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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Wilson Road Overpass Replacement Structure Site No. 22-180 Highway 401 Widening from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

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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 Subsoil Conditions	2
4.2.1 Topsoil	3
4.2.2 Clayey Silt to Silty Clay Fill.....	3
4.2.3 Silt and Sand to Sand and Gravel/Clayey Silt – Glacial Till.....	3
4.2.4 Silty Sand to Sand.....	4
4.2.5 Groundwater Conditions	4
5.0 CLOSURE.....	5

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS	6
6.1 General.....	6
6.2 Foundation Options	6
6.3 Shallow Foundations	7
6.3.1 Founding Elevations.....	7
6.3.2 Geotechnical Resistance/Reaction	8
6.4 Driven Steel H-Pile or Steel Pipe Pile Foundations	8
6.4.1 Founding Elevations.....	8
6.4.2 Axial Geotechnical Resistance/Reaction.....	9
6.5 Caisson Foundations	10
6.5.1 Founding Elevation	10
6.5.2 Geotechnical Axial Resistance/Reaction.....	10
6.6 Retained Soil System (RSS) Walls.....	11
6.6.1 Founding Elevations.....	11



PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

6.6.2	Geotechnical Resistance/Reaction	11
6.6.3	Global Stability of RSS Walls	11
6.6.4	Settlement.....	12
6.7	Construction Considerations.....	12
6.7.1	Excavation and Temporary Protection Systems	12
6.7.2	Groundwater Control.....	12
6.7.3	Subgrade Protection	13
6.7.4	Obstructions.....	13
6.7.5	Vibration Monitoring During Pile Installation.....	13
6.8	Recommendations for Further Work During Detail Design	13
7.0	CLOSURE.....	14

REFERENCES

TABLES

Table 1 Comparison of Replacement Structure Foundation Alternatives

DRAWINGS

Drawing 1 Wilson Road Overpass, Highway 401 Improvements – Borehole Locations
Drawing 2 Wilson Road Overpass, Highway 401 Improvements – Soil Strata

APPENDIX A Borehole Records and Geotechnical Laboratory Test Results 1973 Investigation (GEOCRE No. 30M15-32)

Records of Boreholes 32-1, 32-2, 32-3 and 32-4
Figure 1 – Grain Size Distribution – Glacial Till
Figure 2 – Grain Size Distribution – Sand



PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
WILSON ROAD OVERPASS
STRUCTURE SITE NO. 22-180
HIGHWAY 401 WIDENING 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 widening of Highway 401 from Brock Road easterly to Courtice Road in the Regional Municipality of Durham, Ontario.

This report addresses the geotechnical aspects of the proposed replacement and widening of the existing Wilson Road overpass. It was prepared with information from a previous geotechnical/foundations investigation at the Wilson Road overpass site:

- **MTO GEOCRES No. 30M15-32:** Report titled “Proposed Widening of Existing Overpass Structure at the Crossing of Hwy 401 and Wilson Road, City of Oshawa, County of Ontario” by Ministry of Transportation and Communications- Ontario, dated August 28, 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 AECOM's *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 Wilson Road overpass is located between the Ritson Road overpass to the west and the Harmony Road interchange to the east, in the City of Oshawa, in the Regional Municipality of Durham. The existing overpass is an approximately 12.8 m long, single-span structure that was constructed in 1951. The structure was widened under Contract 77-133 by approximately 1.9 m to the north, and 5.6 m to the south. Based on the *General Plans and Details* drawings for the existing structure, dated February 14, 1951, the abutments are supported on spread footings, founded at approximately Elevation 87.9 m.

The surface topography in the vicinity of the site is generally flat-lying to undulating between approximately Elevation 90 m and 97 m, generally sloping gently downward to the east-northeast. The natural ground surface on either side of Wilson Road is at approximately Elevation 92 to 93 m; the Wilson Road grade within the limits of the overpass abutments is cut into the natural terrain to approximately Elevation 89.4 m. Highway 401 has been constructed on an embankment about 3 m high, with the highway grade at approximately Elevation 94.7 m to 94.6 m, declining toward the east.

3.0 INVESTIGATION PROCEDURES

Four boreholes were advanced at this site as part of a previous geotechnical/foundation investigation by MTO in 1973, in support of the widening of the Wilson Road overpass (MTO GEOCRES Report No. 30M15-32). For the purposes 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. 30M15-32 is referred to throughout this report and on the drawings as Borehole 32-1. The approximate borehole locations are shown on Drawing 1; these borehole locations have been interpreted



based on converting the NAD 27 coordinates shown on the borehole records to NAD 83 coordinates, as well as by scaling measurements from the plan shown in the 1973 GEOCRETS report.

The boreholes were drilled with a continuous flight auger machine using solid stem augers. The boreholes extended to depths ranging from 10.7 m to 11.1 m. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm nominal outside diameter split-spoon sampler driven by a manual hammer, in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions were observed in the open boreholes during drilling.

Index and classification testing (water content, Atterberg limits and grain size distributions) was completed on selected samples.

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* (Brennand, 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.

Bedrock underlying the City of Oshawa is Ordovician shales of the lower Whitby formation, alternately known as Collingwood shale, as indicated in *Aggregate Resources Inventory of the City of Oshawa, Regional Municipality of Durham, Southern Ontario* (Scott and Billings, 1981). The lower Whitby formation is described as a black fossiliferous and highly petroliferous calcareous shale.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the 1973 investigation, together with the results of in situ and laboratory testing, are presented on the borehole records and figures provided in Appendix A. Interpreted stratigraphic profiles along the westbound and eastbound lanes are shown on Drawing 2.

The stratigraphic boundaries shown on the borehole records and on Drawing 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions are expected to vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of a surficial cohesive fill layer, underlain by a glacial till deposit of slight to low plasticity, which has been interpreted as a silt and sand/sand and gravel to clayey silt till. The glacial till is in turn underlain by a deposit of silty sand to sand; a thin layer of silty clay was encountered below the



glacial till and atop the silty sand to sand deposit in one of the boreholes. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Approximately 210 mm of topsoil was encountered immediately below the ground surface at the time of the 1973 investigation in Boreholes 32-1 and 32-2, located east of Wilson Road.

4.2.2 Clayey Silt to Silty Clay Fill

Fill was encountered immediately below the 1973 ground surface at Boreholes 32-3 and 32-4, on the west side of Wilson Road. The fill extended to depths of about 1.5 to 1.9 m below the ground surface (Elevation 88.0 m to 88.3 m).

The cohesive fill contains some sand and gravel, based on the descriptions given on the borehole records. Atterberg limits tests on two samples of the fill measured plastic limits of about 18 and 24 per cent, liquid limits of about 29 and 40 per cent, and plasticity indices of about 11 and 16 per cent. These results indicate the fill is a clayey silt to silty clay of low to intermediate plasticity. Natural water contents were measured on two samples of the fill, at about 22 and 28 per cent.

Two Standard Penetration Test (SPT) "N"-values of 10 and 11 blows per 0.3 m or penetration were measured in the fill, suggesting it has a stiff consistency.

4.2.3 Silt and Sand to Sand and Gravel/Clayey Silt – Glacial Till

A glacial till deposit was encountered underlying the topsoil in Boreholes 32-1 and 32-2, and underlying the fill in Boreholes 32-3 and 32-4. The till surface was encountered within the cut at approximately Elevation 89.6 m to 89.7 m on the east side of Wilson Road, and about Elevation 88.0 m to 88.3 m on the west side of Wilson Road. The till was fully penetrated in all boreholes, and the deposit is approximately 7.4 m to 8.3 m thick, with its base between about Elevation 80.1 m and 81.4 m.

The deposit is described in the 1973 report as being of glacial origin comprised of a heterogeneous mixture of silt, sand and gravel, as well as zones of clayey silt. Grain size distributions on nine samples collected from this deposit indicate it is composed of 9 to 57 per cent gravel sizes, 32 to 62 per cent sand sizes, and 5 to 56 per cent fines; an envelope of grain size distributions for the till deposit is presented on Figure 1 in Appendix A, demonstrating the variability of this deposit. Atterberg limits tests were completed on ten samples of the till, and measured plastic limits ranging from 9 to 13 per cent, liquid limits of 13 to 21 per cent, and plasticity indices of 3 to 9 per cent; these test results suggest that the cohesive portion of the till consists of clayey silt of low plasticity, although the deposit grades to slightly plastic or non-plastic. A thin layer of silty clay near the base of the till in Borehole 32-2 had a plastic limit of 30 per cent, a liquid limit of approximately 50 per cent, and a plasticity index of approximately 20 per cent. The natural water content of soil samples collected from this deposit ranges from about 5 to 13 per cent, generally below or near the plastic limit for the material.

The SPT "N" values measured within the glacial till deposit range from 6 blows to greater than 100 blows per 0.3 m of penetration. In general, the SPT "N" values are all greater than 100 blows per 0.3 m of penetration in the boreholes on the east side of Wilson Road, with the exception of one value of 34 blows per 0.3 m of penetration above approximate Elevation 88.3 m in Borehole 32-1. On the west side of Wilson Road, the SPT "N" values generally increased from 6 and 7 blows per 0.3 m of penetration above Elevation 86.6 m, to about 25 to 87 blows per 0.3 m of penetration with depth; SPT "N" values of greater than 100 blows per 0.3 m of



PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

penetration were generally measured below approximately Elevation 84 m. The till therefore is loose/firm near the surface of the deposit on the west side of Wilson Road, increasing with depth to a very dense/hard nature.

4.2.4 Silty Sand to Sand

A deposit of sand to silty sand was encountered underlying the glacial till deposit. The surface of this stratum was inferred at about Elevation 81.3 m to 81.4 m on the east side of Wilson Road, and at about Elevation 80.1 m to 80.7 m on the west side of Wilson Road. All boreholes were terminated within this deposit after penetrating it for a thickness of 1.4 m to 2.3 m.

Grain size distributions on four samples from this deposit indicate it is composed of 2 to 35 per cent gravel sizes, 54 to 84 per cent sand sizes, and 11 to 33 per cent fines, as indicated on the borehole records and on the envelope of grain size distributions on Figure 2 in Appendix A. The deposit varies in composition from silty sand to sand containing trace to some silt and trace to some gravel; one sample would be described as sand and gravel. The natural water content measured on selected soil samples ranges from about 2 to 6 per cent.

The SPT "N" values recorded within this deposit range from 56 blows to greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

4.2.5 Groundwater Conditions

The groundwater level was noted in the open boreholes during and immediately following completion of the drilling operations in July 1973. The water levels recorded in the open boreholes are summarized below:

Borehole No.	1973 Ground Surface Elevation (m)	Depth to Water Level Below 1973 Ground Surface (m)	Groundwater Elevation (m)	Date
32-1	89.8	1.1	88.7	July 16, 1973
32-2	89.9	1.8	88.1	July 18, 1973
32-3	89.9	1.4	88.5	July 18, 1973
32-4	89.8	0.9	88.9	July 16, 1973

Based on these observations, it appears that the water level at the site is at approximately Elevation 88 m to 89 m, which is just below the existing Wilson Road cut grade of Elevation 89.4 m. However, these 1973 groundwater levels may not represent the stabilized groundwater level at the site, nor the current groundwater regime. The groundwater level is expected to fluctuate seasonally and to be higher during wet periods of the year.



5.0 CLOSURE

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

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
WILSON ROAD OVERPASS
STRUCTURE SITE NO. 22-180
HIGHWAY 401 WIDENING FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



6.0 DISCUSSION AND PRELIMINARY FOUNDATION 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 Wilson Road overpass (MTO Structure Site 22-180). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a 1973 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 the construction or design-build contractor. 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. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect 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 single-span Wilson Road overpass will be replaced with a new single-span overpass structure that is both longer and wider.

The existing rigid frame, single-span overpass was constructed in the 1950s, and widened in 1975 by about 1.9 m to the north and 5.6 m to the south, with the widening matching the original structure. Based on the available design drawings, the existing overpass is supported on spread footings that are founded at approximately Elevation 87.9 m (288.5 ft.), about 1.5 m below the existing Wilson Road cut grade.

Based on the preliminary General Arrangement (GA) drawing provided by AECOM, Wilson Road is proposed to be widened from two to four lanes plus pedestrian sidewalks on both sides, resulting in a new longer span length of about 27.5 m. Highway 401 and the new overpass will be widened by about 11 m on the north side and about 8 m on the south side, relative to the existing structure. New wingwalls/retaining walls will be required parallel to Highway 401, to minimize impacts on the adjacent commercial and residential properties.

Both shallow and deep foundation options have been considered for support of the new Wilson Road overpass. 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 or spread footings founded on the compact to very dense or very stiff to hard till deposit:**
Shallow strip or spread footing foundations are feasible for support of the new structure. This foundation type would preclude the use of integral abutments, but could permit semi-integral abutments. However, subexcavation of firm soil materials (such as encountered in Borehole 32-3) would be required if such soils are present below the west abutment location. Such subexcavation could extend to about Elevation



PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

86.5 m, up to about 3 m below the existing Wilson Road grade; temporary protection systems would be required along Highway 401, as well as in front of and potentially behind the footing excavation. The subexcavation and the proposed footing founding level would extend below the groundwater level at the site (as measured during the 1973 investigation), and groundwater control is expected to be required in the silty sand till to stabilize the subgrade and enable shallow foundations to be constructed in “dry” conditions.

- **Footings “perched” on a compacted granular pad in the approach embankment:** At the abutments, footings “perched” in the approach embankments above the Wilson Road grade are feasible for support of the new abutments and associated wing walls. However, a longer structure span would be required to construct abutment foreslopes for an “open” structure configuration, and the structural costs would be much higher. Therefore, this option is not detailed further in this report.
- **Driven steel H-piles or pipe piles founded within the “100-blow” soils:** Driven steel H-piles or steel pipe piles are feasible for support of the new abutments, and would permit design of conventional abutments, semi-integral abutments (for pipe piles) or integral abutments (for H-piles). A perched pile cap, in conjunction with integral abutments in a false abutment configuration, would minimize excavation and groundwater control requirements at the new abutment locations; the existing Highway 401 embankment would need to be excavated to just below the Wilson Road grade, but not as deep as would be required for spread footings (particularly as related to the potential for subexcavation for strip footings at the west abutment). Very dense “100-blow” soil will be encountered at shallow depths below the Wilson Road cut grade, and pre-augering may be required to ensure that the piles penetrate to adequate depth, remain aligned, and are not damaged. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense till deposit.
- **Caissons founded within the “100-blow” soils:** Caissons are considered feasible for the support of the abutments; however this option would preclude integral abutment design. This option will be more expensive than either shallow foundations or pile foundations, although fewer caisson elements would be required in comparison to the number of steel piles that would be required. If caissons are adopted for support of the abutments, they would extend into and through water-bearing non-cohesive till zones, and temporary liners would be required during construction to control potential ground losses and/or disturbance at the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible and appropriate for the support of the new abutments. However, driven pile foundations are preferred from a geotechnical/foundations perspective as they would permit integral abutments, and a perched pile cap would eliminate the potential requirement for up to 3 m of subexcavation at the west abutment, and minimize groundwater control requirements as compared with spread footings.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the abutments and associated wingwalls and concrete retaining walls (if adopted) for the new overpasses, spread/strip footings should be founded on the dense to very dense/hard portions of the till deposit. Concrete retaining walls beyond the wingwalls may also be founded on the native soils above the Wilson Road cut grade, although further borehole investigation will be required during detailed design to assess the nature



PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

and properties of these soils and confirm the applicable foundation recommendations. Strip or spread 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 the footing, rigid Styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

The existing and proposed Wilson Road grade is at approximately Elevation 89.4 m. As such, the maximum (highest) founding elevation for the preliminary design of the abutment footings is approximately Elevation 88.2 m. However, based on the presence of firm soils as encountered in Borehole 32-3 near the west abutment, local subexcavation could be required down to approximately Elevation 86.5 m, subject to confirmation by further investigation during detail design. The subexcavation should be backfilled with compacted Ontario Provincial Standard Specification (OPSS).PROV 1010 Granular A or Granular B Type II fill prior to construction of the footings at a higher elevation.

6.3.2 Geotechnical Resistance/Reaction

The following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of spread/strip footing founded on the properly prepared sandy silt to silty sand till deposit, or on compacted granular fill following subexcavation of firm soils. These recommendations apply for the new abutment footings, and for retaining wall footings founded below the Wilson Road grade, and are based on an assumed 3 m footing width. Lower geotechnical resistances may apply for the retaining walls above the Wilson Road grade, although further investigation of the soils above this level will be required in detailed design.

Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
Footing on properly prepared dense to very dense/hard till	500	400

Note: The geotechnical resistance/reaction values given above are estimated for a 3 m wide strip footing.

The geotechnical resistances provided above are given for loads will that 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 Table 10.2 in *CFEM* (2006). These preliminary geotechnical resistances will have to be re-evaluated during detail design, subject to additional borehole and groundwater information within the footprint of shallow foundation elements, if adopted.

6.4 Driven Steel H-Pile or Steel Pipe Pile Foundations

6.4.1 Founding Elevations

The abutments and associated wingwalls for the replacement structure may be supported on steel piles driven to found within the “100-blow” till deposit. For preliminary design purposes, if integral abutments are adopted for the design of the structure replacement, it has been assumed that the pile caps would be “perched” above the Wilson Road grade within the approach embankments, at about Elevation 92 m. The existing Highway 401 approach embankments and adjacent ground would have to be excavated to near the Wilson Road grade



PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

(approximately Elevation 89.4 m) for construction of the RSS walls including placement of corrugated steel pipe (CSP) sections.

Based on the 1973 borehole results, the pile tip elevation is expected to vary. Although Boreholes 32-1 and 32-2 near the east abutment encountered 100-blow soil throughout nearly their full depth below the Wilson Road cut grade, the surface of such soil was deeper, below about Elevation 84 m, in Boreholes 32-3 and 32-4 near the west abutment. The following pile tip elevations are recommended for preliminary design; further investigation will be required at the proposed abutment locations during detailed design, to confirm the tip elevation for pile foundations.

Foundation Element	Approximate Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)
West Abutment	84	81
East Abutment	88.5	82.5

Based on the above elevations, the proposed piles are estimated to be approximately 9.5 m to 11 m long, although longer pile lengths may be determined based on additional investigation during detailed design. Given the shallow depth to 100-blow soil, some pre-augering is expected to be necessary at the east abutment prior to driving the piles and this requirement and the potential cost implications should be evaluated further during detail design.

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. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSP 3000.100 (*Steel H-Pile Driving Shoe*) or OPSP 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, as encountered at this site, driving shoes (such as Titus Standard 'H' Bearing Pile Points) are preferred over flange plates.

6.4.2 Axial Geotechnical Resistance/Reaction

For HP 310x110 piles driven to the design tip elevations given above, the factored axial geotechnical resistance at ULS may be taken as 1,400 kN. The axial geotechnical reaction at SLS may be taken as 1,200 kN for 25 mm of settlement. The same axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12-¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.). These preliminary geotechnical resistances/reactions will have to be re-evaluated and modified, as necessary, during the detailed design in consideration of the pile cap elevation and additional subsurface investigation at the foundation elements.

Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria



PRELIMINARY FOUNDATION REPORT

WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

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*) during the final stages of driving to verify that the required ultimate capacity has been achieved.

6.5 Caisson Foundations

6.5.1 Founding Elevation

Caissons founded within the "100-blow" till soils may be considered for support of the abutments for the proposed replacement structure. The following caisson founding elevations may be used for preliminary design purposes:

Foundation Element	Approximate Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)
West Abutment	84	81
East Abutment	88.5	82.5

The caissons will extend into and through water-bearing non-cohesive till, and potentially near the water-bearing lower silty sand to sand deposit. The groundwater pressure associated with the lower silty sand to sand deposit is not known based on the results from the 1973 investigation, and further assessment of this aspect will be required during detailed design if caissons are adopted, to confirm requirements for basal stability. If caisson foundations are adopted, a temporary liner will be required to support the overburden soils during construction, in conjunction with drilling mud to balance groundwater pressures to control base disturbance. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance/Reaction

The following factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) may be used for design of caisson foundations:

Caisson Diameter (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS for 25 mm of Settlement (kN)
0.9	2,500	2000
1.2	4,500	3,500

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



6.6 Retained Soil System (RSS) Walls

If perched pile caps are used in a false abutment configuration, and for retaining walls adjacent to the abutments and wingwalls at this site, retained soil system (RSS) walls are a suitable and feasible alternative to conventional concrete retaining walls supported on shallow foundations. In fact, they are advantageous in that they would minimize the depth of excavation through the existing Highway 401 embankment and surrounding soils to just below the Wilson Road grade, as compared to the greater depth required for frost protection for strip footings.

6.6.1 Founding Elevations

The front facing panels and the reinforced soil mass of the RSS wall should be founded below any existing topsoil or unsuitable fill soils. There is a potential requirement for some subexcavation of firm soils in the vicinity of the west abutment within the footprint of the RSS walls; however, further investigation and assessment is recommended during detail design to confirm this requirement, based on the encountered conditions together with the abutment wall/RSS wall geometry and height.

Typically, the front facing panels are supported on a footing and/or granular levelling pad at a shallow depth below the ground surface in front of the wall. It is recommended that the facing panels be founded at a minimum depth of 0.5 m below the lowest surrounding grade, in accordance with MTO's *RSS Design Guidelines*. The levelling pad should consist of a minimum thickness of 0.3 m of compacted OPSS.PROV 1010 Granular A, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V.

6.6.2 Geotechnical Resistance/Reaction

For the RSS facing panels founded on compacted granular fill as described above, preliminary design may be completed based on a factored geotechnical resistance at ULS of 150 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

The maximum RSS wall height in front of or adjacent to the abutments and wing walls is estimated to be on the order of 6 m. Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for preliminary design), a factored geotechnical resistance at ULS of 500 kPa and a geotechnical reaction at SLS of 350 kPa (for 25 mm of settlement) may be used for preliminary design; these values assume subexcavation and replacement of firm soils, if present.

The preliminary geotechnical resistance/reaction values should be reviewed and revised during detail design after the RSS wall configuration and any "step" elevations are confirmed, taking into account additional subsurface information above the Wilson Road cut grade.

6.6.3 Global Stability of RSS Walls

For retaining walls constructed on the founding conditions noted above, the subsoils beneath the retaining walls will consist of hard or very dense till or dense engineered fill. The factor of safety against global instability of RSS walls, if adopted, will be greater than 1.5. However, this preliminary assessment of the global stability of RSS walls should be reviewed and confirmed as part of the detail design, once the wall geometry (in particular the presence and height of any sloping ground) is refined and further borehole information is obtained within the footprint of the walls, to characterize the soils above the Wilson Road cut grade.



It should be noted that the internal stability of a reinforced earth structure is to be assessed by the proprietary product designer.

6.6.4 Settlement

At this preliminary stage, it is estimated that for widened, approximately 3 m high approach embankments on Highway 401, the settlement of the underlying soils will be less than about 25 mm. This settlement is expected to be completed essentially during construction. Based on this preliminary estimate, it is anticipated that the settlement performance for RSS walls and facing panels will be acceptable; however, this preliminary assessment of RSS settlement should be reviewed and confirmed based on the subsoil conditions encountered beneath the proposed retaining walls during detail design.

6.7 Construction Considerations

The following sections identify construction considerations that may impact the future detail design, and for which provision may be required in the contract documents produced as part of detail design.

6.7.1 Excavation and Temporary Protection Systems

The construction of spread/strip footings would require excavations to about 1.2 m below the existing Wilson Road grade. If integral abutment foundations are adopted, excavation will still be required to at near the Wilson Road grade for construction of RSS walls. For both options, some subexcavation (on the order of 3 m) may be required at the west abutment. These excavations will be made through the existing Highway 401 embankment fill and native till deposit, and they are expected to extend below the groundwater table. The existing fill and native soils above the Wilson Road grade are most likely classified as a Type 3 soil, while the native very dense/hard till deposits are classified as Type 2 soils, provided they are dewatered, according to the Occupational Health and Safety Act (OHSA). 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 eastbound and westbound lanes to facilitate the removal of the existing structure and staged construction of the new, longer-span structure. Shallow temporary protection systems may also be required parallel to Wilson Road. 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. Conceptually, a driven, interlocking sheetpile system may not be suitable at this site, based on the relatively shallow depth to 100-blow till. A pre-augered soldier pile and lagging system is likely more feasible from a constructability perspective, but would likely require additional groundwater control measures if excavations extend below the seasonal groundwater level, as well as measures to mitigate loss of soil particles through the lagging boards if seepage is encountered.

6.7.2 Groundwater Control

The groundwater level was measured in the 1973 investigation between approximately Elevation 88 m and 89 m, near or just below the existing Wilson Road grade. It is anticipated that perched pile caps for an integral



abutment can be maintained above the groundwater level, but that excavations for new retaining wall footings or RSS walls (including subexcavation where necessary at the west abutment) would extend to near or below the groundwater level. However, additional borehole investigation is recommended to confirm the current groundwater level at the site during detailed design.

At this preliminary stage, it is anticipated that an active dewatering system (such as a system of well points or eductors) will be required to lower the groundwater in the fine-grained till deposit to approximately 0.5 m to 1 m below the proposed wall founding level, to maintain a stable subgrade during subexcavation (if required), and/or for the construction of concrete footings. An accurate prediction of the groundwater pumping volumes cannot be made based on the 1973 borehole information; in addition, 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. However, it is considered that it may be possible to maintain pumping volumes at less than 50 m³/day.

At this preliminary stage, 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 very dense soils that are present at this site. However, the potential for settlement impacts on the existing or new structure foundations and any adjacent utilities should be re-assessed at the detailed design phase.

6.7.3 Subgrade Protection

The native soils that will be exposed within the excavations at the subgrade level for concrete foundations will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the foundation subgrade within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

6.7.4 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.

6.7.5 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 are applicable for nearby residential and commercial facilities (between 25 mm/s and 50 mm/s). If pile driving is adopted at the abutments, then vibration monitoring is recommended adjacent to the abutment areas to demonstrate/confirm that vibration levels do not exceed the thresholds.

6.8 Recommendations for Further Work During Detail Design

During the detail design phase, additional geotechnical/foundation investigation is recommended to confirm or assess the following:

- The elevation of the “100-blow” soil within the footprint of the proposed abutment locations, to confirm the design pile tip elevations.



PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

- The properties of any near-surface firm/stiff/loose materials, to confirm subexcavation requirements for footings and/or retaining walls.
- The subsurface conditions above the Wilson Road grade within the footprint of the retaining walls and the embankment widening areas, to confirm the geotechnical resistances, settlement assessment and global stability.
- The current groundwater levels at the site, for more detailed assessment of the groundwater control requirements and measures during construction.
- The properties of the existing Highway 401 embankment fill, for design of protection systems.

7.0 CLOSURE

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

GOLDER ASSOCIATES LTD.



Nikol Kochmanová, P.Eng.
Geotechnical Engineer



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

AKV/NK/LCC/sm

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PRELIMINARY FOUNDATION REPORT WILSON ROAD OVERPASS REPLACEMENT, W.O. 10-20011

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 539	Construction Specification for Temporary Protection Systems
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 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario



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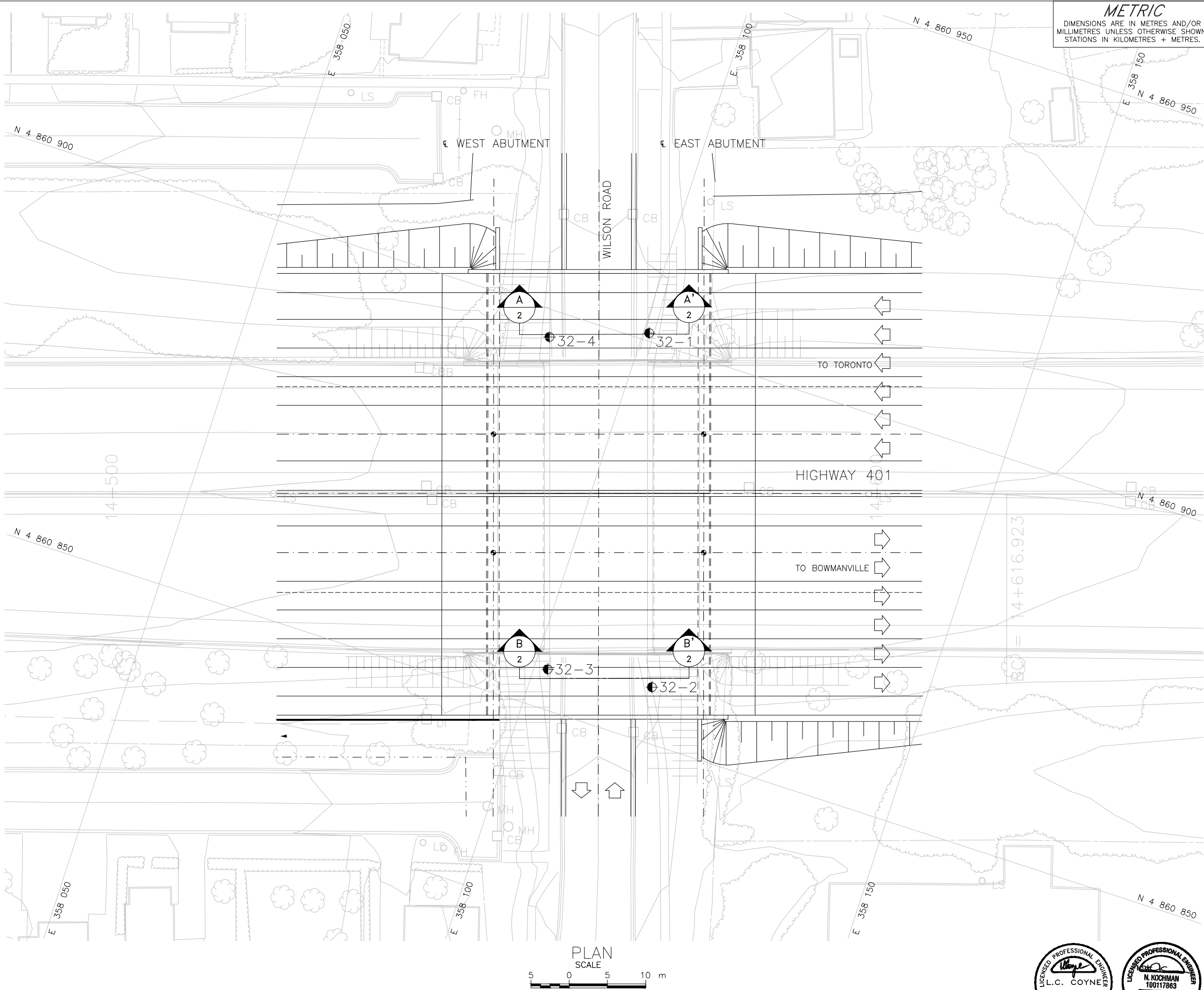
TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> Feasible for support of the new abutments, and for support of concrete retaining walls 	<ul style="list-style-type: none"> Conventional excavation and construction techniques Very dense soils (with SPT “N” values greater than 100 blows per 0.3 m of penetration) present at shallow depth, with good geotechnical resistance and settlement performance 	<ul style="list-style-type: none"> Some subexcavation required at west abutment due to presence of firm soils Excavation will extend below the groundwater level and groundwater control will be required Significant temporary protection systems required through Highway 401, with shorter protection systems likely required parallel to Wilson Road 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations, plus subexcavation costs 	<ul style="list-style-type: none"> Risk of softening/ loosening of footing subgrade; groundwater control is important
Driven steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for support of abutments Not required for support of retaining walls 	<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, potentially reducing depth of excavation, dewatering and protection system requirements compared with footing option Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration 	<ul style="list-style-type: none"> Temporary protection systems still required along Highway 401, but may be slightly shallower than those for concrete footings (i.e., to subgrade level for RSS walls rather than footing level) Due to the shallow depth to “100-blow” material, pre-augering will likely be required 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving



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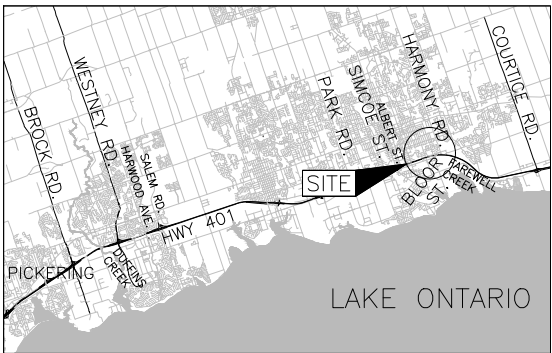
Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none">• Feasible but not recommended for support of abutments	<ul style="list-style-type: none">• Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, dewatering and protection system requirements compared with footing option• 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 non-cohesive soils, with potential for loss of ground or base disturbance• 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• Precludes use of integral abutments	<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 at base of caissons



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

CONT No. _____
WO No. 10-20011

WILSON ROAD OVERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS



KEY PLAN
SCALE
4 0 4 8 km

LEGEND

Borehole - 1973 Investigation
(Geocres No. 30M15-32)

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
32-1	89.8	4860901.0	358100.8
32-2	89.9	4860857.1	358115.5
32-3	89.9	4860855.1	358101.8
32-4	89.8	4860896.5	358088.6

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.

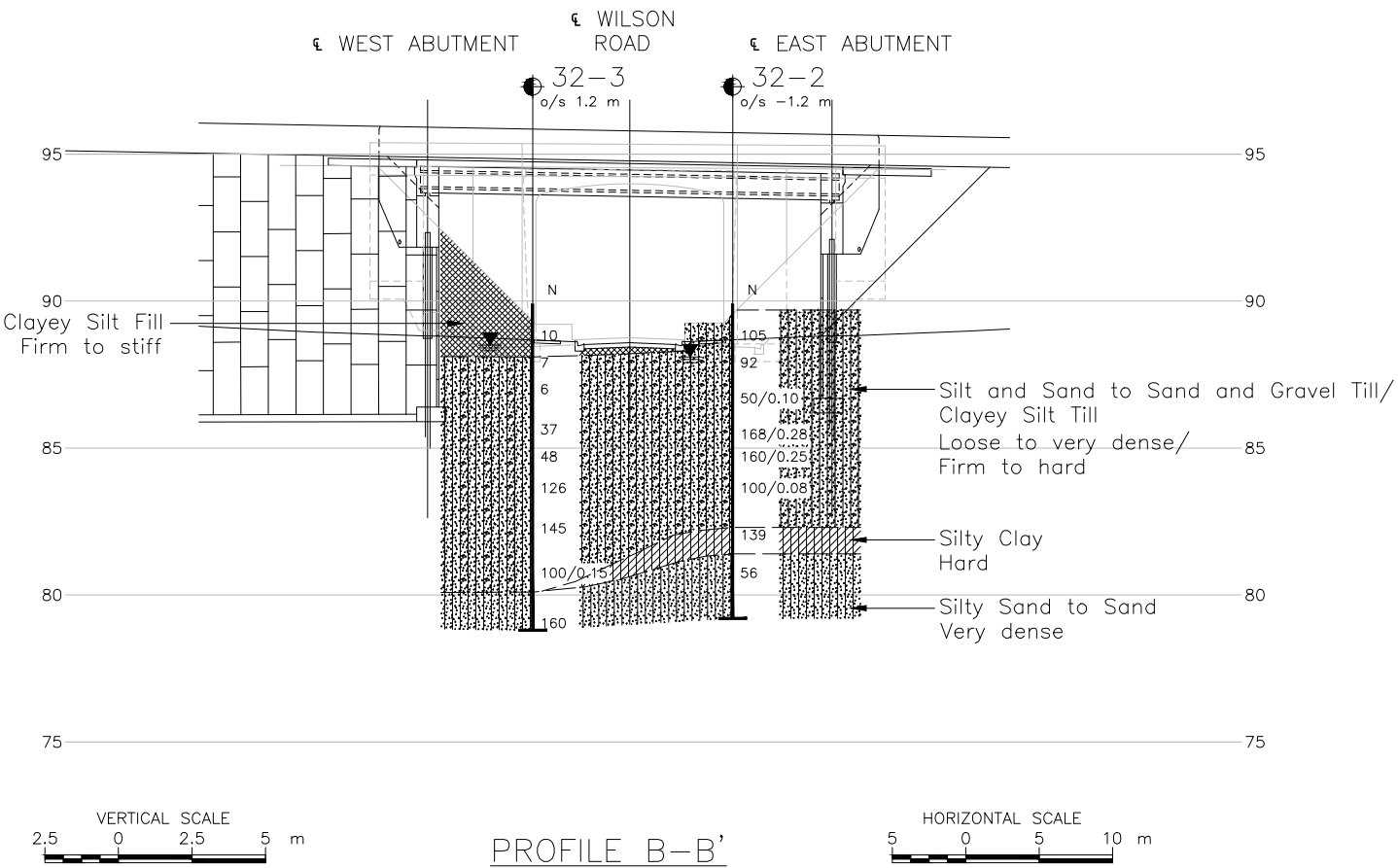
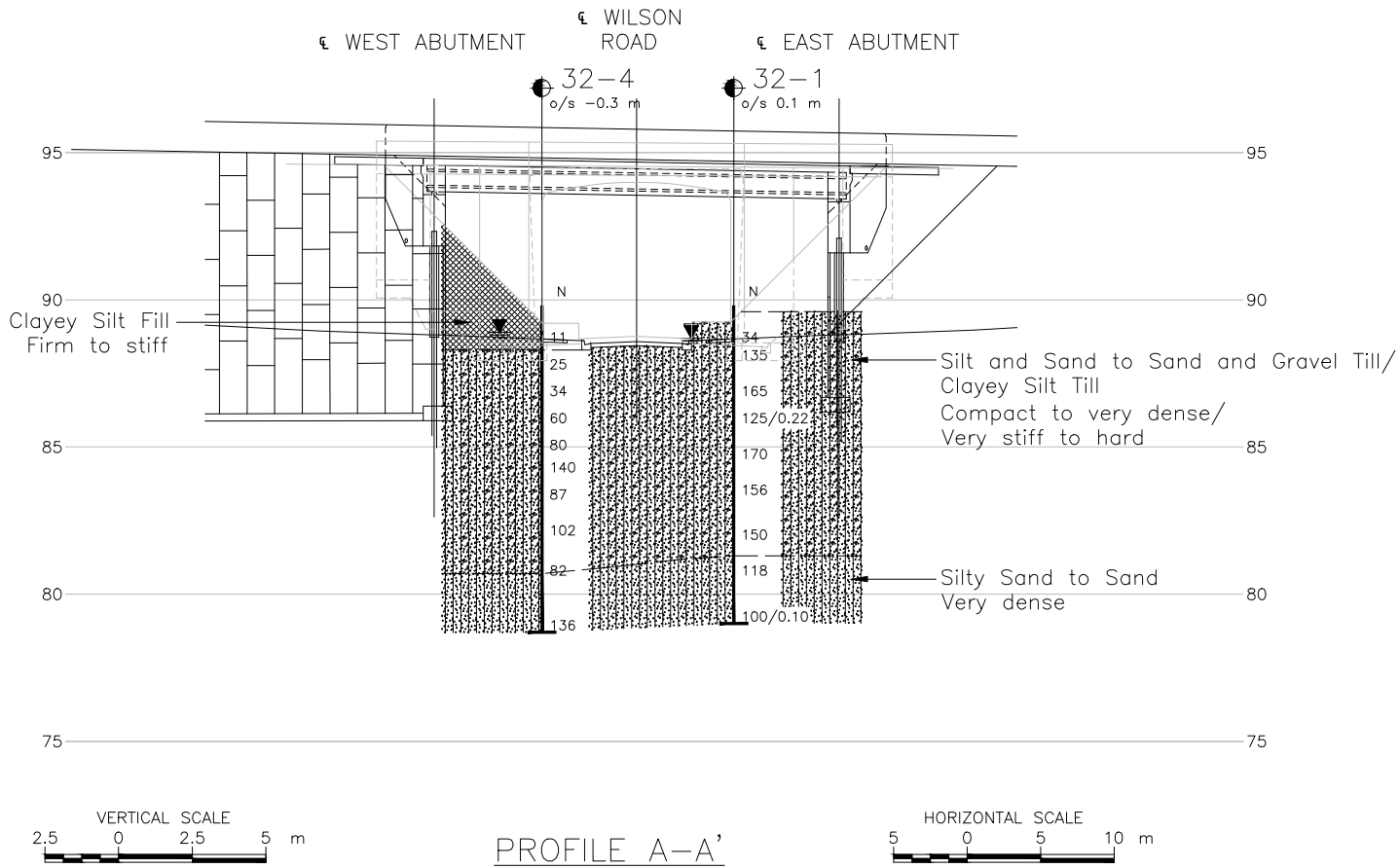
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by AECOM, drawing file no. WilsonRd_Overpass_GA.dgn



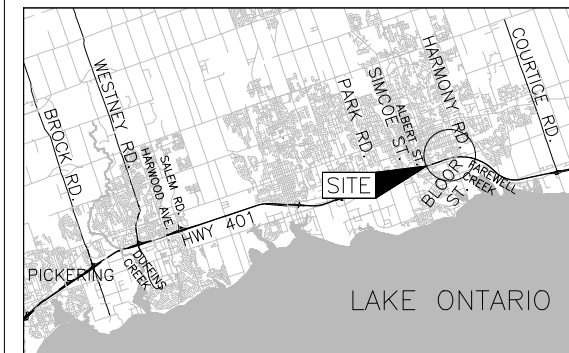
NO.	DATE	BY	REVISION
Geocres No. 30M15-308			
HWY. 401		PROJECT NO. 11-1184-0143	DIST. CENTRAL
SUBM'D. AVD	CHKD. LCC	DATE: Dec. 2016	SITE: 22-180
DRAWN: DD	CHKD. AVD	APPD. LCC	DWG. 1



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

CONT No.
WO No. 10-20011

WILSON ROAD OVERPASS
HIGHWAY 401 IMPROVEMENTS
SOIL STRATA



LEGEND

- Borehole - 1973 Investigation (Geocres No. 30M15-32)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured July 1973

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
32-1	89.8	4860901.0	358100.8
32-2	89.9	4860857.1	358115.5
32-3	89.9	4860855.1	358101.8
32-4	89.8	4860896.5	358088.6

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

Vertical alignment provided in digital format by AECOM, drawing file no. WilsonRd_Overpass_GA.dgn



NO.	DATE	BY	REVISION
Geocres No. 30M15-308			
HWY. 401		PROJECT NO. 11-1184-0143	DIST. CENTRAL
SUBM'D. AVD	CHKD. LCC	DATE: Dec. 2016	SITE: 22-180
DRAWN: DD	CHKD. AVD	APPD. LCC	DWG. 2



APPENDIX A

Borehole Records and Geotechnical Laboratory Test Results – GEOCRES No. 30M15-32

LOCATION Co-ords. 15,947,106 N; 1,174,817 E.

ORIGINATED BY CSP

BORING DATE July 16, 1973

COMPILED BY CSP

BOREHOLE TYPE Auger

CHECKED BY Q/A

15 $\frac{20}{10}$ 5 % STRAIN AT FAILURE

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2

JOB 73-11052

LOCATION Co-ords. 15,946,962 N; 1,174,866 E.

ORIGINATED BY CSP

W.P. 44-71-10

BORING DATE July 18, 1973

COMPILED BY CSP

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — w_L			BULK DENSITY	REMARKS
(4) ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT				PLASTIC LIMIT — w_p				
							SHEAR STRENGTH P.S.F.				WATER CONTENT — w				
295.1	Ground Level														
0.0	Topsoil														
0.7	Het. mix. of silt, sand & gravel, trace of clay (Glacial Till) (with numerous zones of clayey silt, sand and gravel) Grey Very Dense		1	SS	105	290									88.1m
			2	SS	92										289.1
			3	SS	50/7	' bouncing									9' 35' 42" 14
			4	SS	168/11"	280									26 32 37 5
			5	SS	160/10"										
			6	SS	100/7"										
270.1						270									
25.0	Silty clay, traces of sand & gravel.		7	SS	139										
267.1	Sand with some silt and traces of gravel.		8	SS	56										2 84 (14)
260.1	Grey Very Dense														
35.0	End of Borehole					260									

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 73-11052

LOCATION Co-ords. 15,946,955 N; 1,174,820 E.

ORIGINATED BY CSP

W.P. 44-71-10

BORING DATE July 18, 1973

COMPILED BY CSP

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — w_L			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT				PLASTIC LIMIT — w_p				
							SHEAR STRENGTH P.S.F.				WATER CONTENT — w				
							<div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div><div>+ FIELD VANE</div><div>x LAB VANE</div></div>				<div><div>w_p — w — w_L</div><div>10 20 30</div></div>				
294.8	Ground Level														
0.0	Fill Material		1	SS	10	290									88.5m
288.8	Clayey silt, some sand & gravel, traces of organics, Brown-Grey Firm to Stiff		2	SS	7										
6.0	(Glacial Till)		3	SS	6	280									9.62 (29)
	Het. mix. of sand, silt & gravel	4	SS	37											14.39 40.7
	(with occ. zones of clayey silt, sand & gravel)	5	SS	48											
	Grey	6	SS	126											9.60 30.1
			7	SS	145	270									
262.8	Loose to Very Dense		8	SS	100/5"	260									
32.0	Sand with some silt and traces of gravel		9	SS	160										2.65 25.8
258.3	Grey Very Dense					250									
36.5	End of Borehole														

(m)

89.9

0.0

88.0

1.9

80.1

9.8

78.7

11.2

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 4

JOB 73-11052

LOCATION Co-ords. 15,947,091 N; 1,174,777 E.

ORIGINATED BY CSP

W.P. 44-71-10

BORING DATE July 19, 1973

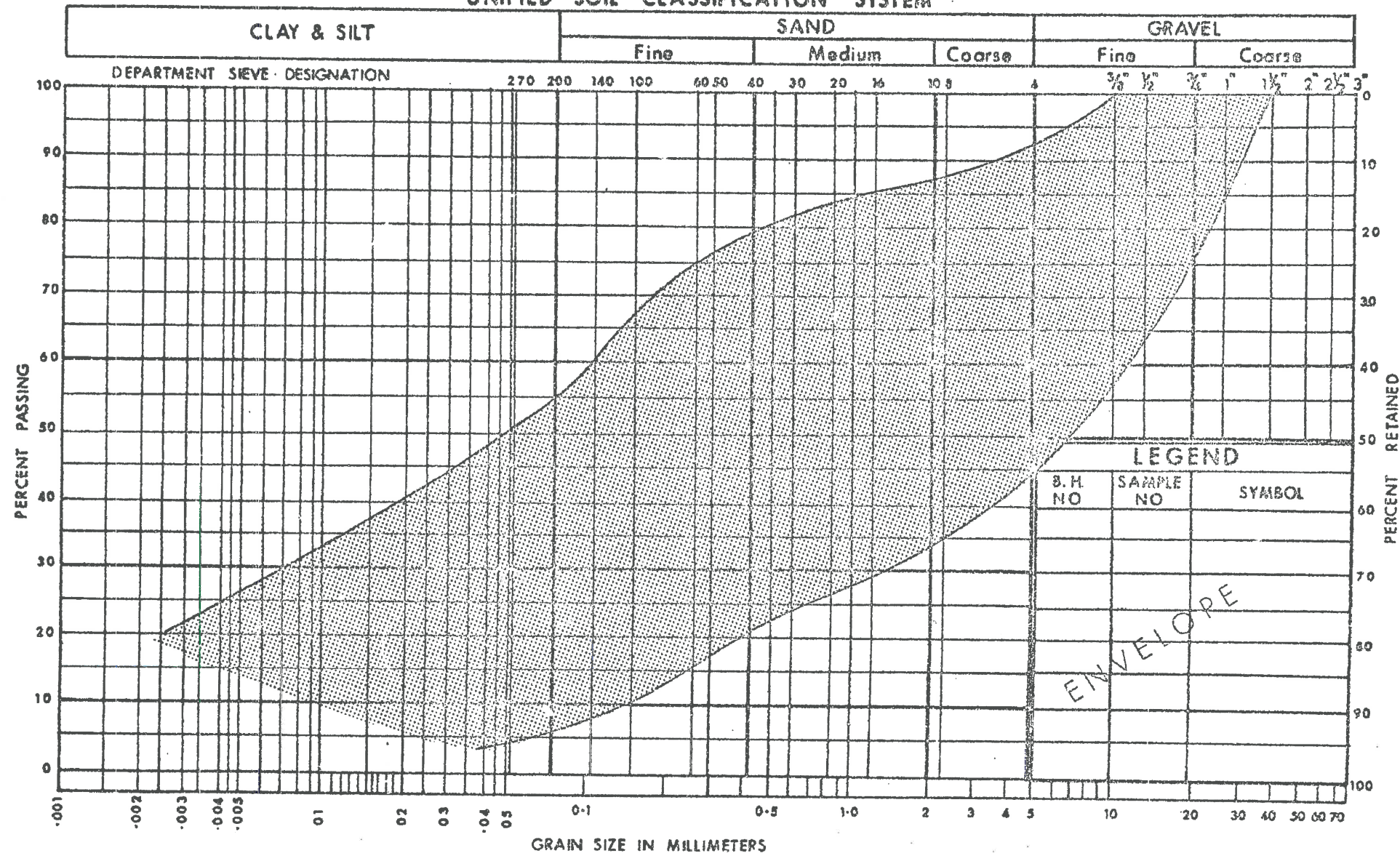
COMPILED BY CSP

DATUM Geodetic

BOREHOLE TYPE Auger

 CHECKED BY *[Signature]*

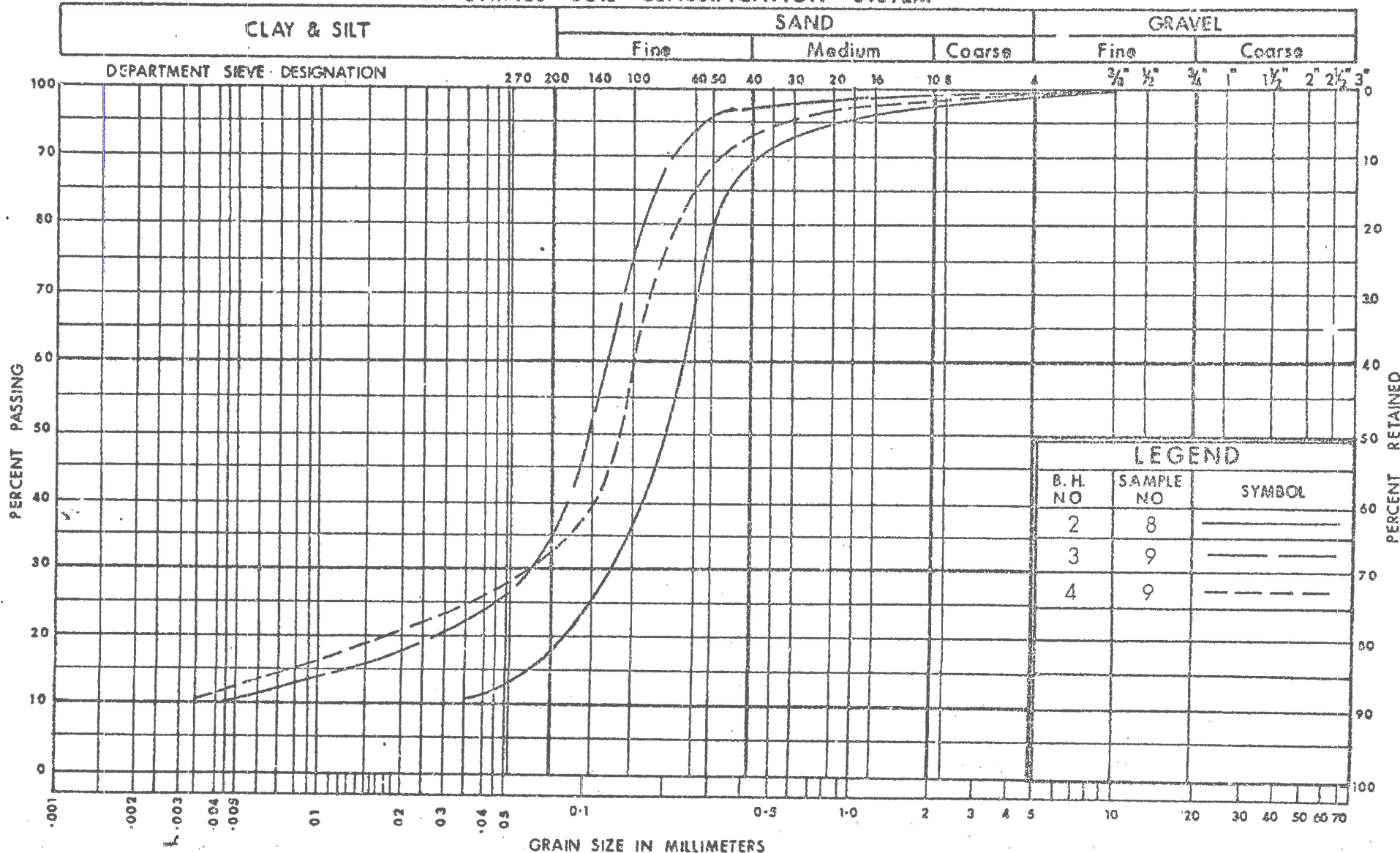
SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — w_L			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT				PLASTIC LIMIT — w_p				
							SHEAR STRENGTH P.S.F.				WATER CONTENT — w				
294.6	Ground Level														
0.0	Fill Material														
289.6	Clayey silt, some sand & gravel. Brown Firm to Stiff		1	SS	11	290									291.8 88.1m
5.0	(Glacial Till)		2	SS	25										9 51 29 11
	Het. mix. of sand, gravel and silt		3	SS	34										
			4	SS	60										
	(with occ. zones of clayey silt, sand and gravel)		5	SS	80	280									57 39 (4)
			6	SS	110										
			7	SS	87										46 41 (13)
	Grey		8	SS	102	270									
264.6	Compact to Very Dense														
30.0	Sand with some silt & traces of gravel.		9	SS	82										3 66 24 7
258.1	Grey Very Dense		10	SS	136	260									
36.5	End of Borehole														
						250									



GRAIN SIZE DISTRIBUTION
GLACIAL TILL
HET. MIXTURE OF SILT, SAND & GRAVEL, TRACES OF CLAY

FIG. 1

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
SAND
SOME SILT, TRACES OF GRAVEL

W.P. No. 44-71-10

JOB No. 73-11052

FIG. 2

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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