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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Farewell Creek Bridge Replacement Structure Site No. 22-183 Highway 401 Widening from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

Submitted to:

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REPORT



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Table of Contents

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 INVESTIGATION PROCEDURES	1
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	2
4.1 Regional Geology	2
4.2 Subsurface Conditions.....	2
4.2.1 Fill	2
4.2.2 Upper Silty Sand	3
4.2.3 Clayey Silt	3
4.2.4 Lower Silty Sand	4
4.2.5 Shale Bedrock.....	4
4.3 Groundwater Conditions	4
5.0 CLOSURE.....	5

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS	6
6.1 General.....	6
6.2 Foundation Options	6
6.3 Driven Steel H-Pile or Steel Pipe Pile Foundations	8
6.3.1 Founding Elevation	9
6.3.2 Geotechnical Axial Resistance/Reaction.....	9
6.4 Caisson Foundations	10
6.4.1 Founding Elevation	10
6.4.2 Geotechnical Axial Resistance/Reaction.....	10
6.5 Approach Embankments	10
6.5.1 Subgrade Preparation and Embankment Construction	11
6.5.2 Slope Stability	11
6.5.3 Settlement Under Widened Embankment Loading	11



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

6.6	Construction Considerations.....	12
6.6.1	Open-Cut Excavation and Temporary Protection Systems	12
6.6.2	Groundwater Control.....	13
6.6.3	Obstructions.....	13
6.6.4	Vibration Monitoring During Pile Installation.....	13
6.7	Recommendations for Future Work in Detail Design	13
7.0	CLOSURE.....	14

REFERENCES

TABLES

Table 1	Comparison of Foundation Alternatives
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DRAWINGS

Drawing 1	Farewell Creek Bridge, Highway 401 Improvements - Borehole Locations and Soil Strata
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FIGURES

Figure 1	Static Global Stability, Farewell Creek Bridge – Approach Embankments
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APPENDIX A	Borehole Records and Laboratory Test Results – GEOCRES No. 30M15-8
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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
FAREWELL CREEK BRIDGE
STRUCTURE SITE NO. 22-183
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario.

This report addresses the proposed replacement and widening of the existing Highway 401-Farewell Creek Bridge. This report was developed with information from a previous foundation investigation at the Farewell Creek bridge site, as follows:

- **MTO GEOCRES No. 30M15-8:** "Foundation Investigation Report for Proposed Widening of the Bridge Structure at the Crossing of Hwy. 401 and Farewell Creek (Site No. 22-183), Township of Whitby, County of Ontario, District No. 6, W.O. 72-11128, W.P. 44-71-12," prepared by Ministry of Transportation and Communications, Ontario, dated March 13, 1973.

The terms of reference for the preliminary foundation engineering services are outlined in MTO's Request for Proposals (RFP) for Assignment No. 2010-E-0062, dated June 2011. The scope of work for the preliminary foundation engineering services is presented in Section 5.8 of the *Technical Proposal* for this assignment, as well as Golder's Scope Change for Foundations Engineering Services letter dated December 8, 2014.

2.0 SITE DESCRIPTION

The Farewell Creek bridge is located approximately 700 m east of the Bloor Street East underpass in the City of Oshawa, in the Regional Municipality of Durham, Ontario. The existing bridge is an approximately 12.2 m long single-span structure with closed abutments. The original structure is supported on spread footings that are founded at approximately Elevation 75.4 m; the structure was widened to the north and south as part of a 1977 contract, and the widened portions of the bridge are supported on steel H-piles driven to bedrock at about Elevation 71.3 m to 72.5 m.

The Farewell Creek channel bed is at approximately Elevation 76.5 m, and is underlain by a 150 mm thick reinforced concrete slab. The banks of Farewell Creek are approximately 0.5 m to 1 m in height with relatively shallow slopes. The floodplain adjacent to the creek is at approximately Elevation 78, rising to the west and east away from the creek.

Highway 401 has been constructed on embankments that are approximately 2 m to 3 m in height, with the highway grade at approximately Elevation 80.5 m in the vicinity of the creek. Vegetation cover on both sides of Highway 401 consists of grass, shrubs and trees.

3.0 INVESTIGATION PROCEDURES

Six boreholes, each accompanied by a dynamic cone penetration test, were advanced at this site as part of the 1972 investigation completed by the Ministry of Transportation and Communications, Ontario. For the purpose of this Preliminary Foundation Investigation Report, the boreholes have been re-numbered such that the 30M15-series GEOCRES number precedes the original borehole number. For example, Borehole 1 from GEOCRES Report No. 30M15-8 is referred to throughout this report and on the drawings as Borehole 8-1. The approximate



locations of the boreholes are shown on Drawing 1; these borehole locations have been interpreted based on scaling measurements from the plan shown in the 1973 GEOCRETS report.

Boreholes 8-1, 8-1A, 8-2 and 8-2A were advanced on the south side of Highway 401, and Boreholes 8-3 and 8-4 were advanced on the north side of Highway 401. The boreholes were advanced using a CME auger machine adapted for soil sampling, with diamond drilling equipment to core through the bedrock. Soil samples were obtained using a 50 mm outside diameter split-spoon sampler driven by a manual hammer in accordance with the Standard Penetration Test (SPT) procedure. BX core samples were obtained from the bedrock in three of the boreholes.

The groundwater conditions in the open boreholes were observed following the drilling operations and are noted on the Record of Borehole sheets in Appendix A.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984) and *Urban Geology of Canadian Cities* (Brennard, 1998). The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt and/or clay are present in the creek valleys.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the previous investigation, together with the results of in situ and laboratory testing, are presented on the borehole records and laboratory test figures contained in Appendix A. Interpreted stratigraphic profiles along the north and south sides of the structure are shown on Drawing 1.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profiles are inferred from observations of drilling progress and non-continuous sampling. These boundaries, therefore, 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 surficial layer of fill underlain by an upper deposit of loose to very dense silty sand, a stiff to hard stratum of clayey silt, and a lower deposit of compact to very dense silty sand. The overburden soils are underlain by shale bedrock of the Whitby Formation. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Four of the boreholes (Boreholes 8-1A, 8-2A, 8-3 and 8-4) were advanced through the fill associated with the existing approach embankments. The fill material was encountered immediately below the then-existing ground



surface and the thickness ranged between approximately 1.4 m and 2.2 m. The base of the fill was encountered between about Elevation 77.4 m and 77.7 m.

The fill material consists of silty sand containing trace to some gravel, with occasional layers of clayey silt noted in Borehole 8-2A. The organic content measured on one sample of the fill recovered from Borehole 4 is about 2.6 per cent. The result of a grain size distribution test performed on one sample of the silty sand fill deposit is included on Figure 1 in Appendix A. Atterberg limits testing was carried out on the fines portion of one sample of this deposit, and measured a plastic limit of about 21 per cent, a liquid limit of about 24 per cent, and plasticity index of about 3 per cent; this result indicates that the fines portion of this deposit is a silt of slight plasticity. The natural water content measured on selected samples of the fill ranges between approximately 5 and 18 per cent.

The Standard Penetration Test (SPT) 'N'-values measured in the fill range from 12 blows to 47 blows per 0.3 m penetration, indicating a compact to dense relative density.

4.2.2 Upper Silty Sand

An upper deposit of silty sand was encountered in all of the boreholes advanced at this site immediately below the existing ground surface or underlying the fill deposit (where present). The thickness of the upper silty sand deposit ranges between 2.1 m and 3.0 m, and the base of the deposit was encountered between about Elevation 74.0 m and 75.6 m.

The deposit is described on the borehole records as consisting of silty sand, trace to some gravel. The results of grain size distribution tests performed on five samples of the silty sand deposit are shown on Figure 1 in Appendix A; these test results demonstrate that selected samples in Boreholes 8-2A and 8-3 contain a higher proportion of gravel, and may be described as sand and gravel containing some silt. The organic content measured on one sample of this deposit recovered from Borehole 2A is about 2.3 per cent. Atterberg limits testing was carried out on the fines portion of two samples of this deposit, and measured plastic limits of about 10 and 11 per cent, a liquid limit of about 13 per cent, and plasticity indices of about 2 and 3 per cent; these results indicate that the fines portion of this deposit consist of silt of slight plasticity. The natural water content measured on selected samples of the silty sand deposit ranges between about 5 and 28 per cent.

The SPT 'N'-values measured in the silty sand deposit range from 4 blows to 59 blows per 0.3 m penetration, indicating a very loose to very dense relative density.

4.2.3 Clayey Silt

A stratum of clayey silt was encountered in all of the boreholes advanced at this site underlying the upper silty sand deposit at depths ranging between 2.5 m and 4.7 m below the ground surface at the time of the 1972 investigation. The thickness of the clayey silt stratum ranges between 1.3 m and 2.8 m, and the base of the deposit was encountered between about Elevation 72.7 m and 73.7 m.

The cohesive deposit consists of clayey silt containing some sand and trace to some gravel. The results of grain size distribution tests performed on three samples of the clayey silt stratum are shown on Figure 2 in Appendix A. Atterberg limits testing was carried out on seven samples of this deposit, and measured plastic limits between about 11 and 15 per cent, liquid limits between about 16 and 25 per cent, and plasticity indices between about 4 and 10 per cent; these results, which are plotted on a plasticity chart on Figure 3 in Appendix A, confirm that this deposit can be classified as a clayey silt of low plasticity. The natural water content measured on selected samples of the clayey silt stratum ranges between 9 and 20 per cent.



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

The SPT 'N'-values measured in the clayey silt stratum range from 7 to 57 blows per 0.3 m penetration. A field vane test measured an undrained shear strength of approximately 80 kPa within this stratum in Borehole 8-1; this result, together with the SPT results suggest a firm to hard consistency.

4.2.4 Lower Silty Sand

A lower deposit of silty sand was encountered in Boreholes 8-1, 8-2, 8-2A and 8.4, underlying the clayey silt deposit. This lower deposit was encountered at depths ranging between 3.9 m and 6.6 m below ground surface at the time of the 1972 investigation. The thickness of the lower silty sand deposit ranges between 1.3 m and 1.8 m, and the base of the deposit was encountered between about Elevation 71.4 m and 71.8 m.

This deposit consists of silty sand containing trace to some gravel. The result of a grain size distribution test performed on one sample of the lower silty sand is included on Figure 1 in Appendix A. The natural water content measured on selected samples of the lower silty sand deposit ranges between about 8 and 16 per cent.

The SPT 'N'-values measured in the lower silty sand deposit range from 17 to 39 blows per 0.3 m penetration, indicating a compact to dense relative density.

4.2.5 Shale Bedrock

Shale bedrock of the Whitby Formation was encountered in all of the boreholes advanced at this site, underlying the clayey silt or lower silty sand (where present) at Elevation 71.4 m to 72.8 m; this corresponds to depths ranging from 5.6 m to 7.9 m below the ground surface at the time of the 1972 investigation.

The upper 0.5 m to 1.1 m of the shale is weathered, as interpreted by the penetrability of this portion of the formation by augering and split-spoon sampling; the bedrock below the weathered zone was generally described as sound shale bedrock.

4.3 Groundwater Conditions

During the 1972 investigation, water levels were observed in the open boreholes at the completion of drilling, and are summarized below. The water level was between Elevation 76.6 m and 77.3 m, generally near the water level in Farewell Creek.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level Below 1972 Ground Surface (m)	Approximate Groundwater Elevation (m)
8-1	77.1	0.4	76.7
8-1A	79.1	2.5	76.6
8-2	77.0	0.3	76.7
8-2A	79.6	2.8	76.8
8-3	79.4	2.7	76.7
8-4	79.6	2.3	77.3

The water levels observed in the open boreholes on completion of drilling may not represent long-term stabilized groundwater levels, nor the current groundwater regime at the site. The water level at the site is expected to



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and following periods of precipitation.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Matthew Kelly, P.Eng. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact and Principal of Golder, conducted an independent technical and quality review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
FAREWELL CREEK BRIDGE
STRUCTURE SITE NO. 22-183
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation recommendations in support of the proposed replacement and widening of the existing Highway 401-Farewell Creek bridge (MTO Structure Site 22-183) and associated wingwalls/retaining walls. These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 1972 subsurface investigation at this site. This Preliminary Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of MTO to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. This Preliminary Foundation Design Report shall not be used or relied upon for any other purpose or by any other parties, including contractors. Further investigation and design will be required during the detailed design stage.

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 or operational constraints may be required in the contract documents in the future. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect the equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

It is understood that as part of the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, the existing Farewell Creek bridge will be replaced and widened. Based on the preliminary General Arrangement drawing provided by AECOM, the new bridge span will be 18 m (or about 20.5 m as measured on skew between the centreline of the abutment bearings), with a total width of 65.3 m. This new span length will place the new abutments behind the existing abutments, eliminating conflicts with the existing foundations and minimizing requirements for in-water work during the new construction.

The following design drawings are available for the existing Farewell Creek bridge:

- Contract No. 77-133, WP No. 44-71-12, Sheet 1: "Widening of Existing Farewell Creek and Highway 401 Bridge, General Plan", prepared by Ministry of Transportation and Communication - Ontario, dated February 1975.
- Contract No. 77-133, WP No. 44-71-12, Sheet 3: "Widening of Existing Farewell Creek and Highway 401 Bridge, Footing Layout", prepared by Ministry of Transportation and Communication - Ontario, dated February 1975.

Based on the available information, the original single-span structure was constructed in 1950 and is supported on spread footings founded in the compact to dense / very stiff to hard native soils at approximately Elevation 75.4 m. The bridge was widened by about 2.7 m on each side under the 1977 contract; based on the design drawings, the widened abutments and associated retaining walls are supported on steel HP 310x79 piles driven to refusal on the shale bedrock. Based on visual observations, the existing abutments are considered to have performed satisfactorily.



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

Both shallow and deep foundation options have been considered for support of the widening. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip footings founded on the compact to dense silty sand deposit:** Although the surficial fill materials and very loose / loose portions of the upper silty sand deposit at the site are not considered suitable for the support of shallow foundations, footings could be founded deeper, on the compact to dense portion of the upper silty sand. The footings would need to be founded at approximately Elevation 75.5 m, which is near the existing footing founding elevation, to extend below the loose soils. Alternatively, the loose materials could be subexcavated and replaced with compacted granular fill to permit footings to be founded at a higher elevation (subject to scour considerations). This founding/subexcavation level would require excavation to a depth of approximately 6 m relative to the existing ground surface. Such excavations could affect the foundations for the existing structure, during the construction staging. Temporary protection systems would be required along the highway to facilitate excavation to the foundation level; protection systems/cofferdams would also be required in front of the abutment footings. The proposed founding level will be below the groundwater level at the site, and groundwater control/cofferdams would be required.
- **Driven steel H-piles or pipe piles founded on the shale bedrock:** Driven steel H-piles or steel pipe piles are feasible for support of the new abutments and wingwalls/retaining walls. Deep foundations are considered to have a technical advantage over spread footings as they would not require significant subexcavation of loose soils below the groundwater table. The pile caps may be able to be maintained higher than a strip footing foundation, thus minimizing excavation and groundwater control requirements compared to the strip footing option. However, excavation, protection systems and dewatering/cofferdams would still be required for the pile caps. The surface of the bedrock is at about Elevation 72.5 m in the northeast quadrant, Elevation 71.8 m in the northwest quadrant, Elevation 71.5 m in the southeast quadrant and Elevation 71.3 m in the southwest quadrant; as such, the piles will be relatively short. Pile driving shoes are recommended to protect the pile tips from damage during driving and seating of the piles on the shale bedrock.
- **Caissons founded in the shale bedrock:** Caissons are considered feasible for the support of the widened abutments and wingwalls / retaining walls. As for piles, caissons are considered to have a technical advantage over spread footings as they would eliminate the requirement for subexcavation associated with the spread footing option. However, excavation, protection systems and dewatering/cofferdams would still be required for the pile caps. If caissons are adopted for support of the abutments, they would extend into and through water-bearing non-cohesive soil deposits; temporary liners would be required during construction to control potential ground loss and disturbance. This option will be more expensive than pile foundations, although fewer caisson elements would be required in comparison to the number of steel piles that would be required.

Based on the above considerations, strip footings, driven piles and caissons are all considered feasible for the support of the new abutments and wing walls / retaining walls. Driven pile foundations are preferred from a geotechnical/foundations perspective as they would minimize the requirement for deep excavations and groundwater control requirements as compared with strip footings, and limit potential for ground loss/disturbance as could occur with caisson construction.



6.3 Shallow Foundations

6.3.1 Founding Elevation

If strip footings are adopted for support of the new abutments, they must either be founded below the loose silty sand layers. For preliminary design, the footings would need to be founded at approximately Elevation 75.5 m, which is near the existing footing founding elevation, to extend below these loose soils. Alternatively, the loose materials could be subexcavated and replaced with compacted granular fill to permit footings to be founded at a higher elevation (subject to scour considerations). Notwithstanding these requirements, strip footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). If adequate soil cover cannot be provided for footings (for example, for retaining walls adjacent to the abutments), rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

The groundwater level at the site was between approximately Elevation 76.6 m and 77.3 m during the 1972 investigation, although this may not represent the stabilized or current groundwater level at the site. This is slightly above the base of the creek channel, which is at approximately Elevation 76.5 m at the bridge site. Groundwater control will be required during excavation to maintain stable subgrade conditions for foundation construction. It is understood that the new abutments will be constructed behind the existing abutment footings; these existing footings are relatively deep and consideration should be given to leaving these footings in place following the removal of the existing structure. The presence of these footings may impact and/or improve requirements related to cofferdams, and further assessment of this aspect should be made at the detail design stage.

The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately, a 100 mm thick concrete working slab (of 20 MPa compressive strength concrete) should be placed on the prepared subgrade within four hours of its inspection and approval.

6.3.2 Geotechnical Axial Resistance/Reaction

The following factored geotechnical axial resistance at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of spread footings founded on the properly prepared subgrade at the design elevations given in the preceding section:

Founding Element	Factored Geotechnical Resistance (ULS) kPa	Geotechnical Reaction at SLS (for 25 mm of settlement) kPa
West and East Abutments	500	350

NOTE: The geotechnical resistance/reaction values given above are estimated for 3 m wide strip footings.

These geotechnical resistances are given for loads that will be applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with the Table 10.2 in *CFEM* (2006).

These preliminary geotechnical resistances should be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation, and the final footing width and founding elevation.



6.4 Driven Steel H-Pile or Steel Pipe Pile Foundations

6.4.1 Founding Elevation

For preliminary design purposes, it has been assumed that the abutment pile caps would be constructed at a minimum depth of 1.2 m below surrounding grade to provide adequate protection against frost penetration. However, the pile caps may need to be founded deeper to provide adequate protection against scour, and this aspect should be assessed by the hydraulic engineer.

The table below provides pile tip levels for preliminary design of pile foundations driven to refusal on or in the shale bedrock. Based on the strength and weathering reported in the upper portion of the bedrock where it could be penetrated by augering and split-spoon sampling, these recommendations assume nominal penetration into the bedrock at the new abutment locations.

Foundation Element	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
East Abutment	71.4 to 72.8	71.5
West Abutment	71.4 to 71.8	71.0

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the bedrock. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (*Steel H-Pile Driving Shoe*) or OPSD 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. When seating piles into weathered bedrock, driving shoes (such as Titus Standard 'H' Bearing Pile Points) are preferred over flange plates.

6.4.2 Geotechnical Axial Resistance/Reaction

For HP 310x110 piles driven to refusal on the shale bedrock at the approximate pile tip elevations given above, the factored axial geotechnical resistance at Ultimate Limit States (ULS) may be taken as 1,600 kN, and the axial geotechnical reaction at Serviceability Limit States (SLS) may be taken as 1,400 kN (for 15 mm of settlement). The same axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.

Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*). The drawings should note that the piles should be equipped with driving shoes or bearing points and driven to bedrock. For piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

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



6.5 Caisson Foundations

6.5.1 Founding Elevation

Caissons founded on or socketed into the shale bedrock could be considered for support of the new abutments. For preliminary design purposes, it has been assumed that the abutment pile caps would be constructed at a minimum depth of 1.2 m below surrounding grade to provide adequate protection against frost penetration. However, the pile caps may need to be founded deeper to provide adequate protection against scour, and this aspect should be assessed by the hydraulic engineer.

For preliminary design, the following caisson base elevations may be assumed. Depending on the strength of the bedrock and the presence of strong limestone interbeds, the sockets may have to be advanced into the rock by churn drilling or rock coring.

Foundation Element	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
East Abutment	71.4 to 72.8	69.5
West Abutment	71.4 to 71.8	69.5

If caisson foundations are adopted, a temporary liner and/or drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and loss of ground. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance/Reaction

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on “socketting” approximately 1 m into the “sound” shale bedrock, below the upper weathered zone. Using the preliminary design elevations given above, and assuming that the caisson subgrade is inspected, the factored axial geotechnical resistance at ULS may be taken as 4,000 kN for a 0.9 m diameter caisson and 6,500 kN for a 1.2 m diameter caisson. The axial geotechnical reaction at SLS (for 25 mm of settlement) will be greater than the factored axial resistance at ULS and as such, the SLS condition does not apply.

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

6.6 Approach Embankments

The Highway 401 approach embankments are up to approximately 3 m to 4 m high relative to the natural ground surface at the site. Based on the General Arrangement drawing provided by AECOM, the approach embankments are proposed to be widened up to about 20 m on the north side of the highway, and up to about 10 m on the south side of Highway 401.



6.6.1 Subgrade Preparation and Embankment Construction

For the widening of the existing embankment, benching into the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (*Benching of Earth Slopes*). The new Highway 401 embankment side slopes should be constructed at a maximum gradient of 2 horizontal to 1 vertical (2H:1V).

Prior to placing any new embankment fill, all topsoil, organic matter and existing fill not comprising part of the existing embankment should be stripped from below the approach embankment areas.

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*).

6.6.2 Slope Stability

Preliminary static slope stability analyses have been completed for the Highway 401 approach embankment, using the commercially available program *Slide 6.0*, from Rocscience, to check that the target minimum factor of safety is achieved. A target minimum factor of safety of 1.3 is normally used in the design of slopes under static conditions per CHBDC 2006. This minimum factor of safety is considered appropriate for the proposed slope widening on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses for an overall 4 m high slope in long-term (effective stress) conditions, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32°	-
Very loose to very dense upper silty sand	20	30°	-
Firm to hard clayey silt	19	30°	50
Compact to dense lower silty sand	20	32°	-

The results of the static global stability analysis indicate that a minimum factor of safety of greater than 1.3 is achieved for 4 m high slopes adjacent to the east and west abutments, oriented no steeper than 2H:1V, as shown on Figure 1. This preliminary assessment of the slope stability of the approach embankments should be reviewed and confirmed during the design-build assignment based on the refined geometry, and additional subsurface information as may be available.

Shallow sloughing could occur on the 2H:1V slope faces, which could be reduced by providing well-vegetated slopes, as recommended in Section 6.6.1.

6.6.3 Settlement Under Widened Embankment Loading

It is understood that no grade raise is proposed for Highway 401, with up to about 20 m and 10 m of widening proposed on the north and south sides of the highway, respectively.



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

Preliminary settlement assessments have been completed for the widening using the commercially available computer program *Settle-3D 2.0* from Rocscience, using the consolidation parameters and estimated elastic deformation moduli given in the table below, based on the results from correlations with the SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Embankment fill	21	-
Very loose to very dense upper silty sand	20	10 MPa
Firm to hard clayey silt	19	15 MPa
Compact to dense lower silty sand	20	50 MPa

Based on this preliminary assessment, the settlement of the foundation soils under the proposed widening is estimated to be less than 25 mm under the approximately 3 m to 4 m high approach embankments. The majority of this settlement is estimated to be completed during and within approximately two to three months following placement of the fill for embankment widening.

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

6.7 Construction Considerations

The following sections identify future construction considerations that may impact the future detail design, and for which provision may be required in the future construction.

6.7.1 Open-Cut Excavation and Temporary Protection Systems

The construction of the pile caps will require excavations up to about 2 m below the existing natural ground surface, and deeper through the Highway 401 approach embankments. These excavations will be made through the existing embankment fill and native upper silty sand deposit. The existing fill and the upper very loose to very dense silty sand can generally be classified as a Type 3 soil, according to the Occupational Health and Safety Act (OHSA) and, as such, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V. All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

Temporary protection systems will be required along the existing Highway 401 lanes to facilitate excavation through the existing highway embankment for widening of the abutments. Protection systems/cofferdams will also be required parallel to the abutments/creek. These temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*). The lateral movement should



meet Performance Level 2 as specified in OPSS.PROV 539, 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.

6.7.2 Groundwater Control

Excavations for the widened abutment pile caps will extend below the groundwater level, which is estimated to be at about Elevation 76.7 m to 77.3 m, consistent with the high water level in Farewell Creek.

At this preliminary stage, it is anticipated that an active dewatering system (such as the use of a system of well points) will be required to lower the groundwater in the upper silty sand deposit to approximately 0.5 m to 1 m below the pile cap excavation level. Further investigation will be required during detailed design to determine the extent of dewatering that will be required for construction.

At this preliminary stage, an accurate prediction of the groundwater pumping volumes cannot be made, as the flow rate would be dependent on whether the contractor includes an interlocking sheetpile cut-off wall between the excavation and Farewell Creek and the duration for which the foundation excavation is open. However, it is considered likely that pumping volumes could exceed 50 m³/day during initial drawdown stages and/or if multiple excavation areas are being dewatered at one time.

It is anticipated that the zone of influence for the 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 upper silty sand deposit. However, the potential for settlement impacts on the existing or new structure foundations and any adjacent utilities should be re-assessed in the detailed design.

6.7.3 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in any future investigation at this site, to further assess the presence of cobbles and boulders and permit the contractor to assess the impact on foundation construction and installation of protection systems.

6.7.4 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition; lower thresholds (between 25 mm/s and 50 mm/s) are applicable where residential and commercial facilities are located nearby. Vibration monitoring is recommended during deep foundation and protection system installation, to demonstrate/confirm that they do not exceed the applicable threshold levels.

6.8 Recommendations for Future Work in Detail Design

During the detailed design phase, additional borehole investigation work is recommended to confirm the following:

- The bedrock surface elevation along the new abutment areas to confirm the founding level for deep foundations.



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

- The properties and thicknesses of near-surface deposits within the footprints of the approach embankments and any associated retaining walls, to develop subexcavation requirements and assess founding levels, global stability and geotechnical resistances, as applicable.
- The stabilized groundwater level at the site and further assess groundwater control requirements.

As noted throughout the report, the preliminary assessment of founding elevation, geotechnical resistances and global stability should be revisited based on the additional borehole information, geometry and other requirements at the detailed design stage.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Matthew Kelly, P.Eng. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact and Principal of Golder, conducted an independent technical and quality control review of this report.

GOLDER ASSOCIATES LTD.



Matt Kelly, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Principal, Designated MTO Foundations Contact

MWK/LCC/sm

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\\golder.gds\gal\mississauga\active\2011\1184\11-1184-0143 urs hwy 401 brock to courtice\6 - reports\15 - farewell creek bridge\11-1184-0143 rpt15 2017-06-16 farewell creek bridge.docx



PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

REFERENCES

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

Ontario Provincial Standard Specifications (OPSS)

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 Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario

Other:

Ontario Regulation 213	Construction Projects (as amended)
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PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

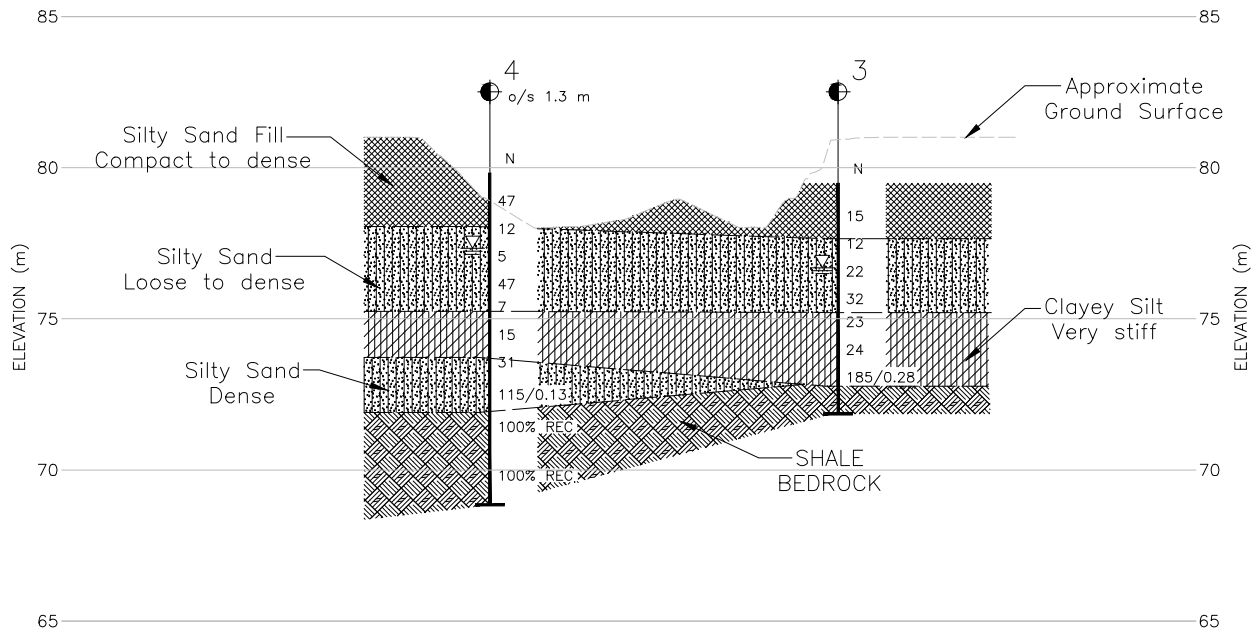
TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> Feasible, but not recommended as excavations of up to 6 m would be required, and greater differential settlement would occur relative to the existing structure 	<ul style="list-style-type: none"> Conventional excavation and construction techniques, although relatively deep excavations would be required to address subexcavation of loose soils (or deeper founding level) 	<ul style="list-style-type: none"> Excavation will extend to nearly 6 m below highway grade and below the groundwater level; groundwater control and protection systems will be required 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations, plus costs for deeper excavation, dewatering and protection systems 	<ul style="list-style-type: none"> Potential for impacts on existing abutment pile caps and piles due to deeper excavation Moderate risk of differential settlement between existing abutments and widening
Driven steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for support of new abutments and wingwalls 	<ul style="list-style-type: none"> Conventional construction methods for H-pile or steel pipe pile foundations Abutment pile caps could be maintained higher than spread footings, reducing depth of excavation, dewatering and protection system requirements 	<ul style="list-style-type: none"> Relatively short piles; care will be required during driving to minimize potential for damage/deflection during driving into shale bedrock 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction, plus dewatering and protection system costs 	<ul style="list-style-type: none"> Low risk of potential for damage/deflection of piles during driving into shale bedrock

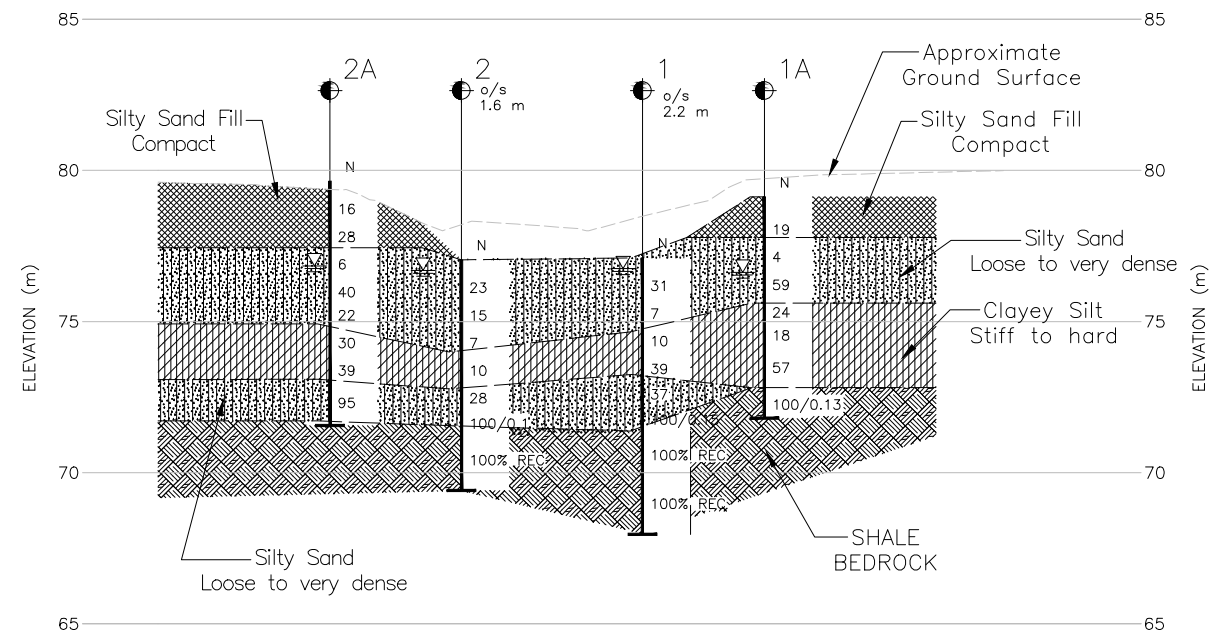


PRELIMINARY FOUNDATION REPORT FAREWELL CREEK BRIDGE REPLACEMENT, W.O. 10-20011

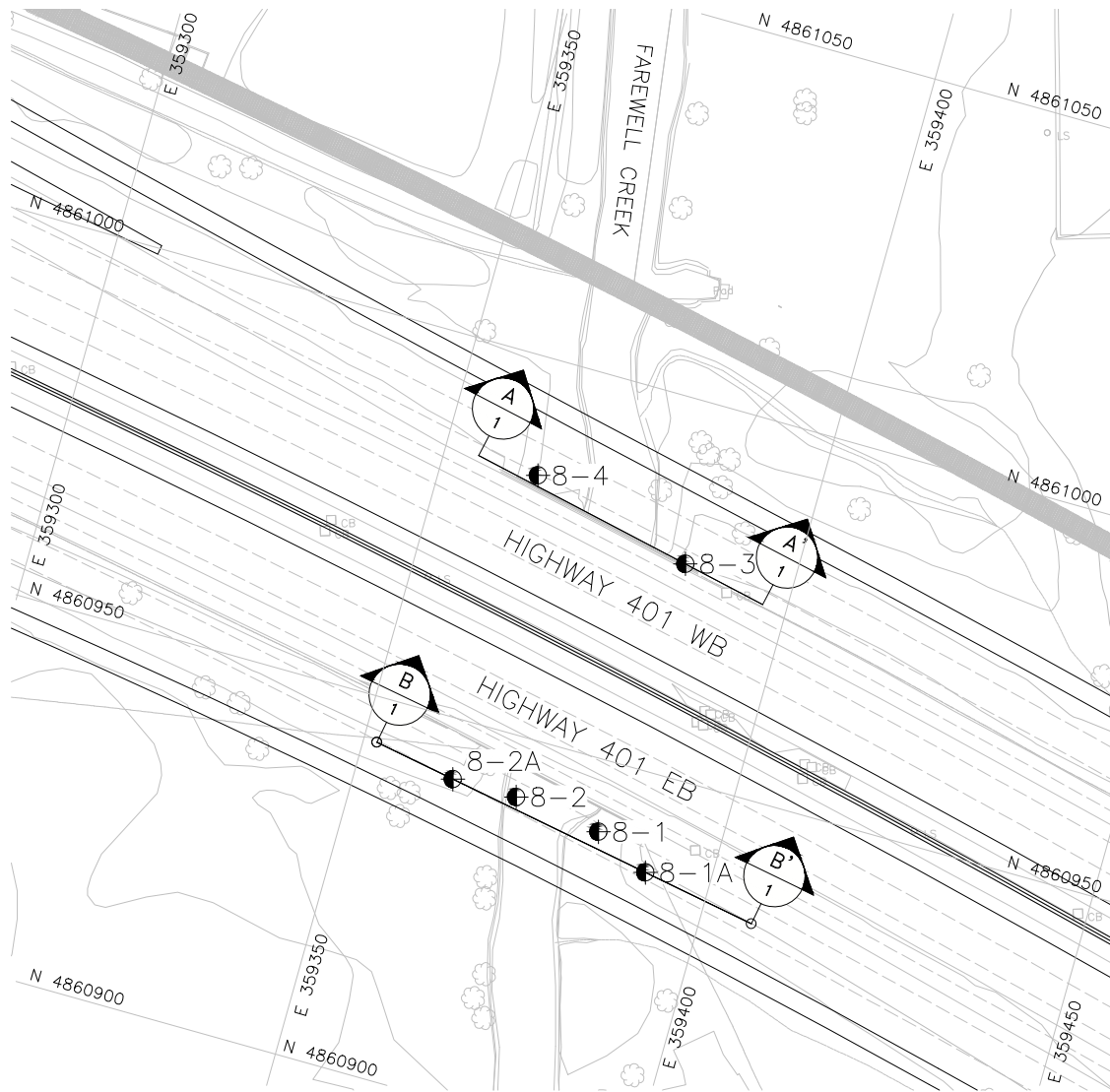
Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none"> Feasible for support of new abutments and wingwalls 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than spread footings, reducing depth of excavation, dewatering and protection system requirements Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles 	<ul style="list-style-type: none"> Caissons would extend below the groundwater level at the site into water-bearing cohesionless soils, with potential for loss of ground Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base Rock coring or churn drilling will be required to form sockets in bedrock 	<ul style="list-style-type: none"> Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for temporary liners 	<ul style="list-style-type: none"> Risk of loosening or disturbing founding soils during construction, although this can be mitigated with the use of temporary liners and placement of concrete by tremie methods Risk of slow progress associated with coring/churn drilling through bedrock, consisting of shale with limestone interbeds



NORTH WIDENING SECTION A-A'



SOUTH WIDENING SECTION B-B'

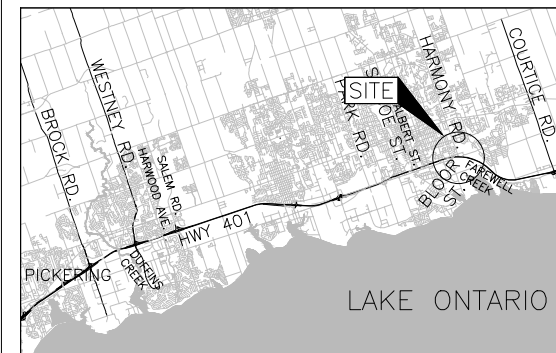


PLAN SCALE
10 0 10 20 m

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

CONT No. WO No. 10-20011

FAREWELL CREEK BRIDGE
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS AND SOIL STRATA



LEGEND

- Borehole - 1972 Investigation - GEOCRE No. 30M15-8
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Core Recovery
- Water Level in open borehole (Nov. 1972)

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
8-1	77.1	4860940.5	359383.7
8-1A	79.1	4860936.9	359391.3
8-2	77.0	4860942.0	359371.8
8-2A	79.6	4860942.0	359363.0
8-3	79.4	4860978.3	359385.3
8-4	79.8	4860984.4	359363.0

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 Preliminary Design Report.

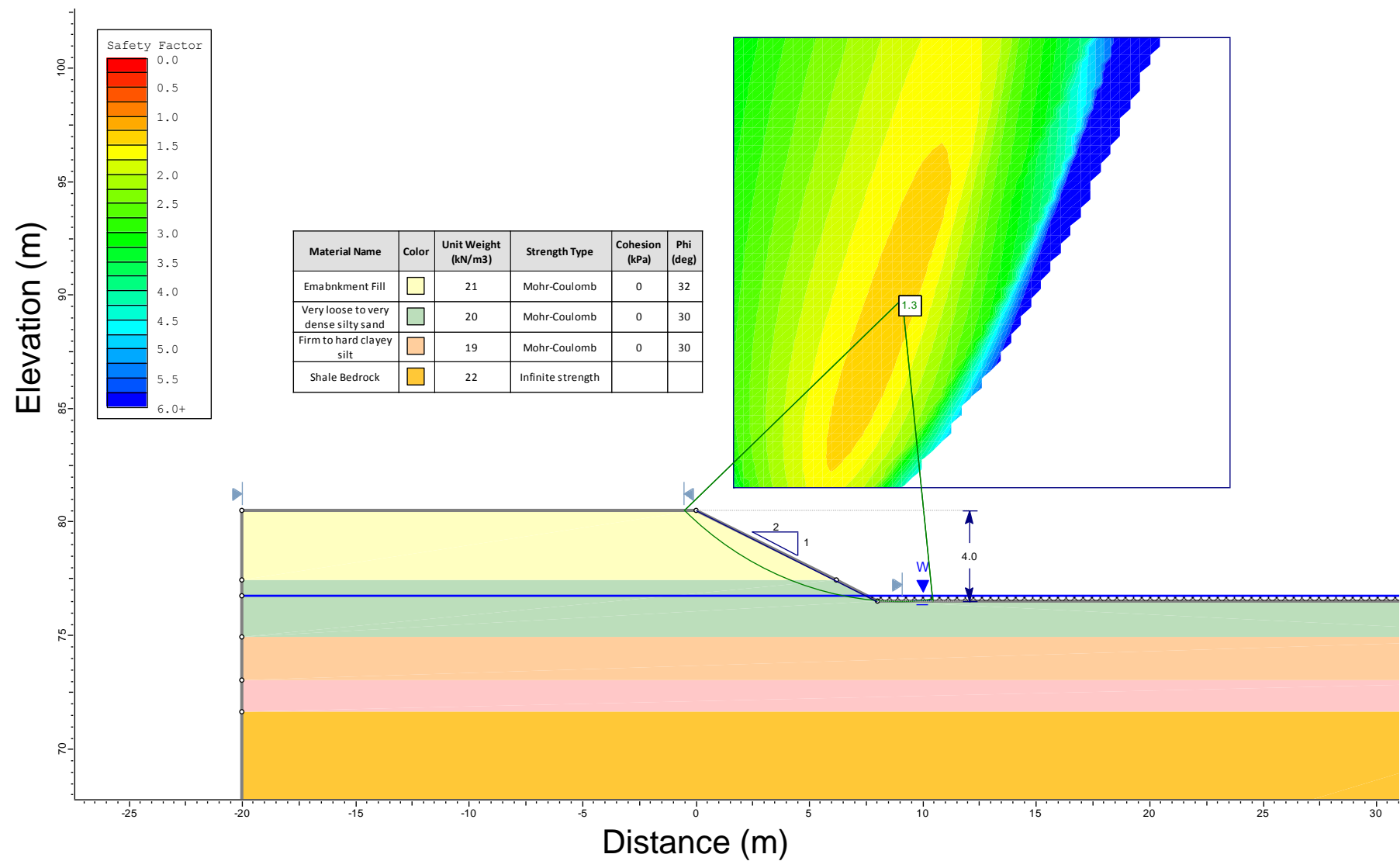
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base.dwg, X-Property.dwg and Street Names.dwg, and the Proposed Design obtained from drawing file x-design_130625.dwg, all dated July 05, 2013, received April 11, 2014.



NO.	DATE	BY	REVISION
Geocres No. 30M15-307			
HWY. 401	PROJECT NO. 11-1184-0143		DIST. CENTRAL
SUBM'D. MWK	CHKD. MWK	DATE: 12/20/2016	SITE: 22-183
DRAWN: JFC/DD	CHKD. MWK	APPD. LCC	DWG. 1





APPENDIX A

Borehole Records and Laboratory Test Results GEOCRES No. 30M15-8

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 72-11128

LOCATION 15,947,240 N. 1,179,025 E.

ORIGINATED BY V.K.

W.P. 44-71-12

BORING DATE November 24, 1972

COMPILED BY G.P.

DATUM Geodetic

BOREHOLE TYPE Auger, BXL Rock Core & Cone Test

 CHECKED BY *[Signature]*

(m)		(ft)		SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT		BULK DENSITY	REMARK
				DESCRIPTION		STRAT. PLOT		BLOWS/FOOT		WATER CONTENT %			
		ELEV. DEPTH				NUMBER		TYPE		SHEAR STRENGTH P.S.F.		Y	
										UNCONFINED		P.C.F.	
										FIELD VANE		GR SA SI	
										LAB VANE			
										100 800 1200 1600 2000			

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 14

JOB 72-11128

LOCATION 15,543,227 N. 1,180,650 E.

ORIGINATED BY J.K.

W.P. 44-72-12

BORING DATE November 29, 1972

COMPILED BY G.P.

DATUM Geodetic

BOREHOLE TYPE Auger & Cone Test

CHECKED BY C.K.

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 30 40 50 100 SHEAR STRENGTH P.S.F. O UNCONFINED * FIELD VANE • QUICK TRIAXIAL * LAB VANE	LIQUID LIMIT — % PLASTIC LIMIT — % WATER CONTENT — % Wp — % — Wl	BULK DENSITY Y P.C.F. GR SA SI.C	REMARKS
ELEV. DEPTH	DESCRIPTION	SIRAL PLOT	NUMBER	TYPE					
71.1 0.0 77.7 1.4	259.4 Ground Level 0.0 Fill, silty sand, some gravel. 255.0 Contact 4.4 Silty sand, trace to some gravel.		1 SS 19						
75.6 3.5	247.9 Loose to Very Dense 11.5 Clayey silt, some sand & occ. gravel.		2 SS 4 3 SS 59 4 SS 21 5 SS 18		250				
72.8 6.3 71.7 7.4	238.7 Very Stiff to Hard 20.7 Weathered Shale 235.4 21.0 End of Borehole		6 SS 57 7 SS 100		240 250	100/7"			

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2

JOB 72-11128

LOCATION 15,547,245 N., 1, 178,986 E.

ORIGINATED BY Y.Y.

W.P. 14-71-12

BORING DATE November 27, 1972

COMPILED BY P.B.

DATUM Geodetic

BOREHOLE TYPE Auger, BKL Rock Core, & Cone Test

CHECKED BY

SOIL PROFILE				SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT		BULK DENSITY	REMARKS	
(m)	(ft)	ELEV DEPTH	DESCRIPTION	STRAT. PILOT	NUMBER	TYPE		BLOWS/FOOT	BLOWS / FOOT		PLASTIC LIMIT			
									20	40	60			80
								SHEAR STRENGTH P.S.F.		WATER CONTENT %		Y		
								O UNCONFINED * FIELD VANE						
								* QUICK TRIAXIAL * LAB VANE						
								100 800 1200 1600 2000		10 20 30		P.C.F.	GR. S.A. (1)	
77.0	0.0	252.6	Ground Level										251.6	
		0.0	Silty sand, trace to some gravel		1	SS	23						76.7m	
					2	SS	15						9 77 12	
74.0	3.0	242.6	Clayey silt, some sand & occasional gravel.		3	SS	7						9 38 38 1	
72.7	4.3	238.6	Very Stiff		4	SS	10						28 62 (10	
71.4		234.1	Compact to Very Dense		5	SS	28							
70.9	6.1	232.6	Weathered		6	SS	100/L"							
69.4	7.6	227.6	Shale Bedrock		7	BKL R.C.	100 REC							
		225.0	Sound											
			End of Borehole											

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2A

JOB 72-11128

LOCATION 15,947,250 N., 1, 178,957 E.

ORIGINATED BY W.

W.P. 44-72-12

BORING DATE November 28, 1972

COMPILED BY G.P.

DATUM Geodetic

BOREHOLE TYPE Auger & Cone Test

CHECKED BY C.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — w_L			BULK DENSITY	REMARK	
(ft) ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT				PLASTIC LIMIT — w_p					
							20	40	60	80	100	WATER CONTENT — w				
							SHEAR STRENGTH P.S.F.				WATER CONTENT %					

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 72-11128

LOCATION 15,947,364 N., 1,175,030 E.

ORIGINATED BY V.K.

W.P. 44-72-12

BORING DATE November 29, 1972

COMPILED BY G.P.

DATUM Geodetic

BOREHOLE TYPE Auger & Cone Test

CHECKED BY C.R.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION / RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L		
79.4 0.0	260.6 Ground Level					260										
77.6 1.8	0.6 Fill, silty sand, some gravel. Compact		1	SS	15	250										5 59 31
75.2 4.2	6.0 Silty sand, trace to some gravel. Compact to Dense		2	SS	12	250										251.6
	246.6 Grey		3	SS	22	250										35 36 (5)
	14.0 Clayey silt, some sand & occasional gravel.		4	SS	32	250										5 16 52
72.7	238.6 Very Stiff		5	SS	23	240										
71.8	22.0 Weathered Shale		6	SS	24	240										
71.8 7.6	235.6 End of Borehole		7	SS	185	240										
	25.0					230										

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 4

JOB 72-11128

LOCATION 15,917,384 N., 1, 178,957 E.

ORIGINATED BY V.K.

W.P. 44-72-12

BORING DATE November 30, 1972

COMPILED BY G.P.

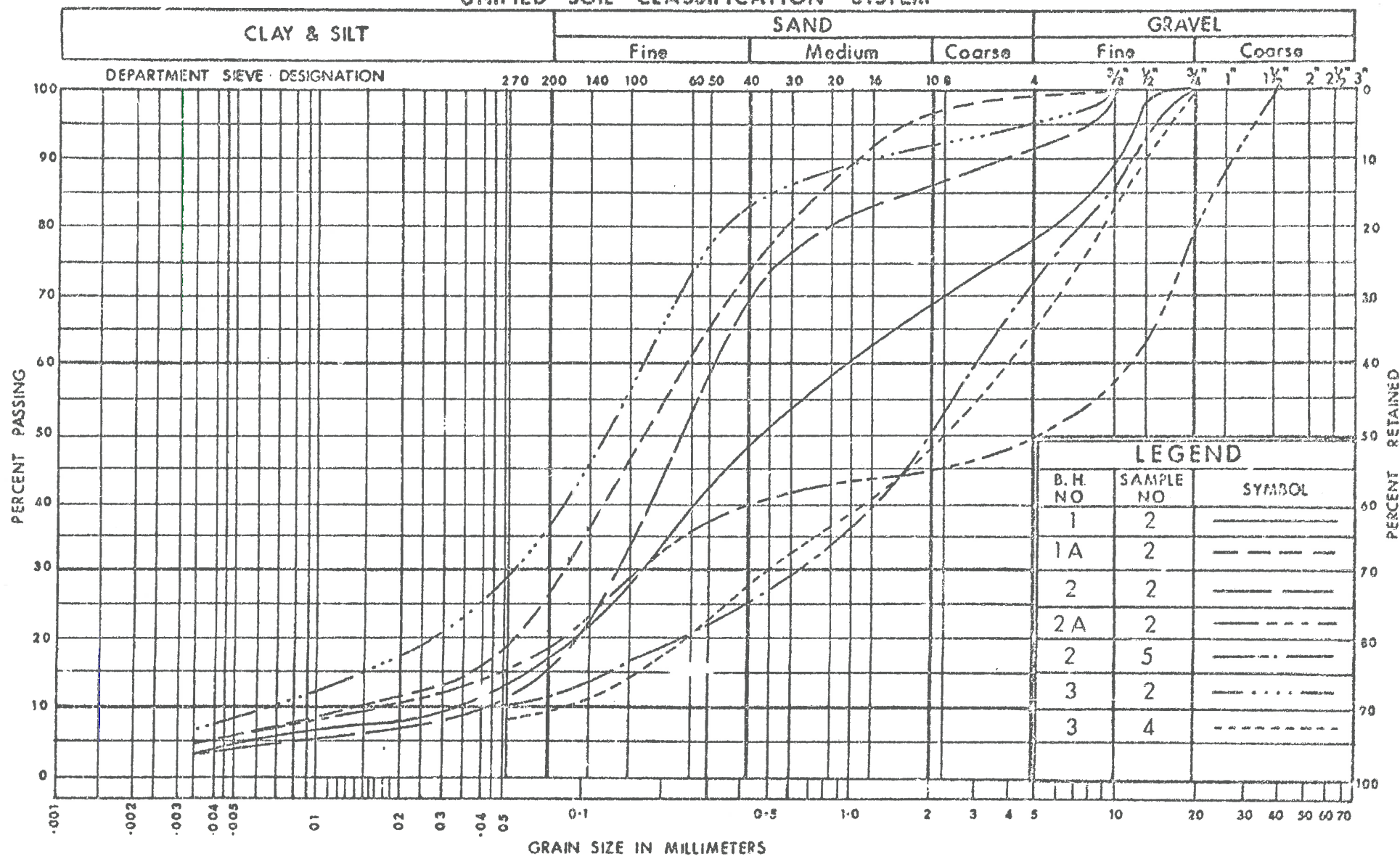
DATUM Geodetic

BOREHOLE TYPE Auger BXL Rock Core & Cone Test

CHECKED BY *SR*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F. GR. SA. SI. CL.	REMARKS	
(ft) ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					w_p — w — w_L WATER CONTENT % 10 20 30
							20	40	60	80	100	UNCONFINED + FIELD VANE QUICK TRIAXIAL x LAB VANE					
							400	800	1200	1600	2000						
261.7	Ground Level					260											
0.0	Fill, silty sand, some gravel.		1	SS	47												
24.9	Compact		2	SS	12												
5.8	Silty sand, some gravel & trace of organics.		3	SS	5												
246.7			4	SS	47												
15.0	Clayey silt, some sand & occasional gravel.		5	SS	7												
241.7	Firm to Stiff		6	SS	15												
20.0			7	SS	31												
235.7	Loose to Dense		8	SS	115/6"												
26.0	Weathered		9	BXL R.C.	100% Rec.												
27.0	Sound Shale Bedrock		10	BXL R.C.	100% REC.												
225.7																	
36.0	End of Borehole																
						220											

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

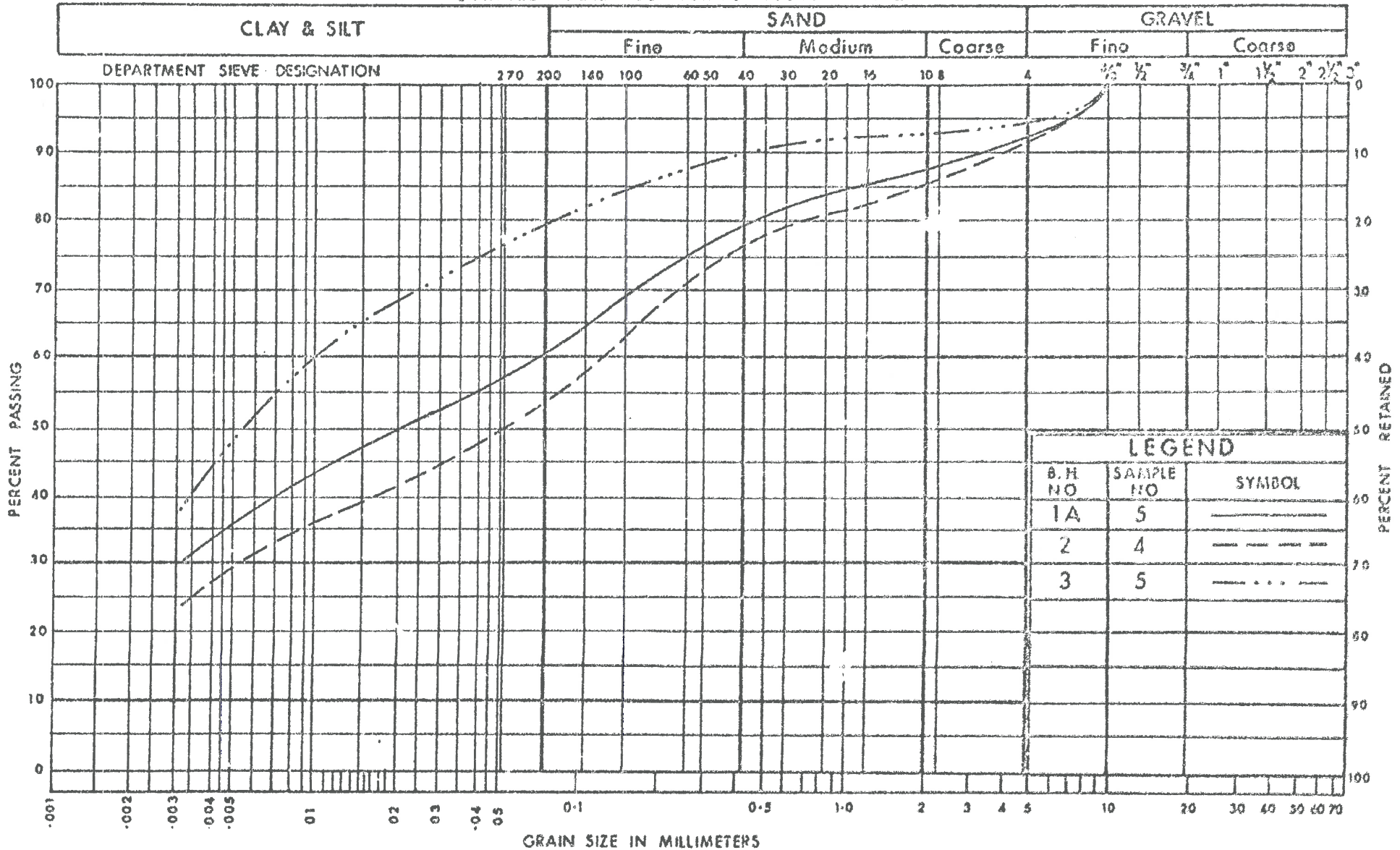
GRAIN SIZE DISTRIBUTION
SILTY SAND
TRACE TO SOME GRAVEL

V.P. No. 44-71-12

JOB No. 72-11128

FIG. 1

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



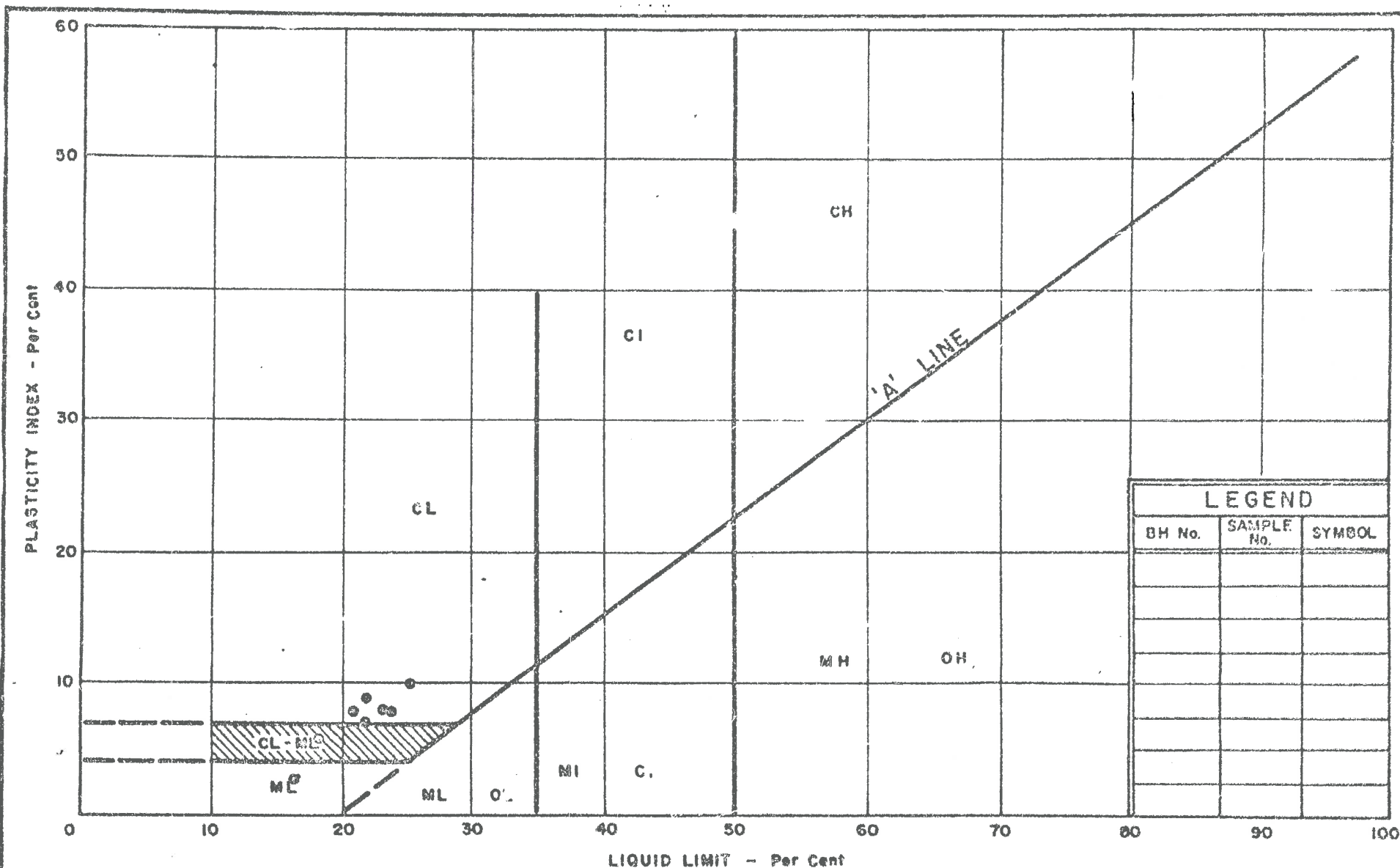
DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
SOME SAND & GRAVEL

W.P. No. 44-71-12

JOB No. 72-11128

FIG. 2



LEGEND		
BH No.	SAMPLE No.	SYMBOL



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART CLAYEY SILT SOME SAND & GRAVEL

WP No. 44-71-12
JOB No. 72-11128
FIG. 3

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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