIONERS CO.

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Dense grey generally fine calcareous SAND.	220.3m			50'-0"	15.2m		
Very stiff grey calcareous silty CLAY.	217.7m		● 12	58'-6"	17.8m	62	Moist. Sand: Low dry strength. Clay: High dry strength.
	216.8m		End of Borehole	61'-6"	18.7m		

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL GEOTECHNIQUE LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT: Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT: Department of Highways, Ontario.

BOREHOLE NO. BH.3 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	773.58			Zero —	0.0m		
Firm brown sand, gravel, little clay and bits of wood. FILL.	235.8m		● 1			29	Moist
Firm brown sand and black organic matter. Probably FILL.			● 2			21	do
					Free Water		
	233.8m			6'-7"	2.0m		
Firm grey to iron-stained yellow fine to medium SAND with fine to medium generally subrounded gravel.			● 3			27	Wet Low dry strength.
do No iron staining.			● 4			33	Moist Low dry strength.
do			● 5			23	do
	230.2m			18'-6"	5.6m		
Dense grey generally fine calcareous silty SAND.			● 6			37	do
do Slight iron staining.			● 7			37	do
do No iron staining.			● 8			22	Wet Low dry strength.
	225.4m			34'-0"	10.4m		
Brown grey fine to medium calcareous SAND.			● 9				Wash Sample
Grey generally fine calcareous SAND.			● 10				Wash Sample
do			● 11			22	Moist Low dry strength.
	222.1m			45'-0"	13.7m		
Firm grey silty CLAY.							
Hard do	220.6m		● 12	50'-0"	15.2m	52	Last sample

End of Borehole

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

FORM G-1A 500-6-54

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.4 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ———

FORM G-1A 500-6-54 UNITED STATIONERY CO.

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	774.26 236.0m			Zero	0.0m		
Firm grey brown fine to medium somewhat clayey sand with gravel. Probably FILL.			● 1			22	Moist
do With traces of organic matter. Probably FILL.			● 2	Free Water		17	Wet
Firm grey brown fine to coarse calcareous SAND and fine to medium generally subrounded GRAVEL.			● 3			22	Wet No dry strength.
	232.0m			13'-0"	4.0m		
Dense brown sandy SILT with thin lenses of clay. Exhibits bedding.			● 4			33	Damp Low to medium dry strength.
Dense grey generally fine calcareous SAND.			● 5			34	Wash sample
Dense grey brown generally fine calcareous SAND with occasional fine gravel. Exhibits faint bedding and some iron staining.			● 6			55	Moist. Low to medium dry strength.
	228.2m		End of Borehole	25'-6"	7.8m		

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.5 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ———

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	771.38 235.1m			Zero	0.0m		
Firm grey brown sand, gravel and little clay. Probably FILL.			● 1			25	Moist
do With some organic matter.			● 2	Free Water		26	Moist
	233.0m			7'-0"	2.1m		
Firm grey brown fine to coarse SAND with fine to medium generally subrounded GRAVEL.			● 3			26	Wet No dry strength.
	231.4m			12'-0"	3.7m		
Firm brown sandy SILT with some gravel and clay bands.			● 4			24	Moist Low to medium dry strength.
Dense grey brown fine to medium calcareous SAND.			● 5			39	Moist Low dry strength.
do	227.5m		● 6 End of Borehole	25'-1"	7.6m	39 (7")	Wet Low dry strength.

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

FORM G-1A 500-6-54
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UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.6 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ———

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	772.48 235.5m			Zero	0.0m		
Firm grey sand and black organic matter. Probably FILL.			• 1			12	Moist
Firm grey and iron-stained yellow sand, little clay, Probably FILL.			• 2	Free Water		29	Moist
Dense medium to coarse calcareous SAND and fine to medium generally subrounded GRAVEL.			• 3			39	Wet No dry strength.
	231.5m			13'-0"	4.0m		
Very stiff brown sandy silty calcareous CLAY with fine to medium subangular to subrounded gravel.			• 4			30	Moist High dry strength.
Firm grey brown fine to coarse SAND and fine to medium subangular to subrounded GRAVEL.			• 5			25	Wet No dry strength.
Dense grey brown fine to medium calcareous SAND with gene. ·lly subrounded gravel.	227.7m		• 6	25'-6"	7.8m	37	Moist Low dry strength.
			End of Borehole				

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

FORM G-1A 500-6-54 UNITED STATIONERY CO.

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Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444