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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Simcoe Street N/S-W Ramp Bridge Over Oshawa Creek Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

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REPORT



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**PRELIMINARY FOUNDATION REPORT
SIMCOE STREET N/S-W RAMP BRIDGE, W.O. 10-20011**

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
SIMCOE STREET N/S-W RAMP BRIDGE OVER OSHAWA CREEK
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 Simcoe Street N/S-W Ramp bridge over Oshawa Creek. Data from a previous investigation at the Highway 401 bridge over Oshawa Creek, immediately south of this proposed ramp structure, have been used in the preparation of this report; this previous investigation is referenced as follows:

- **MTO GEOCREs No. 30M15-033:** *Foundation Investigation Report for Widening of Oshawa Creek Bridge, Highway 401, Oshawa, Site 22-175. W.P. 44-71-08 – W.O 73-11022*, by MTO Foundations Section, dated June 21, 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 site of the proposed Simcoe Street N/S-W Ramp bridge over Oshawa Creek is located immediately north of the existing Highway 401-Oshawa Creek bridge, approximately 1.5 km east of the Stevenson Road interchange, between Cubert Street and Simcoe Street in the City of Oshawa, in the Regional Municipality of Durham.

The existing creek channel base at the proposed ramp bridge site is at approximately Elevation 90 m. The natural ground surface rises to about Elevation 95 m west of the creek, and to about Elevation 91 m to 92 m east of the creek. A public trail runs along the west side of the creek at the site, and the area is vegetated with grasses and trees.

3.0 INVESTIGATION PROCEDURES

Six boreholes were advanced along the north side of the Highway 401-Oshawa Creek bridge, immediately south of the proposed N/S-W Ramp structure, as part of a previous investigation by the Foundations Section of the Ministry of Transportation and Communications Ontario in December 1972 and May 1973. 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 12 from GEOCREs Report No. 30M15-033 is referred to throughout this report and on the drawings as Borehole 033-12.

The approximate locations of Boreholes 033-9, 033-10, 033-12, 033-14, 033-15 and 033-16 are shown on Drawing 1. The boreholes were advanced through the overburden using both wash-boring and augering techniques as necessary, with equipment supplied and operated by Canadian Longyear Limited of Rexdale, Ontario. Soil samples were obtained at regular intervals using split-spoon samplers driven by a manual hammer, and the resulting Standard Penetration Test (SPT) 'N' values were recorded, as shown on the borehole records



contained in Appendix A. Dynamic cone penetration tests were completed adjacent to Boreholes 033-9, 033-14 and 033-15, and these results are also shown on the borehole records in Appendix A.

The groundwater conditions in the open boreholes were observed during and immediately after the 1972/1973 drilling operations, and these measurements are noted on the borehole records contained 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* (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.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced on the north side of Highway 401 as part of the 1972/1973 investigation and the results of in situ and laboratory testing are presented on the borehole records and laboratory test figures contained in Appendix A. An interpreted stratigraphic profile along the ramp structure is shown on Drawing 1.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 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 interpreted stratigraphic profiles shown on Drawing 1 represent simplifications of the subsurface conditions at the site. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the adjacent Oshawa Creek bridge site consist of surficial layers of topsoil and fill associated with the existing Highway 401 embankments, underlain by a non-cohesive deposit of interlayered sand, silt and gravel. This interlayered deposit is in turn underlain by a silty sand to sandy silt till deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Fill material was encountered immediately below the then-existing ground surface in Boreholes 033-9, 033-10, 033-14 and 033-16, during the 1972/1973 investigation near the north side of Highway 401. The thickness of the fill ranged from approximately 0.6 m to 7.4 m, with the base of the fill encountered between Elevation 89.5 m and 92.6 m. This thickness of fill may not extend throughout the footprint of the proposed ramp structure site.

The fill material encountered in the boreholes was generally non-cohesive, varying in composition from sand and gravel, to gravelly sand, to sand containing some silt, to silt and sand. The results of grain size distribution tests on selected samples of the fill are annotated on the borehole records in Appendix A, and included within the



envelopes of grain size distribution shown on Figures 1 to 3 in Appendix A. The natural water content measured on selected samples of the fill varied between approximately 5 and 20 per cent.

The Standard Penetration Test (SPT) 'N' values measured within the fill material in these boreholes range from 4 blows to 41 blows per 0.3 m penetration, indicating a variable, loose to dense relative density.

4.2.2 Interlayered Sandy Silt, Sand and Silt, and Sand and Gravel Deposit

A non-cohesive deposit with variable proportions of silt, sand and gravel was encountered below the fill layer in Boreholes 033-9, 033-10 and 033-16, and immediately below the then-existing ground surface in Borehole 033-15. The base of this deposit was encountered between approximately Elevations 88.5 m and 90.1 m in the boreholes; Borehole 033-16 was terminated within this deposit at about Elevation 87.7 m. The deposit thickness ranges from about 2.1 m to greater than 4.9 m.

The deposit consists of layers of non-cohesive soil that vary in composition from sandy silt, to sand and silt, to sand and gravel; the coarser (sand and gravel) layers were encountered near the eastern portion of the site, in Boreholes 033-15 and 033-16. Trace quantities of organic matter were observed in samples recovered within this deposit, as noted on the borehole records. The results of grain size distribution tests completed on samples of this deposit are annotated on the borehole records, and envelopes of grain size distributions are shown on Figures 1 to 3 in Appendix A. The measured water contents of samples of this deposit range from about 11 to 38 per cent.

The SPT 'N'-values measured within this deposit range from about 8 to 18 blows per 0.3 m of penetration, except in the sand and gravel layer encountered in Borehole 033-16 where SPT 'N' values of 57 to greater than 100 blows per 0.3 m of penetration were measured. Based on these results, the deposit generally has a loose to compact relative density, while the sand and gravel layer has a very dense relative density.

4.2.3 Silty Sand to Sandy Silt Till

A deposit of silty sand to sandy silt till was encountered underlying the interlayered deposit (Boreholes 033-9, 033-10 and 033-15), directly below the ground surface (Borehole 033-12), or below a thin layer of fill (Borehole 033-14); Borehole 033-16 was terminated above the till deposit. Where encountered, all boreholes terminated within this deposit, penetrating it for a thickness of 2.2 m to 4.9 m.

The till deposit varies in composition from silty sand to sandy silt, containing trace to some gravel. The results of grain size distribution tests completed on selected samples of the till deposit are annotated on the borehole records, and an envelope of the grain size distributions is shown on Figure 4 in Appendix A. Natural water content measurements on selected samples of this deposit range from about 8 to 13 per cent.

The SPT 'N' values measure within the silty sand to sandy silt to silt with sand till are generally greater than 100 blows per 0.3 m of penetration, with one SPT 'N' value of 72 blows per 0.3 m of penetration, indicating a very dense relative density.



4.3 Groundwater Conditions

During the 1972/1973 investigation, water levels were observed in the open boreholes at the completion of drilling and are summarized as follows:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level below 1973 Ground Surface (m)	Groundwater Elevation (m)	Date
033-9	94.8	5.2	89.6	May 9, 1973
033-10	99.0	8.0	91.0	May 3, 1973
033-12	89.8	0.0	89.8	May 10, 1973
033-14	90.1	0.3	89.8	May 10, 1973
033-15	93.1	3.1	90.0	May 8, 1973
033-16	97.8	6.1	91.6	May 7, 1973

The groundwater level was encountered at or immediately below the then-existing ground surface in Boreholes 033-12 and 033-14, which are located immediately adjacent to the creek channel. On May 8, 1973, the water level in the creek was at approximately Elevation 89.8 m at the north end of the bridge and Elevation 89.3 m at the south end (i.e., at or near the ground surface in Boreholes 033-12 and 033-14).

Although not encountered in the boreholes reported in this Preliminary Foundation Investigation Report for the N/S-W Ramp structure site, artesian water pressures were encountered in Boreholes 033-1 and 033-5 near the south side of the Highway 401-Oshawa Creek bridge site. These artesian conditions were encountered within the till deposit at about Elevation 83.2 m to 83.5 m, and the water level was noted to rise above the then-existing ground surface to approximately Elevation 91.4 m.

It should be noted that 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 fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.



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5.0 CLOSURE

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

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**PRELIMINARY FOUNDATION REPORT
SIMCOE STREET N/S-W RAMP BRIDGE, W.O. 10-20011**

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
SIMCOE STREET N/S-W RAMP BRIDGE OVER OSHAWA CREEK
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6.0 DISCUSSION AND PRELIMINARY GEOTECHNICAL ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed construction of a new bridge carrying the Simcoe Street N/S-W Ramp over Oshawa Creek, as part of the Highway 401 improvements in this area. These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 1972/1973 subsurface investigation at the Highway 401-Oshawa Creek bridge, immediately to the south of the proposed ramp structure 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 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, a new bridge is proposed to carry the realigned Simcoe Street N/S-W Ramp over Oshawa Creek. For the purposes of this report, construction north has been assigned such that the abutment on the northeast side of the creek is referred to as the north abutment, while that on the southwest side of the creek is referred to as the south abutment. Based on the preliminary General Arrangement drawing provided by AECOM, the proposed ramp bridge is to consist of a single-span structure with a total span length of approximately 35 m, and an approximate width of 14 m. The ramp grade will rise from the northeast to the southwest, from approximately Elevation 98.3 m at the north abutment, to approximately Elevation 99.3 m at the south abutment. This will require embankments up to approximately 7.5 m high at the north approach, and up to approximately 4 m high at the south approach. Short concrete retaining walls will be required adjacent to the wingwalls, parallel to the ramp, to retain the embankment fill in the vicinity of the creek valley.

Both shallow and deep foundation options have been considered for support of the N/S-W Ramp bridge. 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 within the compact interlayered deposit or on very dense till:**
Shallow footings are feasible for support of the abutments, due to the relatively shallow depth to very dense, “100-blow” soils. This option would require excavation to a minimum depth of 1.2 m below the existing grade at the ramp site, to meet frost protection requirements. The near-surface deposits as encountered at the adjacent Highway 401-Oshawa Creek bridge site include a zone of loose to compact sandy silt, sand and silt, and sand and gravel, and additional subexcavation may be required to remove loose soils from the



footprint of the footings, prior to placement of granular fill or tremie concrete; site-specific investigation will be required within the footprint of the ramp bridge during detailed design to confirm any subexcavation requirements and geotechnical resistances. It is anticipated that the south abutment footing will be able to be maintained above the groundwater level at the site (although subexcavation, if required, could extend below the water table); at the north abutment, however, it is expected that the footing and any subexcavation would extend below the water table, with dewatering/cofferdams required to facilitate a stable subgrade for construction.

- **Driven steel H-piles or pipe piles founded within the very dense till:** Driven steel H-piles or pipe piles are feasible for support of the abutments, and would permit design of conventional, semi-integral (pipe piles) or integral abutments (H-piles). This option may permit the pile cap to be maintained higher than spread footings, thus minimizing excavation, protection system and groundwater control requirements as compared to a spread footing option. However, due to the relatively shallow depth to very dense, “100-blow” soils, pre-augering through the very dense till deposits would likely be required to achieve minimum penetration depth for the required pile length and capacity, and to ensure that the piles remain aligned and are not damaged. Heavier pile sections and/or the use of pile driving shoes would be recommended to protect the pile and pile tip from damage during driving.
- **Caissons founded within the very dense till:** Caissons are considered feasible for the support of the new structure abutments, and would permit design of conventional abutments; however, this option would preclude integral abutment design. Similar to a driven pile option, this option may permit the pile cap to be maintained higher than for spread footings, thus minimizing excavation, protection system and groundwater control requirements. However, artesian groundwater pressures were encountered within the till deposit at depth; if caissons are adopted, these will extend through water-bearing non-cohesive deposits, and temporary liners and drilling mud would be required during caisson construction to balance the artesian pressures, and control potential ground losses and disturbance at the caisson base. For this new structure site, this option is expected to be more expensive than either shallow foundations or pre-augured/driven pile foundations.

Based on the above considerations, shallow foundations are preferred from a geotechnical/foundations perspective, due to the presence of “100-blow” soil at relatively shallow depth, and the anticipated limited dewatering requirements at the south abutment location. However, further investigation will be required during detailed design to assess the relative density of the near-surface soils and confirm any requirements for subexcavation to achieve the required geotechnical resistances.

6.3 Shallow Foundations

6.3.1 Founding Elevation

For support of the new ramp bridge abutments, spread footings should be founded on compact or better native soils; concrete retaining walls may also be founded on compacted granular pads that are stepped up into the approach embankments. 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.



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Some of the near-surface soils encountered in the 1972/1973 boreholes at the adjacent bridge site had a loose relative density, and such soils should be subexcavated and replaced with compacted granular fill or mass concrete prior to construction of the abutment footings, or alternatively the abutment footings may be founded at a lower grade. The table below summarizes the recommended footing founding levels for preliminary design:

Foundation Element	Maximum (Highest) Founding Elevation (m)	Founding Soil
North Abutment	90.0	Compact sand and gravel/ Very dense till
South Abutment	92.0*	Compact, interlayered sand/silt over very dense till

* At this preliminary stage, it is recommended that allowance be made for approximately 1 m of subexcavation at the south abutment footing; however, further investigation will be required at the detailed design stage to assess and confirm subexcavation requirements, if any.

The groundwater level at the adjacent Highway 401-Oshawa Creek bridge site was between approximately Elevation 89.6 m to 91.6 m during the 1972/1973 investigation, although this may not represent the stabilized or current groundwater level at the new N/S-W Ramp bridge site. The maximum (high) water level at the creek is understood to be at approximately Elevation 95.6 m, based on the General Arrangement drawing provided by AECOM. Groundwater control/cofferdams will be required during construction for the north abutment, and may be required during subexcavation at the south abutment, to maintain stable subgrade conditions for foundation construction.

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 and Reaction

The following factored geotechnical axial resistance at ULS and geotechnical resistance at 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	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Abutments	5	500	350
Retaining wall footing perched in approach embankment on minimum 1 m thick Granular 'A' pad	3	500	350

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

Steel H-piles or pipe piles should be founded within the very dense (“100-blow”) till deposit in either a conventional, integral (for H-piles) or semi-integral (for pipe piles) abutment configuration. Pre-augering into the shallow “100-blow” soil will most likely be required, to avoid misalignment during initial driving of the piles, minimize damage due to heavy driving and achieve sufficient length embedment for the necessary geotechnical resistances.

Due to the relatively shallow depth to “100-blow” soils, the preliminary foundation design recommendations have been developed assuming that piles will have to extend approximately 5 m into the very dense soil to achieve the reasonable resistances for bridge foundation design. The table below provides pile tip levels for preliminary design; these tip levels extend below the boreholes at the adjacent Highway 401-Oshawa Creek bridge, and further borehole investigation will be required as part of detailed design.

Foundation Element	Elevation of “100-blow” Till (m)	Design Pile Tip Elevation (m)
North Abutment	89	84
South Abutment	89	84

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. 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 Geotechnical Axial 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 1,000 kN. The axial reaction at SLS for 25 mm of settlement may also be taken as 1,000 kN; where lesser deflections is desired/required, the axial reaction at SLS for 15 mm deflection may be taken as 800 kN. The same axial resistances may be used in the design of close-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 detailed design in consideration of any 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 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 very dense silty sand to sandy silt till may be considered for support of the new abutments. If adopted, the following caisson founding elevations may be used for preliminary design purposes, assuming approximately 5 m of penetration into “100-blow” soil to achieve an adequate resistance for bridge foundation design. It is noted that these elevations extend below the base of the 1972/1973 boreholes at the adjacent Oshawa Creek bridge site, and these values should therefore be taken as preliminary; further investigation will be required at the new structure abutments during detailed design:

Foundation Element	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)
North Abutment	88	82
South Abutment	89	84

If caisson foundations are adopted, the presence of water-bearing non-cohesive soils and artesian groundwater pressures must be taken into account. A temporary liner and drilling slurry would be required to support the overburden soils during construction and to balance the artesian groundwater pressures to minimize disturbance to the base. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance/Reaction

If caissons are adopted, based on the preliminary founding elevations given above, the factored geotechnical resistance at ULS may be taken as 2,400 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.

If this foundation option is adopted for the widening of the abutments and/or piers, 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 Retained Soil System (RSS) Walls

Retained soil system (RSS) walls are a suitable and feasible alternative to conventional concrete retaining walls supported on shallow foundations, for the retaining walls adjacent to the wingwalls at this new structure site. The base of the RSS walls would likely be below the design high water level in Oshawa Creek, and this must be considered in the selection of the wall type for this site. If adopted, appropriate materials and procedures will be required to minimize the potential for loss of granular backfill materials during flooding, and these requirements will have to be incorporated into the contract documents in detailed design.

6.6.1 Founding Elevations

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 1 horizontal to 1 vertical (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.

Assuming that the RSS wall (estimated to be up to approximately 5 m high) 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. 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 any additional subsurface information at that time.

6.6.3 Global Stability of RSS Walls

Preliminary slope stability analyses have been performed for conceptual RSS walls adjacent to the east and west abutments using the commercially available program *Slide 6.0*, produced by Rocscience Inc., to check that a minimum factor of safety of 1.5 is achieved for the proposed maximum retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this site, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses, 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
Embankment fill	21	32°
Compact silt to sand	21	32°
Dense to very dense sand and gravel	21	34°
Very dense silty sand to sandy silt till	21	34°

The results of the static global stability analyses indicate that a minimum factor of safety of 1.5 is achieved for RSS walls up to approximately 5 m in retained soil height, assuming level ground in front of and behind the wall, as shown on Figure 1. This preliminary assessment of the global stability of the retaining 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 above or below the wall) is refined and further borehole information is obtained within the footprint of the walls.

It should be noted that the internal stability of RSS walls is to be assessed by the proprietary product designer.

6.6.4 Settlement

At this preliminary stage, it is estimated that for the new ramp embankments, the settlement of the underlying soils will be up to approximately 50 mm under the north approach, and up to approximately 25 mm under the south



approach, depending on the thickness of “loose” non-cohesive soils within the embankment footprint. This settlement is expected to occur immediately, concurrent with the embankment and retaining wall construction. Based on these estimates, it is anticipated that the settlement performance for RSS walls and facing panels will be acceptable.

6.7 Approach Embankments

6.7.1 Subgrade Preparation and Embankment Construction

The ramp grade will rise from the northeast to the southwest, from approximately Elevation 98.3 m at the north abutment, to approximately Elevation 99.3 m at the south abutment. This will require embankments up to approximately 6.5 m to 7.5 m high (depending on the existing ground surface elevation) at the north approach, and up to approximately 4 m high at the south approach.

For the new approach embankment construction, it is recommended that any topsoil within the embankment footprint be stripped. The new embankment side slopes should be formed at a maximum gradient of 2H:1V. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be completed as soon as practicable after the construction. The erosion protection should be in accordance with OPSS 804 (*Seed and Cover*).

6.7.2 Global Slope Stability

Preliminary static slope stability analyses have been completed for the new ramp approach embankments, 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 (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 of a maximum 7.5 m high approach embankment in long-term drained (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/m³)	Effective Friction Angle (°)
Fill	21	32
Compact silt to sand	21	32
Dense to very dense sand and gravel	21	34
Very dense glacial till	21	34

The results of the static global stability analysis indicate that a minimum factor of safety of greater than 1.3 is achieved for an approximately 7.5 m high embankment, oriented no steeper than 2H:1V, as shown on Figure 2. This preliminary assessment of the slope stability of the embankments should be reviewed and confirmed during detailed design.



6.7.3 Settlement Under New Embankment Loading

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

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Fill	21	-
Loose to compact silt to sand	21	35
Dense to very dense sand and gravel	21	100
Very dense glacial till	21	150

Based on this preliminary assessment, the settlement of the foundation soils under the new ramp embankments is estimated to be up to approximately 50 mm under the maximum approach embankment height at the north abutment, and up to approximately 25 mm under the maximum approach embankment height at the south abutment. This settlement will occur immediately, concurrent with the embankment construction, based on the non-cohesive nature of the soils underlying the site.

These preliminary settlement estimates will have to be reassessed during detailed design, based on the results of additional investigation within the footprint of the new ramp bridge abutments and approaches.

6.8 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 contract documents produced as part of detail design.

6.8.1 Open-Cut Excavation and Temporary Protection Systems

If adopted at the abutments, the construction of the new strip footings would require excavation to approximately 1.5 m to 2.5 m below the ground surface, through anticipated loose to compact sand/silt/gravel, and potentially into dense to very dense sand and gravel or till deposits. The loose to compact sand/silt/gravel are classified as Type 3 soils, while the native dense to very dense soils are classified as Type 2 soils (provided they are dewatered), according to the Occupational Health and Safety Act (OHSA). Temporary excavations should be made with side slopes no steeper than 1H:1V, assuming that groundwater control is implemented to maintain the water level below the base of temporary excavations.

Temporary protection systems are likely to be required in front and behind the abutment foundation excavations. These temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the protection systems should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any adjacent utilities can tolerate this magnitude of deformation.



The selection and design of the protection system will be the responsibility of the Contractor. However, for preliminary planning purposes, it is noted that the very dense ("100-blow") till soils that are present near the footing founding levels will likely pose some challenges for driven steel sheetpiles.

6.8.2 Groundwater Control

Assuming that shallow foundations are adopted for support of the abutments widening, excavation for their construction will extend to approximately 90 m to 92 m, with deeper subexcavation required below the footing if loose soils are encountered. The groundwater level was measured to be between Elevation 89.6 m and 91.6 m, during the 1972/1973 investigation, although this may not represent the stabilized or current groundwater level at the site. It is anticipated that the footing excavations will extend below the groundwater level at the site, particularly for the north abutment area.

At this preliminary stage, it is recommended that dewatering be assumed to be required, potentially in conjunction with cofferdams to create a groundwater cut-off through the more permeable upper deposits. An active dewatering system (such as wellpoints or eductors, given the fine-grained nature the silt and till deposits) will be needed to lower the groundwater level to approximately 1 m below the proposed founding elevation and ensure a stable base. Further investigation and monitoring of the groundwater level will be required during detailed design to confirm the current groundwater levels and determine the extent of dewatering that would be required for construction.

The water bearing soils at the proposed founding elevations are generally fine to medium grained in nature (silty to sandy) and therefore will have a lower to moderate permeability. At this preliminary stage, an accurate prediction of the groundwater pumping rates cannot be made, but it is considered possible for pumping volumes to exceed 50 m³/day during initial drawdown stages and/or if both abutment excavation areas are being dewatered at one time.

It is anticipated that the zone of influence for the dewatering operation 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 compact to very dense silt to sand deposits at the site. However, the potential for settlement impacts on any adjacent utilities (if present) should be re-assessed once the groundwater levels are confirmed.

6.8.3 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 construction of foundations and protection systems.

6.8.4 Vibration Monitoring During Pile or Caisson 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 applicable. Vibration monitoring is recommended if caissons are adopted and for temporary protection system installation, to demonstrate/confirm that vibration levels do not exceed the threshold levels.



6.9 Recommendations for Future Work in Detail Design

The existing preliminary design recommendations are based on boreholes advanced as part of previous investigations for the Highway 401-Simcoe Street underpass, to the south of the proposed new bridge; no boreholes are available within the proposed new ramp bridge footprint. During the detail design phase, geotechnical/foundation investigation will be required within the footprint of the proposed abutments and approach embankments for the new Simcoe Street N/S-W Ramp bridge to confirm or assess the following:

- The thickness and relative density of the near-surface deposits, to confirm any subexcavation requirements and geotechnical resistances for shallow foundations;
- The elevation of the “100-blow” soil in the abutment and protection system areas, to confirm tip elevations for deep foundations (if adopted) and to support the contractor’s selection and design of protection systems;
- The subsurface conditions within the footprint of the approach embankments and retaining walls, to confirm subgrade preparation/subexcavation requirements if any, settlement and global stability.
- The current groundwater levels at the site, for more detailed assessment of the groundwater control requirements and measures during construction.



PRELIMINARY FOUNDATION REPORT SIMCOE STREET N/S-W RAMP BRIDGE, W.O. 10-20011

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., EIT, and was reviewed by Ms. Nikol Kochmanova, P.Eng. Ms. Lisa Coyne, P.Eng., Designated MTO Foundations Contact and a Principal of Golder, conducted an independent technical and quality review of this report.

GOLDER ASSOCIATES LTD.

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Designated MTO Foundations Contact, Principal

MGP/NK/LCC/

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PRELIMINARY FOUNDATION REPORT SIMCOE STREET N/S-W RAMP BRIDGE, W.O. 10-20011

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ASTM International:

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

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 804	Construction Specification for Seed and Cover

Ontario Provincial Standard Drawings (OPSD)

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 SIMCOE STREET N/S-W RAMP BRIDGE, W.O. 10-20011

TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footing	<ul style="list-style-type: none"> Recommended for support of new abutments 	<ul style="list-style-type: none"> Conventional excavation and construction techniques Dense to very dense soils (with SPT “N” values greater than 30 blows per 0.3 m of penetration) present at a reasonable depth below existing ground surface, with good geotechnical resistance and settlement performance South abutment can likely be maintained above the groundwater level at the site, although subexcavation (if required) may extend below the water table 	<ul style="list-style-type: none"> Excavations anticipated to extend below the groundwater level at the north abutment, and groundwater control will be required Temporary protection systems (and/or cofferdams) required parallel to footings Precludes use of integral abutments potentially greater maintenance requirements at abutments 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations, plus the cost of protection systems and dewatering at this site 	<ul style="list-style-type: none"> Minor risk associated with excavations and dewatering in water-bearing, fine-grained soils
Steel H-piles or pipe piles founded in very dense glacial till	<ul style="list-style-type: none"> Feasible for support of new abutments, but pre-augering would be required given shallow depth to “100-blow” till 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system and groundwater control requirements 	<ul style="list-style-type: none"> Pre-augering would be required due to very dense soils present at relatively shallow depths Relatively short piles would have relatively lower geotechnical resistances in soil; higher capacities might be achieved, but additional investigation will 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, plus the cost of protection systems and dewatering 	<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

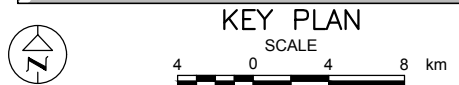


PRELIMINARY FOUNDATION REPORT SIMCOE STREET N/S-W RAMP BRIDGE, W.O. 10-20011

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
			<ul style="list-style-type: none"> • Potential for damage during driving through very dense soils • Temporary excavation support still required to facilitate pile cap construction • Design geotechnical resistance may not be achieved if piles refuse on cobble and boulder layers 		
Caissons	<ul style="list-style-type: none"> • Feasible for support of new abutments 	<ul style="list-style-type: none"> • Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, protection system and groundwater control requirements 	<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; must address artesian groundwater pressures • 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 	<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, plus the cost of protection systems and dewatering for pile caps 	<ul style="list-style-type: none"> • Risk of loss of ground during construction of caissons through water bearing layers, although the risk can be mitigated with the use of temporary liners, drilling slurry and the placement of concrete by tremie methods



**Golder
Associates**



	Borehole — 1972 to 1973 Investigation (Geocres No. 30M15-033)
	Seal
	Piezometer
	Standard Penetration Test Value
	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL in piezometer, measured 1972 to 1973
	WL upon completion of drilling

NOTES

This drawing is for subsurface information only. The proposed structure and/or 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.

Base plans provided in digital format by AECOM, drawing file no.
ACAD-01-Simcoe St._Ramp over Oshawa Creek_GA.dwg

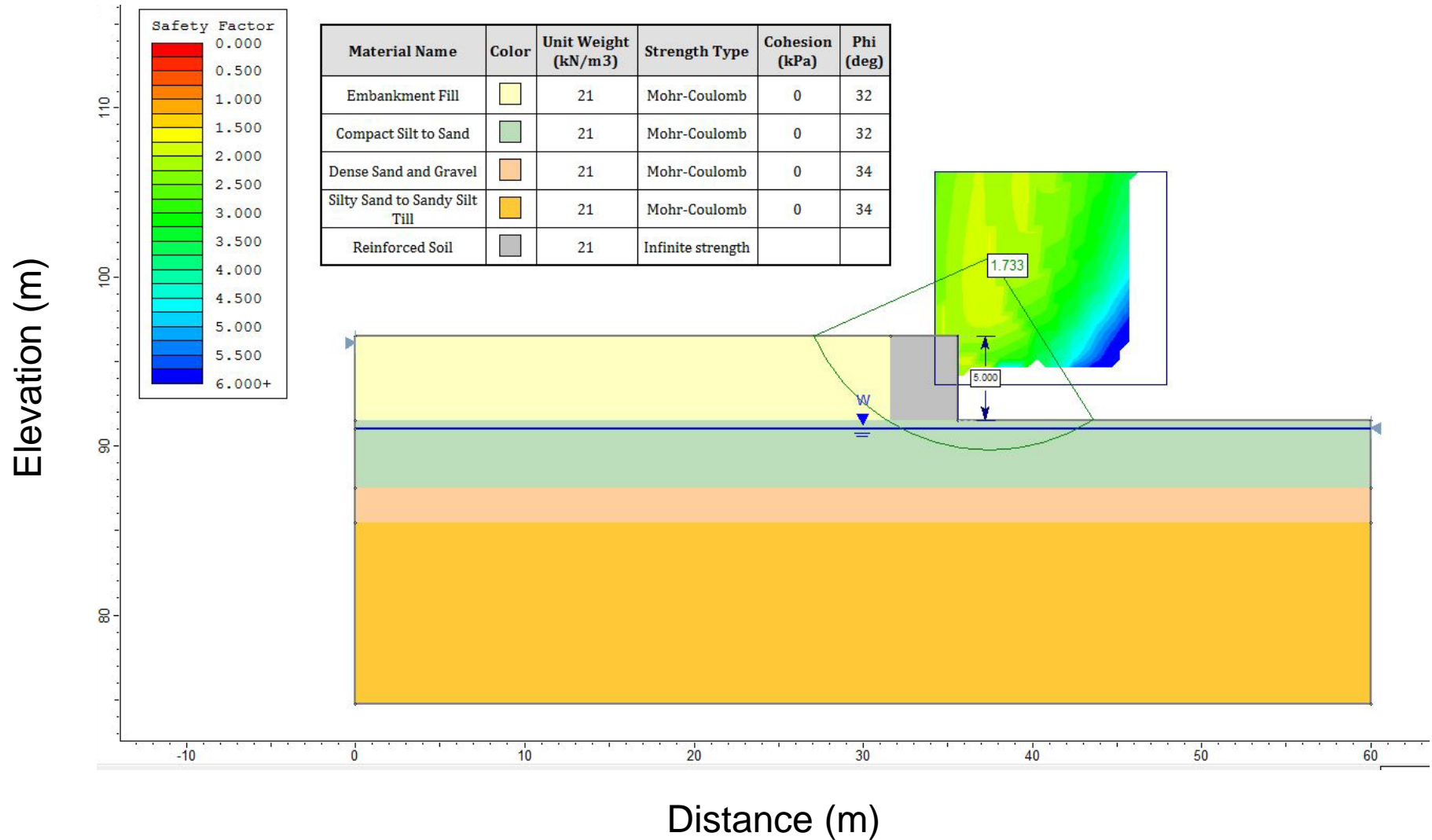


NO.	DATE	BY	REVISION
Geocres No. 30M-15-301			
HWY. 401		PROJECT NO. 11-1184-0143	DIST. CENTRAL
SUBM'D. MGP	CHKD. LCC	DATE: 4/5/2017	SITE: —
DRAWN: DD	CHKD. MGP	APPD. LCC	DWG. 1



STATIC GLOBAL STABILITY SIMCOE STREET N/S-W RAMP BRIDGE OVER OSHAWA CREEK – RETAINING WALLS

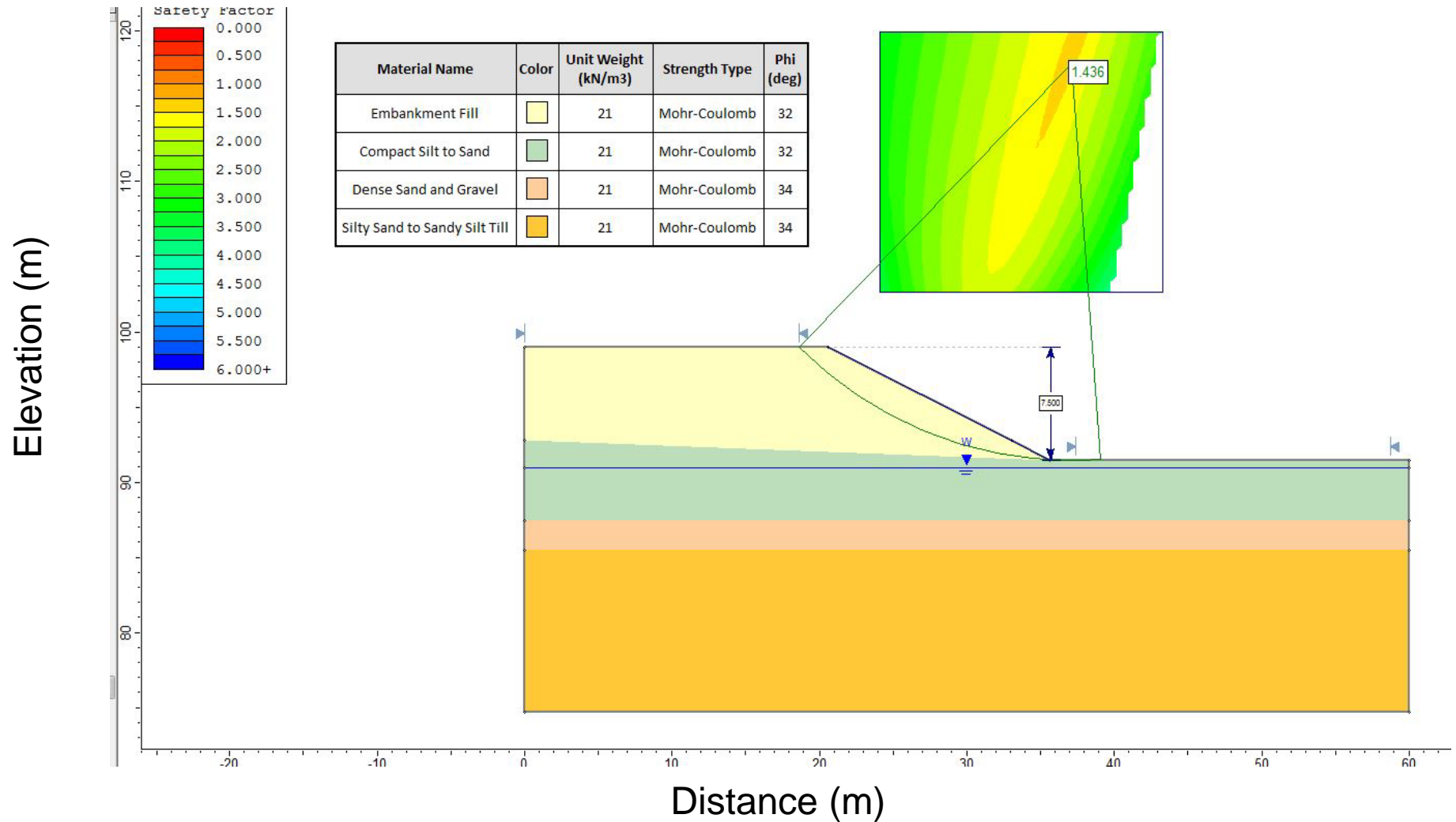
Figure 1





STATIC GLOBAL STABILITY SIMCOE STREET N/S-W RAMP BRIDGE OVER OSHAWA CREEK – APPROACH EMBANKMENTS

Figure 2





APPENDIX A

Borehole Records and Laboratory Test Results GEOCRES No. 30M15-033 – 1972 to 1973 Investigation

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-9

JOB 73-11022

LOCATION Co-ords. 15,945,232 N; 1,169,030 E.

ORIGINATED BY PK

W.P. 44-71-08

BORING DATE May 8 and 9, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY

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. PLOT	NUMBER	TYPE	BLOWS/FOOT		25	50	75	100	125	W_P	W	W_L	
94.8	Ground Level														
0.0	Sand, some silt and gravel.					310									
	Loose Brown Fill		1	SS	4										
91.8			2	SS	10	300									
3.0	Sand with silt, traces of organics.		3	SS	10										
89.6	Dark Brown Compact		4	SS	13										
5.2	Silt with sand, traces of gravel.		5	SS	17	290									
	Very Dense Grey (Till)		6	SS	100	280									
84.7			7	SS	157										
10.1	End of Borehole		8	SS	100	270									

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE N^o 033-10

JOB 73-11022 LOCATION Co-ords. 15,945,202 N; 1,169,038 E. ORIGINATED BY PK
 W.P. 44-71-08 BORING DATE May 3, 1973 COMPILED BY PK
 DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY PK

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 GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				WATER CONTENT % W_P — W — W_L 10 20 30				
99.0	Ground Level														
0.0	Silt with sand, traces of gravel. Loose to Dense Brown (Fill)		1	SS	6	320									8 36 (56)
			2	SS	11										
			3	SS	12	310									5 38 (57)
			4	SS	32										
91.6			5	SS	11	300									
7.4	Sandy silt, traces of gravel & organics. Compact Grey		6	SS	18										 5 43 (52) organic content
89.5			7	SS	9									8.4	
9.5	Silt with sand, traces of gravel. Grey Very Dense (Till)		8	SS	100/2"	290									1 35 (64)
			9	SS	100/6"										
86.4			10	SS	100/9"										
12.6	End of Borehole					280									

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-12

JOB 73-11022 LOCATION Co-ords. 15,945,220 N; 1,169,082 E.
 W.P. 44-71-08 BORING DATE May 10, 1973
 DATUM Geodetic BOREHOLE TYPE Auger

ORIGINATED BY PK
 COMPILED BY PK
 CHECKED BY PK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P — W — W_L WATER CONTENT % 10 20 30			BULK DENSITY γ P.C.F. GR. SA. SI. CL.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
89.8	Ground Level												
0.0	Silty sand, traces of gravel. Grey		1	SS	100	290							
87.6	Very Dense (Till)		2	SS	100	9"							1 54 (45)
2.2	End of Borehole												
						280							

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-14

JOB 73-11022

LOCATION Co-ords. 15,945,240 N; 1,169,136 E.

ORIGINATED BY PK

W.P. 44-71-08

BORING DATE May 10, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY

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		25	50	75	100	125	w_p	w	w_L		
90.1	Ground Level															
89.5	Gravelly sand. Grey Fill	X														
0.6	Silt with sand, traces of gravel. Grey		1	SS	100											
			2	SS	100/26"	290										
			3	SS	100/27"											
	Very Dense (Till)		4	SS	100/28"											
84.6			5	SS	100/30"	280										
5.5	End of Borehole					270										

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-15

JOB 73-11022

LOCATION Co-ords. 15,945,280 N; 1,169,175 E.

ORIGINATED BY PK

W.P. 14-71-08

BORING DATE May 7 and 8, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY PK

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 25 50 75 100 125	LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W W _P — W — W _L	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE					
93.1	Ground Level								
0.0	Sandy silt.								
	Loose		1	SS	8				0 47 (53)
89.4	Brown		2	SS	9				
3.7	Sand with gravel, traces		3	SS	17				32 55 (13)
88.5	of silt. Compact		4	SS	72				12 51 (37)
4.6	Sand with silt, some gravel. (Till)		5	SS	100	5"			
	Grey								
85.4	Very Dense		6	SS	100	6"			9 90 (1)
7.7	Sand, traces of gravel		7	SS	100	8"			
84.4	& silt. Very Dense								
8.7	End of Borehole								

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE N^o 033-16

JOB 73-11022 LOCATION Co-ords. 15,945,252 N; 1,169,188 E. ORIGINATED BY PK
W.P. 44-71-08 BORING DATE May 4 to May 7, 1973 COMPILED BY PK
DATUM Geodetic BOREHOLE TYPE Washboring CHECKED BY PK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P W W_L			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				WATER CONTENT % 10 20 30				
97.8	Ground Level					320									
0.0	Sand with gravel, some silt. (Fill)		1	SS	28										
			2	SS	10	310									34 48 (18)
92.6	Brown Compact		3	SS	9										
5.2	Sandy silt, traces of gravel.		4	SS	17	300									
90.1	Compact		5	SS	14										
7.7	Sand with gravel, traces of silt.		6	SS	57										
	Grey		7	SS	100 2 1/2"	290									
87.7	Very Dense		8	SS	100 2 1/2"										
10.1	End of Borehole					280									

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

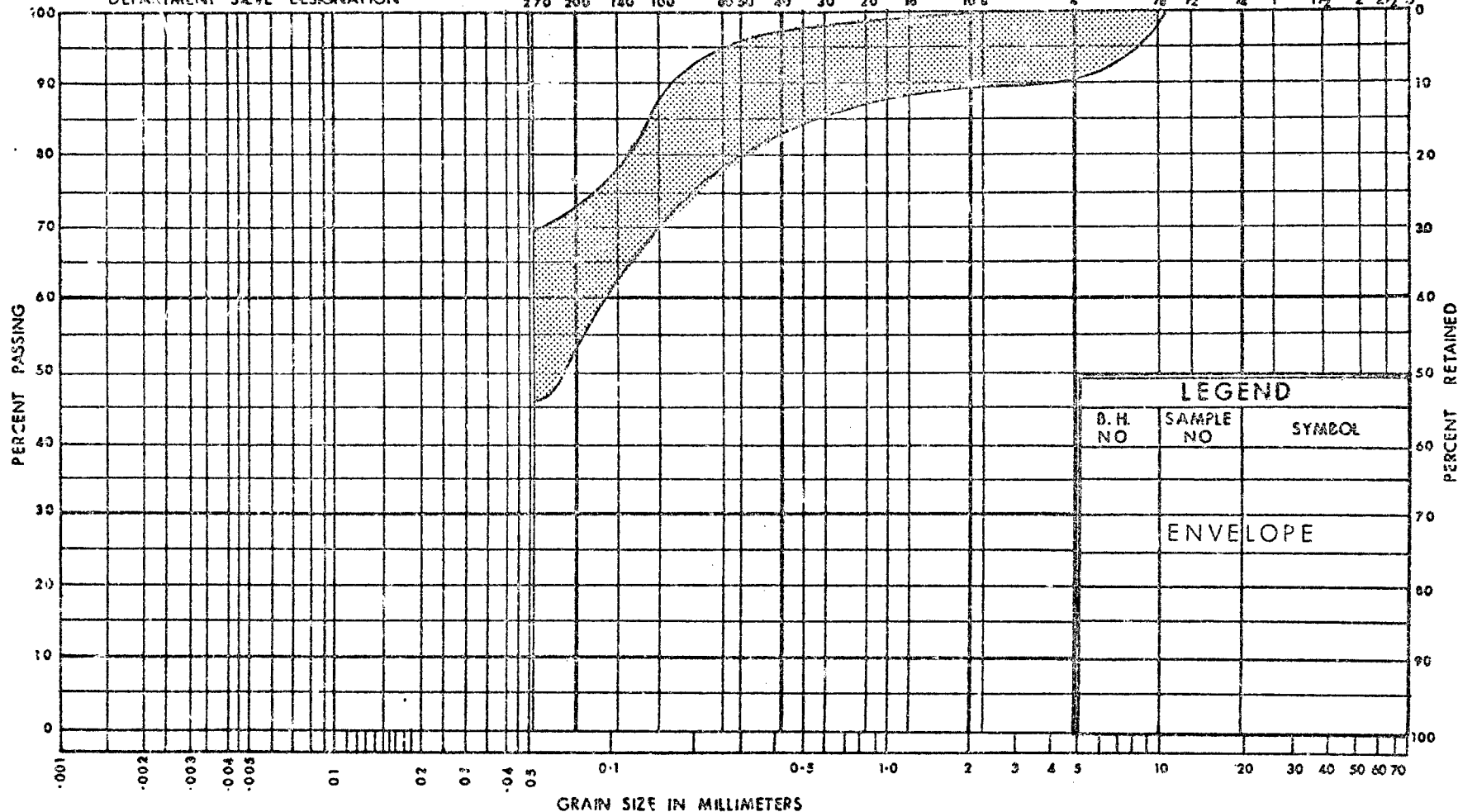
Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

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FIG.3

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