

Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone: (905) 567-4444
Fax: (905) 567-6561



**FOUNDATION INVESTIGATION
AND DESIGN REPORT
HOGG'S HOLLOW BRIDGE AND RETAINING WALLS
HIGHWAY 401
EASTBOUND AND WESTBOUND CORE LANES
FROM AVENUE ROAD TO BAYVIEW AVENUE
G.W.P. 5-98-00**

Submitted to:

Morrison Hershfield Limited
235 Yorkland Boulevard, Suite 600
Toronto, Ontario
M2J 1T1

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PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the rehabilitation/widening of Highway 401 eastbound and westbound core lanes between Avenue Road and Bayview Avenue, in the City of Toronto. Foundation engineering services are required for the widening of Hogg's Hollow bridge, replacement of the retaining walls associated with Hogg's Hollow bridge, new high mast light poles, new trichord overhead signs, and replacement of a noise barrier wall.

This report addresses the foundation investigation carried out for the northward widening of the Hogg's Hollow bridge (Highway 401 westbound core structure), and the replacement of the retaining walls at the northwest, southwest, northeast and southeast quadrants of the Hogg's Hollow bridge (westbound and eastbound core structures)

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal for Agreement No. 2005-E-0035, issued in January 2005, and in Section 6.8 of MH's *Technical Proposal* for G.W.P. 5-98-00.

2.0 SITE DESCRIPTION

The section of Highway 401 between Avenue Road and Bayview Avenue crosses the West Don River valley immediately to the west of Yonge Street in Toronto, Ontario. The Hogg's Hollow bridges carry the Highway 401 core and collector lanes over the West Don River valley. The Highway 401 grade rises from about Elevation 169 m at the west end of the core bridges, to about Elevation 171 m at the east end of the core bridges.

The ground surface immediately adjacent to the West Don River banks is at about Elevation 134 m, while the normal water level in the West Don River is at about Elevation 131 m to 131.5 m. The valley slopes rise approximately 33 m to 40 m, to about Elevation 167 m at the western crest of the valley, and about Elevation 171 m to 174 m at the eastern crest of the valley. To the west and east of the West Don River valley, the "tableland" is relatively flat; at about Elevation 167 m to 171 m to the west of the river valley, and about Elevation 174 m to 178 m to the east of the river valley. Based on an elevation survey provided by MH, the western valley slope has an overall orientation of approximately 4 horizontal to 1 vertical (4H:1V), and the eastern valley slope has an overall orientation of approximately 2H:1V.

The Highway 401 eastbound core bridge was originally constructed in the 1920s as an eight-span bridge; subsequently, an additional span was added on each end and new west and east abutments were constructed; the original west and east abutments became Piers 1 and 9, respectively. This structure has end spans of approximately 17.2 m in length, and spans between the piers of about 42.7 m to 43.4 m. Based on the available design drawings for Contract 28-01, Piers 1 to 9 for the eastbound core bridge are supported on groups of driven timber piles; the pile lengths are unknown. The newer abutments for this structure are understood to be supported on driven steel H-piles (west abutment) and a spread footing that apparently becomes pile-supported toward its northern end (east abutment); the lengths of these piles are also not known. The foundation types and founding elevations for the Highway 401 eastbound core structure are summarized below.

<i>Foundation Element</i>	<i>Foundation Type (Number of Piles)</i>	<i>Elevation of Underside of Pile Cap</i>
West Abutment	Driven steel H-piles	N/A
Pier 1 (former West Abutment)	Timber piles (108)	158.0 m
Pier 2	Timber piles (48)	145.7 m
Pier 3	Timber piles (60)	138.0 m
Pier 4	Timber piles (65)	135.9 m
Pier 5	Timber piles (70)	133.3 m
Pier 6	Timber piles (78)	130.1 m
Pier 7	Timber piles (84)	130.0 m
Pier 8	Timber piles (84)	132.4 m
Pier 9 (former East Abutment)	Timber piles (108)	159.7 m
East Abutment	Spread footing/driven steel H-piles	N/A

The Highway 401 westbound core bridge was constructed in the early 1960s as a ten-span bridge. This structure has end spans of approximately 18.6 m in length, and spans between the piers of about 42.1 m to 42.7 m. Based on the available drawings for Contracts 59-81 and 59-151, the abutments and Piers 6 to 8 are supported on driven steel H-piles, while Piers 1 to 5 and Pier 9 are supported on spread footings. The foundation types and founding elevations for the Highway 401 westbound core structure are summarized below.

<i>Foundation Element</i>	<i>Foundation Type</i>	<i>Founding Elevation</i>
West Abutment	Vertical and battered 10BP42 piles, approximately 9.1 m in length	Pile cap underside: 164.7 m Pile tip: 155.6 m
Pier 1	Pair of spread footings, each approximately 3.8 m x 5.5 m	155.4 m
Pier 2	Pair of spread footings, each approximately 4.9 m x 11.6 m	146.3 m
Pier 3	Pair of spread footings, each approximately 4.9 m x 12.2 m	137.2 m
Pier 4	Pair of spread footings, each approximately 5.2 m x 12.8 m	134.1 m
Pier 5	Pair of spread footings, each approximately 5.2 m x 12.8 m	132.6 m
Pier 6	Two groups of 45 vertical 12BP53 piles, approximately 10.7 m in length	Pile cap underside: 131.1 m Pile tip: 120.4 m
Pier 7	Two groups of 55 vertical 12BP53 piles, approximately 7.6 m in length	Pile cap underside: 129.5 m Pile tip: 121.9 m
Pier 8	Two groups of 60 vertical/battered 12BP53 piles, about 7.6 m in length	Pile cap underside: 136.6 m Pile tip: 129.0 m
Pier 9	Pair of spread footings, each approximately 3.8 m x 5.5 m	158.5 m
East Abutment	Vertical and battered 10BP42 piles, approximately 4.6 m in length	Pile cap underside: 166.6 m Pile tip: 162.0 m

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Hogg's Hollow bridge site in April and June 2007, during which time eleven boreholes (Boreholes 07-1 to 07-9, 07-12 and 07-13) were advanced using track- and truck-mounted drill rigs, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The borehole locations are shown on Drawing 1.

The boreholes were advanced to depths ranging from 9.8 m to 18.6 m, using solid stem augers. Soil samples were obtained at 0.76 m and 1.5 m intervals of depth, using 50 mm outer diameter split-spoon samplers driven by an automatic or manual hammer (as noted on the borehole records) in accordance with the Standard Penetration Test (SPT) procedure.

The water level in the open boreholes was observed throughout the drilling operations. Upon completion, all boreholes were backfilled to ground surface using bentonite in accordance with Ontario Regulation 128 (amendment to Ontario Regulation 903).

The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. Index and classification testing, consisting of water contents, Atterberg limits and grain size distributions, was carried out on selected soil samples.

The northings, eastings and elevations of the as-drilled borehole locations were measured in the field by a member of Golder's technical staff, relative to existing structure elements and other site features. The borehole locations (including MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) are summarized below and are shown on Drawing 1.

<i>Borehole Number</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
07-1	4,845,882.4	312,074.8	171.6
07-2	4,845,523.4	311,722.8	168.5
07-3	4,845,828.9	312,064.6	171.3
07-4	4,845,839.5	312,028.7	171.0
07-5	4,845,582.7	311,740.6	169.1
07-6	4,845,555.9	311,758.7	169.0
07-7	4,845,813.6	312,047.5	171.0
07-8	4,845,548.6	311,702.2	169.5
07-9	4,845,587.2	312,048.4	171.1
07-12	4,845,830.7	312,009.1	162.0
07-13	4,845,597.3	311,748.0	161.5

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This area of Highway 401 is located within the Peel Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). A surficial till sheet, which generally follows the surface topography, is generally present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with occasional sand to silt zones; it is mapped in this area as the Halton Till. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial meltwater ponds scattered throughout the Peel Plain and concentrated near river valleys, such as the West Don River valley. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay.

4.2 Subsurface Conditions

Eleven boreholes (Boreholes 07-01 to 07-09, 07-12 and 07-13) were advanced at this site at the locations shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are summarized on the Record of Borehole sheets; the laboratory test results are also shown on Figures 1 to 7. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole location.

In summary, the subsoils encountered in the boreholes consists of fill overlying an upper deposit of firm to hard clayey silt till, underlain by a deposit of dense to very dense sand, to sand and gravel, to sandy silt, in turn underlain by a lower deposit of hard clayey silt till.

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

4.2.1 Asphalt / Fill

Boreholes 07-1 to 07-9 were drilled through the existing Highway 401 pavement structure, and encountered approximately 100 mm to 500 mm of asphalt and concrete, typically overlying 0.2 m to 1.2 m of road base fill, which ranged in composition from sand and gravel, to gravelly sand, to sand containing some gravel.

In Boreholes 07-2 to 07-9, the road base fill was underlain by between 0.7 m and 3.1 m of embankment fill; the surface of the embankment fill was encountered in these boreholes between

Elevations 167.7 m and 170.5 m, and its base was encountered between Elevations 165.8 m and 169.8 m, typically rising away from the West Don River valley. Approximately 1.5 m of fill was encountered immediately below the ground surface in Borehole 07-12, which was drilled adjacent to Pier 9 on the east slope of the river valley; the surface of the fill was encountered in this borehole at Elevation 162.0 m, and its base at Elevation 160.5 m.

The embankment fill varies in composition from sand containing some silt and trace to some gravel, to sand and gravel containing trace silt, to clayey silt with sand to trace sand, trace gravel; the fill encountered in Borehole 07-12 adjacent to Pier 9 on the east slope of the river valley consists of sand and gravel containing concrete rubble. The results of grain size distribution tests completed on six selected samples of the embankment fill are shown on Figure 1.

Atterberg limits tests were conducted on four selected sample of the clayey silt fill and measured plastic limits of 12 to 14 per cent, liquid limits of 19 to 23 per cent, and plasticity indices of 7 to 10 per cent. These results, which are plotted on a plasticity chart on Figure 2, confirm that the cohesive portion of the embankment fill is a clayey silt of low plasticity.

The measured Standard Penetration Test (SPT) “N” values within the cohesionless embankment fill range from 6 to 33 blows per 0.3 m of penetration, indicative of a loose to dense (but typically compact) relative density, while the measured SPT “N” values in the cohesive embankment fill range from 7 to 34 blows per 0.3 m of penetration, indicative of a firm to hard (but typically stiff to very stiff) consistency. One SPT “N” value of 79 blows per 0.3 m of penetration was measured in the fill encountered adjacent to Pier 9 in Borehole 07-12; this SPT “N” value is indicative of a very dense relative density.

4.2.2 Upper Clayey Silt Till

An upper till deposit was encountered below the Highway 401 embankment fill in Boreholes 07-1 to 07-9, and below the fill in Borehole 07-12 adjacent to Pier 9; this upper till deposit was absent in Borehole 07-13, which was drilled adjacent to Pier 1 on the west valley slope. The surface of this till deposit was encountered between Elevation 165.8 m and 170.8 m below the Highway 401 embankment fill (generally rising away from the crest of the river valley), and at Elevation 160.5 m in Borehole 07-12 adjacent to Pier 9. The upper till deposit ranges in thickness from 2.3 m to 13.7 m in the boreholes in which it was fully penetrated.

The upper till deposit consists of clayey silt with sand to trace sand, and trace to some gravel; cohesionless soil interlayers are also present within the cohesive till deposit, such as encountered in Borehole 07-1 (sandy silt till) and in Borehole 07-3 (sand and silt). The results of grain size distribution tests completed on twelve selected samples of the clayey silt till deposit are shown on Figures 3A and 3B; Figure 3A also shows the grain size distribution for a sample from the 1.5 m thick interlayer of sand and silt that was encountered within the upper till deposit in Borehole 07-3. Boulders and cobbles were not encountered in this deposit during the borehole investigation; however, these glacially-derived soils may contain boulders and cobbles.

Atterberg limits testing was carried out on eighteen selected samples of the deposit, and measured plastic limits of 10 to 15 per cent, liquid limits of 16 to 29 per cent, and plasticity indices of 5 to 14 per cent. These results, which are plotted on plasticity charts on Figures 4A, 4B and 4C, confirm that the upper till deposit consists of clayey silt of low plasticity.

The measured SPT “N” values within the upper clayey silt till deposit range from 4 blows to greater than 100 blows per 0.3 m of penetration. The lower SPT “N” values of 4 to 15 blows per 0.3 m of penetration were measured in the upper portion of the clayey silt till deposit encountered in Boreholes 07-1, 07-3, 07-6 and 07-12, and this portion of the till at these locations has a firm to stiff consistency. Elsewhere, the SPT “N” values are greater than about 20 blows per 0.3 m of penetration, and generally greater than 30 blows per 0.3 m of penetration, indicative of a very stiff to hard (but typically hard) consistency.

4.2.3 Sand and Gravel to Sandy Silt

A cohesionless soil deposit was encountered below the upper clayey silt till in Boreholes 07-1 to 07-6, 07-8, 07-9 and 07-12, and directly below the ground surface in Borehole 07-13, which was drilled adjacent to Pier 1 on the west slope of the river valley. The surface of this deposit was encountered between Elevation 156.1 m and 164.9 m in all boreholes except Borehole 07-12 on the east slope of the river valley, where the deposit surface was encountered at Elevation 153.2 m. This deposit ranges in thickness from 3.4 m to 7.9 m in the boreholes in which it was fully penetrated.

This cohesionless soil deposit varies in composition from sand and gravel to gravelly sand containing some silt, to sand containing trace to some silt and gravel, to silty sand containing trace to some gravel, to sand and silt or sandy silt containing trace gravel and clay, to silt containing trace clay. The results of grain size distribution tests completed on ten selected samples of this deposit are shown on Figures 5A and 5B. Boulders and cobbles were not encountered in this deposit during the borehole investigation; however, these glacially-derived soils may contain boulders and cobbles.

The measured SPT “N” values within the sand and gravel to sandy silt deposit generally range from 39 to greater than 100 blows per 0.3 m of penetration, indicating that this deposit has a dense to very dense, but generally very dense, relative density. However, in Borehole 07-13, where this deposit was encountered immediately below the ground surface, the measured SPT “N” values range from 4 to 13 blows per 0.3 m of penetration, indicating that at this location the deposit has a loose to compact relative density.

4.2.4 Lower Clayey Silt Till

A lower glacial till deposit was encountered below the sand and gravel to sandy silt deposit in Boreholes 07-2, 07-5, 07-6, 07-8, 07-12 and 07-13. The surface of this lower till deposit was encountered between Elevation 155.4 m and 158.6 m in all of the boreholes except Borehole 07-12, which was drilled on the east slope of the river valley, where the surface of the lower till was encountered at Elevation 149.8 m. This lower till deposit was not fully penetrated in any of the boreholes.

The lower till consists of clayey silt with sand to trace sand, and trace to some gravel; the results of grain size distribution tests completed on six selected samples of this cohesive till are shown on Figure 6. Boulders and cobbles were not encountered in this deposit during the borehole investigation; however, these glacially-derived soils may contain boulders and cobbles.

Atterberg limits testing was carried out on six selected samples of the deposit, and measured plastic limits of 14 to 21 per cent, liquid limits of 20 to 33 per cent, and plasticity indices of 5 to 14 per cent. These results, which are plotted on a plasticity chart on Figure 7, confirm that the lower till deposit consists of clayey silt of low plasticity.

The measured SPT “N” values within the lower clayey silt till deposit range from 21 to greater than 100 blows per 0.3 m of penetration, but generally greater than 100 blows per 0.3 m of penetration; the lower SPT “N” values of 21 to 57 blows per 0.3 m of penetration were encountered near the surface of this lower till deposit in Boreholes 07-12 and 07-13, which were drilled on the east and west slopes of the river valley. The SPT “N” values indicate that the lower clayey silt till has a very stiff to hard, but generally hard, consistency.

4.3 Groundwater Conditions

The boreholes drilled as part of the current assignment were generally dry upon completion of drilling. However, in Boreholes 07-1, 07-3, and 07-9, the water level in the open borehole was measured between Elevation 156.7 m and 162.0 m (a depth of between 9.6 m and 14.6 m) on completion of drilling. These measurements do not represent stabilized groundwater levels.


The stabilized groundwater level across the site is expected to follow the surface topography of the West Don River valley. Based on water level monitoring completed in previous boreholes advanced at the site, the groundwater level in the “tableland” to the west of the valley is typically at or below approximately Elevation 167 m (at a depth of approximately 3 m to 5 m below the ground surface), while the groundwater level in the “tableland” to the east of the valley varies from approximately Elevation 164.5 m to 170.5 m (at a depth of about 1.5 m to 5 m below the ground surface), generally rising toward the east. Below the valley side slopes, the stabilized groundwater level is typically at a depth of between 5 m and 10 m; in the vicinity of Piers 1 and 9, the groundwater level has been measured between approximately Elevation 148 m and 150 m. The groundwater level declines to about Elevation 133 m (at a depth of approximately 1.5 m to 4.5 m) within the relatively flat floodplain area; this is slightly above the normal West Don River water level of approximately Elevation 131.0 m to 131.5 m.


The groundwater level across the site expected to fluctuate seasonally and is expected to rise during periods of high precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate with Golder. Mr. Jorge Costa, P.Eng., a Principal and Designated MTO Contact for Golder, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.


LISA C. COYNE, P.Eng.
Associate




Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Contact



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PART B

**FOUNDATION DESIGN REPORT
HOGG'S HOLLOW BRIDGE AND RETAINING WALLS
HIGHWAY 401
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6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical/foundation recommendations for the design of the proposed northward widening of the Highway 401 westbound core lane structure (Hogg's Hollow bridge) over the West Don River valley, and for the replacement of four retaining walls in the northwest, southwest, northeast and southeast quadrants of the Highway 401 westbound and eastbound core lane structures.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out design of the foundations for the proposed structure widening and retaining wall replacement. Where comments are made on construction they are provided in order to highlight those aspects which could affect design, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling, and the like.

6.2 Foundation Options

The existing Highway 401 westbound core structure – a ten-span bridge with the west abutment supported on driven steel H-piles, the east abutment supported on spread footings and driven steel H-piles, and the piers supported on either spread footings or steel H-piles – is to be widened by approximately 2.5 m to the north; consideration is being given to full replacement of the west and east abutments, at a location approximately 3 m behind the existing abutments, as part of this work. In addition, four retaining walls, approximately 28 m to 70 m in length and up to about 4.5 m in height, will be replaced in the northwest, southwest, northeast and southeast quadrants of the Hogg's Hollow core lane bridges.

At this site, in the vicinity of the abutments, Piers 1 and 9, and the retaining walls (i.e. near the west and east crests of the West Don River valley), up to about 4.5 m of Highway 401 embankment fill is present on top of an upper clayey silt till deposit, which typically has a very stiff to hard consistency. This upper till deposit is underlain by a dense to very dense cohesionless deposit, which varies in composition from sand and gravel, to sand, to sandy silt. This dense to very dense cohesionless deposit is, in turn, underlain by a lower deposit of hard clayey silt till.

6.2.1 Foundation Options – Abutments and Piers

Shallow and deep foundation options have been considered for support of the abutment widening or replacement and the northward widening of Piers 1 and 9. 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 generally very stiff to hard clayey silt till deposit:** Although strip or spread footings are feasible at this site, they are not the preferred solution from a foundations perspective due to the potential for up to about 15 mm of differential settlement between the existing structure and the new widened/replaced portions. In addition, deeper excavations (as compared to the 1.2 m depth requirement for frost protection purposes) would be required due to the presence of the valley slopes in front of the abutments and Piers 1 and 9, potentially increasing temporary excavation support requirements. In particular, northward widening of spread footings for Piers 1 and 9 would require relatively deep excavations at the crest of the valley slope, and immediately adjacent to foundation elements of the existing core and collector bridge piers.
- **Steel H-piles driven to found within “100-blow” clayey silt till, sand and gravel, sand, or sandy silt:** Driven steel H-piles are suitable and feasible for support of the abutment widenings or replacements, because differential settlement between the existing westbound core structure (which has both spread footing- and pile-supported foundation elements) and the widenings or replacements would be negligible. Driven steel H-piles are also suitable for support of the northward widening of Piers 1 and 9; however, the use of a full-size piling rig will be limited below the existing bridge deck at these piers, and it may not be possible to batter steel H-piles sufficiently or to achieve high enough capacities if they are driven using a “mini-rig” below the bridge deck. A full-size piling rig could be used from the existing bridge deck at this location, though it would require a minimum two-lane footprint during pile driving operations.
- **Caissons founded within “100-blow” clayey silt till, sand and gravel, sand, or sandy silt:** Caisson foundations are suitable and feasible for support of the abutment widenings or replacements and the northward widening of Piers 1 and 9. For all of the foundation elements, temporary or permanent liners would be required during installation to control the ground and, where present, groundwater, within cohesionless soil deposits; this requirement will make caisson installation less cost-effective than installation of driven steel H-piles. In addition, at Piers 1 and 9, a caisson rig will not fit under the bridge deck and would have to operate from on top of the bridge deck; discussions with caisson contractors have confirmed that this approach is feasible for these foundation elements. From a foundations perspective, caissons constructed using a rig mounted on the bridge deck are considered more practicable than driven steel H-piles for support of the northward widening of Piers 1 and 9, since a single vertical (non-battered) caisson is understood to offer sufficient axial and lateral resistance; in addition, if a single vertical caisson is used for the pier widenings, no pile cap will be required (i.e. caisson and column will be continuous).

- **Micropiles:** Micropiles are feasible at this site for the northward widening of Piers 1 and 9, where the smaller micropile rig has an advantage over full-size pile driving or caisson rigs, which cannot operate below the bridge deck. However, micropiles would require a site-specific design, more piles than for caissons or other pile types, and are generally much more expensive than other types of foundations, and so these do not represent a preferred foundations solution for this site.

From a foundations perspective, driven steel H-piles are considered to be the most practicable and cost-effective foundation option for replacement of the west and east abutments, as these can be driven from the Highway 401 grade behind the existing abutments. Caisson foundations are considered to be the most practicable foundation option for northward widening of Piers 1 and 9, as a single caisson can support the widening load, this caisson can be continuous with the above-grade column (i.e. no pile cap would be required), and the caisson can be constructed from the existing bridge deck.

Recommendations for the various foundation options for the abutment widening or replacement and the northward widening of Piers 1 and 9 are provided in Section 6.3.

6.2.2 Foundation Options – Retaining Walls

Shallow and deep foundation options have been considered for support of the retaining wall replacements at this site. 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 2 following the text of this report.

- **Strip or spread footings founded on the generally very stiff to hard clayey silt till deposit:** Although strip or spread footings are feasible at this site, they are not the preferred solution from a foundations perspective as they would require deeper excavation to facilitate removal of the pile caps (and likely the upper portion of the piles) for the existing retaining walls.
- **Steel H-piles driven to found within “100-blow” clayey silt till, sand and gravel, sand, or sandy silt:** Driven steel H-piles are suitable and feasible for support of the retaining wall replacements. Depending on the location of the new retaining walls relative to the existing walls, it may be necessary to remove the existing pile caps and upper portion of the piles.
- **Caissons founded within “100-blow” clayey silt till, sand and gravel, sand, or sandy silt:** Caisson foundations are suitable and feasible for support of the retaining wall replacements. Temporary or permanent liners would be required during installation to control the ground and, where present, groundwater within cohesionless soil deposits; this requirement will make caisson installation less cost-effective than installation of driven steel H-piles.

- **Soldier pile and concrete lagging walls:** A soldier pile and concrete panel wall could be used for replacement of the retaining walls, although this type of wall is generally more advantageous in “top-down” construction applications (i.e., as part of a cut widening, rather than for an embankment widening). In addition, installation of soil anchors may be difficult or impossible given the limited horizontal distance between the new retaining walls and the adjacent collector lane structures.
- **Retained soil system (RSS) walls:** RSS walls are feasible at this site for replacement of the existing retaining walls in the northwest, southwest, northeast and southeast quadrants of the Hogg’s Hollow core bridge structures, where there is limited working space between the existing core and collector lane bridge structures. The use of RSS walls for these retaining wall replacements will require less vertical excavation in the limited working space available at these locations, and would allow the existing pile caps/footings to remain in place below the reinforced soil mass following removal of the existing retaining wall stems.

From a foundations perspective, RSS walls are considered to be the most practicable and cost-effective option for replacement of the existing retaining walls in the northwest, southwest, northeast and southeast quadrants of the Highway 401 core lane bridges, as they can be constructed in smaller sections suitable to the local space constraints, and as they do not require removal of the existing pile caps.

Recommendations for the various foundation options for the retaining wall replacements are provided in Section 6.4.

6.3 Abutments and Piers

6.3.1 Shallow Foundations

Founding Elevations

For support of new west and east abutments or northward widening of the west and east abutments, and northward widening of Piers 1 and 9, strip or spread footings should be founded below the fill and any loose surficial soils, on the generally very stiff to hard upper clayey silt till deposit. 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. For the Hogg’s Hollow bridge foundation elements, based on the near-surface soil conditions and the sloping ground conditions adjacent to the West Don River valley, deeper founding levels are typically required.

The following founding elevations are recommended for strip or spread footings for support of the abutments and Piers 1 and 9:

<i>Foundation Element</i>	<i>Borehole Number(s)</i>	<i>Strip/Spread Footing Founding Elevation</i>
West Abutment	07-5, 07-6	165.5 m
Pier 1	07-13	156.0 m
Pier 9	07-12	158.0 m
East Abutment	07-4, 07-7	167.0 m

The clayey silt till that will typically form the subgrade for the footings will be susceptible to softening and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick layer of lean mix concrete or mass concrete be placed on the footing subgrade to form a working mat, to protect the subgrade from degradation. If shallow foundations are adopted, the use of a lean mix concrete or mass concrete mat can be addressed either by a note on the General Arrangement and Foundation Layout drawings, or a Non-Standard Special Provision (as discussed further in Section 6.8).

Geotechnical Resistance

Strip or spread footings placed on the properly prepared, very stiff to hard clayey silt till or compact cohesionless subgrade, at or below the design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) given below. These design values take into account the depth of embedment (based on the design founding elevations), the layered nature of the soil and proximity to the West Don River valley slope, where applicable.

<i>Foundation Element</i>	<i>Footing Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS*</i>
West Abutment	2 m	400 kPa	250 kPa
	3 m	600 kPa	300 kPa
	4 m	800 kPa	300 kPa
Pier 1	1.5 m	950 kPa	300 kPa
	3 m	1,100 kPa	300 kPa
	4.5 m	1,300 kPa	300 kPa
Pier 9	1.5 m	950 kPa	300 kPa
	3 m	1,100 kPa	300 kPa
	4.5 m	1,300 kPa	300 kPa
East Abutment	2 m	400 kPa	275 kPa
	3 m	600 kPa	300 kPa
	4 m	800 kPa	300 kPa

* For 25 mm of settlement.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC)*.

Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the generally very stiff to hard clayey silt till, the coefficient of friction, $\tan \phi'$, can be taken as 0.45. This value is unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3.2 Steel H-Pile Foundations

Steel H-pile foundations are feasible for support of the new or widened east and west abutments and the northward widening of Piers 1 and 9. However, the head room beneath the Highway 401 westbound core and collector structures is relatively limited for the use of full-size, conventional pile driving equipment to operate from the existing ground surface at the Pier 1 and Pier 9 sites. Other pile driving equipment options or configurations exist for the Pier 1 and 9 sites, as follows:

- A mini-rig can operate in low headroom (approximately 2.1 m), but it is relatively slow, would require pile splicing approximately every 1.5 m, and would only be able to develop a factored axial geotechnical resistance of about 400 kN to 500 kN per pile. This equipment cannot achieve 1H:3V pile batters; the maximum batter that can be achieved is approximately 1H:6V.
- A larger mini-rig can operate with head room of approximately 4.3 m, and would be faster and more powerful than the smaller mini-rig option, requiring less frequent pile splicing. This larger mini-rig would be able to develop a factored axial geotechnical resistance of about 600 kN per pile. As for the smaller mini-rig, although small pile batters may be achieved (1H:6V), the larger mini-rig cannot achieve 1H:3V pile batters.
- A full-size, conventional pile driving rig can operate from the existing bridge deck. However, it may still be difficult to achieve a 1H:3V pile batter based on horizontal clearance from the Highway 401 westbound core lane bridge deck. This approach may also require larger highway closures/staging areas.
- The use of vertical piles in combination with dead-man reaction anchors to supplement lateral resistance are not considered suitable due to the difficulty associated with excavation back into the abutment foreslope.

Founding Elevations

The widened or new abutments and the northward widenings for Piers 1 and 9 could be supported on steel H-piles driven to found within the “100-blow” soil (which varies in composition from clayey silt till, to sand and gravel, to sand to sandy silt). For design, the following pile tip levels may be assumed based on the borehole results:

<i>Foundation Element</i>	<i>Borehole Number(s)</i>	<i>Pile Tip Elevation</i>
West Abutment	07-5, 07-6	North End – 155.0 m South End – 160.5 m
Pier 1	07-13	149.0 m
Pier 9	07-12	144.5 m
East Abutment	07-4, 07-7	153.5 m

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the clayey silt till and glaciofluvial deposits at this site. It is recommended that the piles be fitted with standard H-pile bearing points (such as Titus or Pruyne points), in accordance with the manufacturer’s specifications, for protection during pile driving; this requirement should be noted on the Contract Drawings.

Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal within the hard clayey silt till, very dense sand and gravel, or very dense sand to sandy silt, a factored axial resistance of 1,650 kN can be used for design; the axial geotechnical resistance at SLS may be taken as 1,100 kN.

Pile installation should be in accordance with MTO’s Special Provision SP903S01. 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. For piles driven into the hard or very dense deposits to the design tip elevations given in the preceding section, the following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.5 is applied to the use of the Hiley formula:

“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,300 kN per pile.”

Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below (CFEM, 1992, as noted in Section 6.8.7.3 of the *CHBDC*).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (MPa/m);
 n_h is the constant of subgrade reaction (MPa/m);
 z is the depth (m); and
 B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
 s_u is the undrained shear strength of the soil (kPa); and
 B is the pile diameter (m).

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. Approximate elevation intervals are given in the table for each deposit for each foundation element; however, the deposit boundaries vary slightly at the abutments, and reference can be made to the borehole records and to the interpreted stratigraphic sections on Drawing 1 to assess the variation along each abutment.

<i>Soil Unit</i>	<i>n_h</i>	<i>s_u</i>
West Abutment:		
Fill above Elevation 166 m	5 MPa/m	—
Firm to hard clayey silt till between Elevations 166 m and 162 m	—	100 kPa
Dense to very dense sand to silt between Elevations 162 m and 156 m	20 MPa/m	—
Hard clayey silt till below Elevation 156 m	—	500 kPa
Pier 1:		
Loose to compact silty sand to sandy silt above Elevation 157 m	3 MPa/m	—
Very stiff to hard clayey silt till between Elevations 157 m and 151 m	—	250 kPa
Hard clayey silt till below Elevation 151 m	—	500 kPa
Pier 9:		
Stiff to very stiff clayey silt till above Elevation 158 m	—	100 kPa
Hard clayey silt till between Elevations 158 m and 153 m	—	250 kPa
Very dense sand and silt between Elevations 153 m and 150 m	20 MPa/m	—
Hard clayey silt till below Elevation 150 m	—	500 kPa
East Abutment:		
Fill above Elevation 167 m	—	150 kPa
Very stiff to hard clayey silt till between Elevations 167 m and 159 m	—	200 kPa
Very dense sand and silt below Elevation 159 m	20 MPa/m	—

The following maximum factored lateral resistances at ULS, and maximum lateral resistances at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for HP 310x110 piles, based on the “Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS” provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*, using the values provided for piles with a flange width of 310 mm (i.e., HP 310x79 sections):

- For Pier 1:
 - Maximum factored lateral resistance at ULS = 110 kN
 - Maximum lateral resistance at SLS = 40 kN
- For the south half of the west abutment:
 - Maximum factored lateral resistance at ULS = 160 kN
 - Maximum lateral resistance at SLS = 65 kN
- For the north half of the west abutment, the east abutment and Pier 9:
 - Maximum factored lateral resistance at ULS = 200 kN
 - Maximum lateral resistance at SLS = 110 kN

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

<i>Pile Spacing in direction of Loading (d = Pile Diameter)</i>	<i>Subgrade Reaction Reduction Factor</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2.
Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.3.3 Caisson Foundations

Caisson foundations are feasible for support of the new or widened east and west abutments and the northward widening of Piers 1 and 9. However, the head room beneath the Highway 401 westbound core and collector structures will not allow a full-size, conventional caisson rig to operate from the existing ground surface at the Pier 1 and Pier 9 sites. Golder has undertaken

discussions with a caisson contractor and has confirmed that it would be feasible to operate a caisson rig from the existing bridge deck to install caissons for support of the northward widening of Piers 1 and 9. The caisson rig would likely require working space equivalent to two lanes plus a shoulder. Battered caissons would not be recommended when drilling from the bridge deck based on constructability (clearance) and safety considerations.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur where water-bearing cohesionless soils are present at/near the caisson base. If caisson foundations are adopted for support of any of the foundation elements associated with the core lane structure widening, a liner would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is expected that the liner would be installed (and removed, if a temporary liner is used) using a vibratory hammer; as discussed further under *Construction Considerations* in Section 6.8, vibration monitoring is recommended during liner installation and removal, if caisson foundations are adopted at the site. Further, it is recommended that an NSSP be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction; this recommendation is summarized under *Construction Considerations* in Section 6.8.

Construction experience in similar soil conditions has demonstrated that temporary liners can be difficult to withdraw, owing to the length of the liners and the hard/very dense nature of the 100-blow material. Such difficulties can result in “necking” of the caisson, although this can be controlled by tremie-pumping the concrete into the caisson and ensuring that the base of the liner always remains below the surface of the pumped concrete during withdrawal. Alternatively, permanent liners could be considered for the construction of the caissons in these soil conditions.

Founding Elevations

The widened or new abutments and the northward widenings for Piers 1 and 9 could be supported on caissons founded within the “100-blow” soil (which varies in composition from clayey silt till, to sand and gravel, to sand to sandy silt). For design, the following caisson base elevations may be assumed based on the borehole results:

<i>Foundation Element</i>	<i>Borehole Number(s)</i>	<i>Caisson Base Elevation</i>
West Abutment	07-5, 07-6	North End – 155.5 m South End – 160.5 m
Pier 1	07-13	149.0 m
Pier 9	07-12	145.0 m
East Abutment	07-4, 07-7	154.0 m

Axial Geotechnical Resistance

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on “socketting” approximately 2 m into the “100-blow” till/residual soil deposit. Using the design elevations given above, and assuming that all caisson excavations are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS are given below for various caisson diameters:

<i>Caisson Diameter</i>	<i>Axial Geotechnical Resistance</i>	
	Factored ULS	SLS
0.76 m	2,000 kN (Approximately 2650 kPa)	1,350 kN (Approximately 2,900 kPa)
0.9 m	2,800 kN (Approximately 4,400 kPa)	1,850 kN (Approximately 2,900 kPa)
1.2 m	4,600 kN (Approximately 4,100 kPa)	3,000 kN (Approximately 2,700 kPa)
1.5 m	7,250 kN (Approximately 4,100 kPa)	4,750 kN (Approximately 2,700 kPa)

Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.3.2.

The following maximum factored lateral resistances at ULS and maximum lateral resistances at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons, based on the “Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS” provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*:

- For Pier 1:
 - Maximum factored lateral resistance at ULS = 225 kN
 - Maximum lateral resistance at SLS = 90 kN
- For the south half of the west abutment:
 - Maximum factored lateral resistance at ULS = 325 kN
 - Maximum lateral resistance at SLS = 150 kN
- For the north half of the west abutment, the east abutment and Pier 9:
 - Maximum factored lateral resistance at ULS = 400 kN
 - Maximum lateral resistance at SLS = 250 kN

Values for alternative caisson diameters can be developed if larger diameter caisson foundations are adopted for support of foundation elements at this site.

Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.4 Retaining Walls

6.4.1 Shallow Foundations

Founding Elevations

For support of the new retaining walls, strip or spread footings should be founded below the fill and any loose surficial soils, on the generally very stiff to hard upper clayey silt till deposit; further, 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. The following founding elevations are recommended for strip or spread footings for support of the new retaining walls:

<i>Foundation Element</i>	<i>Borehole Number(s)</i>	<i>Strip/Spread Footing Founding Elevation</i>
Northwest Retaining Wall	07-5, 07-8	165.5 m
Southwest Retaining Wall	07-2, 07-6	166.0 m
Northeast Retaining Wall	07-1, 07-4, 07-9	166.5 m
Southeast Retaining Wall	07-3, 07-7	West End – 167.0 m East End – 168.5 m

The clayey silt till that will typically form the subgrade for the footings will be susceptible to softening and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick layer of lean mix concrete or mass concrete be placed on the footing subgrade to form a working mat, to protect the subgrade from degradation. If shallow foundations are adopted, the use of a lean mix concrete or mass concrete mat can be addressed either by a note on the General Arrangement and Foundation Layout drawings, or a Non-Standard Special Provision (as discussed further in Section 6.8).

Geotechnical Resistance

Strip or spread footings placed on the properly prepared subgrade, at or below the design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) given below. These design values take into account the depth of embedment (based on the design founding elevations) and proximity to the West Don River valley slope, where applicable.

Foundation Element	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
Northwest Retaining Wall	1 m	750 kPa	275 kPa
	2 m	900 kPa	275 kPa
Southwest Retaining Wall	1 m	500 kPa	275 kPa
	2 m	650 kPa	275 kPa
Northeast Retaining Wall	1 m	750 kPa	350 kPa
	2 m	900 kPa	400 kPa
Southeast Retaining Wall	1 m	750 kPa	350 kPa
	2 m	900 kPa	400 kPa

* For 25 mm of settlement.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC)*.

Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the generally very stiff to hard clayey silt till, the coefficient of friction, $\tan \phi'$, can be taken as 0.45. This value is unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.4.2 Steel H-Pile Foundations

Founding Elevations

The new retaining walls could be supported on steel H-piles driven to found within the “100-blow” soil (which varies in composition from clayey silt till, to sand and gravel, to sand to sandy silt). For design, the following pile tip levels may be assumed based on the borehole results:

Foundation Element	Borehole Number(s)	Pile Tip Elevation
Northwest Retaining Wall	07-5, 07-8	155.0 m
Southwest Retaining Wall	07-2, 07-6	160.5 m
Northeast Retaining Wall	07-1, 07-4, 07-9	West End – 153.5 m East End – 158.0 m
Southeast Retaining Wall	07-3, 07-7	157.0 m

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the clayey silt till and glaciofluvial deposits at this site. It is recommended that the piles be fitted with standard H-pile bearing points (such as Titus or Pruyne points), in accordance with the manufacturer's specifications, for protection during pile driving; this requirement should be noted on the Contract Drawings.

Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal within the hard clayey silt till, very dense sand and gravel, or very dense sand to sandy silt, a factored axial resistance of 1,650 kN can be used for design. The axial geotechnical resistance at SLS may be taken as 1,100 kN.

Pile installation should be in accordance with MTO's Special Provision SP903S01. 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. For piles driven into the hard or very dense deposits to the design tip elevations given in the preceding section, the following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.5 is applied to the use of the Hiley formula:

"Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,300 kN per pile."

Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below (CFEM, 1992, as noted in Section 6.8.7.3 of the *CHBDC*).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (MPa/m);
 n_h is the constant of subgrade reaction (MPa/m);
 z is the depth (m); and
 B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B} \quad \text{where} \quad \begin{array}{l} k_h \text{ is the coefficient of horizontal subgrade reaction (kPa/m);} \\ s_u \text{ is the undrained shear strength of the soil (kPa); and} \\ B \text{ is the pile diameter (m).} \end{array}$$

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. Approximate elevation intervals are given in the table for each deposit for each retaining wall foundation element; however, the deposit boundaries vary slightly at each of the foundation elements, and reference can be made to the borehole records and to the interpreted stratigraphic sections on Drawing 2 to assess the variation along each foundation element.

<i>Soil Unit</i>	<i>n_h</i>	<i>s_u</i>
Northwest Retaining Wall:		
Fill above Elevation 166 m	5 MPa/m	–
Very stiff to hard clayey silt till between Elevations 166 m and 163 m	–	150 kPa
Dense to very dense sand/gravel between Elevations 163 m and 156.5 m	25 MPa/m	–
Hard clayey silt till below Elevation 156.5 m	–	500 kPa
Southwest Retaining Wall:		
Fill above Elevation 167 m	5 MPa/m	–
Firm to hard clayey silt till between Elevations 167 m and 163 m	–	100 kPa
Very dense sand/gravel between Elevations 163 m and 157 m	25 MPa/m	–
Hard clayey silt till below Elevation 157 m	–	500 kPa
Northeast Retaining Wall:		
Fill/Stiff to hard clayey silt till above Elevation 167 m	–	150 kPa
Generally hard clayey silt till between Elevations 167 m and 159 m	–	250 kPa
Very dense silty sand to sandy silt below Elevation 159 m	20 MPa/m	–
Southeast Retaining Wall:		
Fill above Elevation 168 m	–	100 kPa
Very stiff to hard clayey silt till between Elevations 168 m and 159 m	–	200 kPa
Very dense sand and silt between Elevations 159 m and 157 m	20 MPa/m	–
Hard clayey silt till below Elevation 157 m	–	500 kPa

The following maximum factored lateral resistances at ULS, and maximum lateral resistances at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for HP 310x110 piles, based on the “Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS” provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*, using the values provided for piles with a flange width of 310 mm (i.e., HP 310x79 sections):

- For the southwest retaining wall:
 - Maximum factored lateral resistance at ULS = 160 kN
 - Maximum lateral resistance at SLS = 65 kN
- For all other retaining walls:
 - Maximum factored lateral resistance at ULS = 200 kN
 - Maximum lateral resistance at SLS = 110 kN

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

<i>Pile Spacing in direction of Loading (d = Pile Diameter)</i>	<i>Subgrade Reaction Reduction Factor</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2.
Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.4.3 Caisson Foundations

Caisson foundations are feasible for support of the new retaining walls. Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur where water-bearing cohesionless soils are present at/near the caisson base. If caisson foundations are adopted for support of the new retaining walls, a liner would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is expected that the liner would be installed (and removed, if a temporary liner is used) using a vibratory hammer; as discussed further under *Construction Considerations* in Section 6.8, vibration monitoring is recommended during liner installation and removal, if caisson foundations are adopted at the site. Further, it is recommended that an NSSP be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction; this recommendation is summarized under *Construction Considerations* in Section 6.8.

Construction experience in similar soil conditions has demonstrated that temporary liners can be difficult to withdraw, owing to the length of the liners and the hard/very dense nature of the 100-blow material. Such difficulties can result in “necking” of the caisson, although this can be controlled by tremie-pumping the concrete into the caisson and ensuring that the base of the liner always remains below the surface of the pumped concrete during withdrawal. Alternatively, permanent liners could be considered for the construction of the caissons in these soil conditions.

Founding Elevations

The new retaining walls could be supported on caissons founded within the “100-blow” soil (which varies in composition from clayey silt till, to sand and gravel, to sand to sandy silt). For design, the following caisson base elevations may be assumed based on the borehole results:

<i>Foundation Element</i>	<i>Borehole Number(s)</i>	<i>Caisson Base Elevation</i>
Northwest Retaining Wall	07-5, 07-8	155.5 m
Southwest Retaining Wall	07-2, 07-6	160.5 m
Northeast Retaining Wall	07-1, 07-4, 07-9	West End – 154.0 m East End – 158.5 m
Southeast Retaining Wall	07-3, 07-7	157.0 m

Axial Geotechnical Resistance

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on “socketting” approximately 2 m into the “100-blow” till/residual soil deposit. Using the design elevations given above, and assuming that all caisson excavations are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS are given below for various caisson diameters:

<i>Caisson Diameter</i>	<i>Axial Geotechnical Resistance</i>	
	Factored ULS	SLS
0.76 m	2,000 kN (Approximately 2650 kPa)	1,350 kN (Approximately 2,900 kPa)
0.9 m	2,800 kN (Approximately 4,400 kPa)	1,850 kN (Approximately 2,900 kPa)
1.2 m	4,600 kN (Approximately 4,100 kPa)	3,000 kN (Approximately 2,700 kPa)
1.5 m	7,250 kN (Approximately 4,100 kPa)	4,750 kN (Approximately 2,700 kPa)

Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.4.2.

The following maximum factored lateral resistances at ULS and maximum lateral resistances at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons, based on the “Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS” provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*:

- For the southwest retaining wall:
 - Maximum factored lateral resistance at ULS = 325 kN
 - Maximum lateral resistance at SLS = 150 kN

- For all other retaining walls:
 - Maximum factored lateral resistance at ULS = 400 kN
 - Maximum lateral resistance at SLS = 250 kN

Values for alternative caisson diameters can be developed if larger diameter caisson foundations are adopted for support of foundation elements at this site.

Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.4.4 Soldier Pile and Concrete Panel Walls

A soldier pile and concrete panel wall could be adopted for the approximately 2 m to 4.5 m high new retaining walls, provided that there is sufficient space between the new retaining wall face and the face of the existing collector structure wing walls/retaining walls.

This wall system would consist of soldier piles socketted to sufficient depth to provide the necessary passive resistance for the 2 m to 4.5 m retained soil height. Lateral support to the soldier pile and concrete panel wall system could be provided in the form of permanent soil anchors.

The concrete lagging panels should be backfilled using compacted granular fill, such as Granular A or Granular B Type II meeting the standards set out in MTO's Special Provision SP110S13, and in accordance with OPSD 3121.150, to aid in achieving proper drainage. A 50 mm thick insulation layer should also be provided immediately behind the wall to provide for frost protection to the soils behind the wall.

Passive Toe Resistance for Soldier Pile Sockets

The factored passive resistance at ULS in front of the soldier piles at any depth below the base of the wall may be assessed using the equation and the design parameters provided below:

$$P_p = 1.5 K_p \gamma' z B$$

- where
- P_p is the factored lateral resistance at ULS (kN) per metre length of socket;
 - K_p is the coefficient of passive earth pressure, which may be taken as 3.0;
 - γ' is the effective unit weight of the soil in front of the soldier pile socket, which may be taken as 20 kN/m³;
 - z is the depth from the ground surface in front of the pile to the base of the pile socket (m); and
 - B is the diameter of the soldier pile socket (m).

The equation above assumes that the lateral resistance acts over a width equal to three times the socket diameter. The upper 1.2 m of overburden should be ignored in the calculation of the passive resistance, to account for frost effects.

Permanent Soil Anchors

Soil anchor support must be designed to accommodate the loads applied from lateral earth pressures and surcharge pressures from area, line or point loads. Soil anchors may be sized based on the following factored bond stresses acting between the grout and very stiff to hard upper clayey silt till deposit.

<i>Soil Deposit</i>	<i>Single-Stage Grouted Anchors</i>	<i>Secondary Grouted Anchors</i>
Very stiff to hard clayey silt till	65 kPa	100 kPa

The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. The fixed length (bond zone) of the anchors should be maintained behind a line drawn upward at 45 degrees from the base of the soldier piles. The permanent soil anchors should be provided with suitable corrosion protection.

Anchor installation, grouting and testing should be carried out in accordance with MTO's Special Provision SP999S26.

6.4.5 Retained Soil System (RSS) Walls

Mechanically-reinforced soil retaining systems (retained soils system or RSS walls) are considered, from a foundations perspective, to be the most cost-effective and practicable solution for replacement of the existing retaining walls in the northwest, southwest, northeast and southeast quadrants of the Hogg's Hollow core bridge structures. A high appearance and high performance wall is recommended for use at this site.

Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall; this footing, and the RSS mass, should be founded below any topsoil, loose fill or unsuitable native soils. For this site, it is recommended that the existing fill material within the RSS wall footprint be subexcavated down to the level of the underside of the existing core structure pile caps prior to construction of both the facing footing and the RSS mass, as follows:

<i>Wall Location</i>	<i>Subexcavation Elevation</i>
Northwest Retaining Wall	164.6 m
Southwest Retaining Wall	166.1 m
Northeast Retaining Wall	167.0 m
Southeast Retaining Wall	167.2 m

The facing footing and reinforced soil mass can be constructed immediately on top of the subgrade exposed at the above-noted elevations; alternatively, the subexcavated soil can be replaced with compacted Ontario Provincial Standard Specification (OPSS) 1010 Granular A or Granular B Type II fill prior to construction of the facing footing and/or reinforced soil mass. This compacted granular pad should extend at least 0.3 m beyond the outside edge of the facing footing, then outward/downward at 1 horizontal to 1 vertical (1H:1V).

The pile caps for the existing retaining walls can be left in place or removed following removal of the existing retaining walls and prior to construction of the new RSS walls

Geotechnical Resistance

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which has been taken as two-thirds of the height of the wall, the factored geotechnical resistances at ULS and the geotechnical resistances at SLS (for 25 mm of settlement) given below may be used for assessment of the reinforced mass founded on the properly prepared compacted granular fill or on the stiff to hard clayey silt till deposit.

<i>RSS Wall Location</i>	<i>Wall Height</i>	<i>Assumed Reinforced Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>
Southwest RSS Wall	2.0 m	1.3 m	125 kPa	75 kPa
All Other RSS Walls	2.0 m	1.3 m	175 kPa	175 kPa
	4.5 m	3.0 m	300 kPa	175 kPa

The settlement of the new RSS walls adjacent to the Hogg's Hollow core structures will be up to 10 mm for the placement of up to 3.5 m to 4.5 m of additional fill for the new northwest, northeast and southeast retaining walls, and about 10 mm to 15 mm for the placement of up to 2 m of additional fill for the new southwest retaining wall.

Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted backfill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.6. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

Global Stability

The static global stability of RSS walls adjacent to the Hogg's Hollow core structures has been analyzed using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A target factor of safety of 1.5 against global failure of the RSS walls is normally used for the design under static conditions. This factor of safety is considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

Based on the analysis results, the factor of safety against global instability of RSS walls is greater than 1.5 for the four retaining wall areas, assuming that the reinforcing strips have a length equal to at least two-thirds of the height of the wall. The result of a static slope stability analysis for the highest RSS wall section is provided on Figure 8.

6.5 Static Stability of West Don River Valley Slopes

The following sections outline the methods and parameters used to assess the static stability of the West Don River valley slopes, and the results of the stability analyses.

6.5.1 Analysis Methods

Static slope stability analyses were performed for the West Don River valley slopes, based on a surveyed profile below the Highway 401 westbound core lane structure provided by MH.

The static slope stability analyses were performed using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A target factor of safety of 1.3 against deep-seated, global failure that would affect the operation of the highway and bridge is normally used for the design of embankment slopes under static conditions. This factor of safety is considered appropriate for the river valley slopes at this site, considering the design requirements and the field data available.

Effective stress parameters were employed in the static stability analyses assuming drained conditions for the soils. The effective stress parameters (effective friction angle and cohesion) for these soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT) and Atterberg limits, in conjunction with engineering judgement considering experience in similar soil conditions. The soil parameters that have been used in the stability analyses are summarized below.

<i>Soil Type</i>	<i>Bulk Unit Weight</i>	<i>Effective Angle of Friction</i>	<i>Cohesion</i>
Highway 401 embankment fill	19 kN/m ³	31°	0 kPa
Stiff to hard clayey silt to silty clay till	21 kN/m ³	34°	0 kPa
Dense to very dense sand and gravel to sand to sandy silt	20 kN/m ³	36°	0 kPa

6.5.2 Results of Static Stability Analyses

The results of the static slope stability analyses, using the parameters given above and the survey profile provided by MH, indicate that the West Don River valley slopes have a factor of safety of greater than 1.3 against deep-seated slope instability. The results of static slope stability analyses for the west and east river valley slopes are provided on Figures 9 and 10, respectively.

6.6 Liquefaction Potential and Seismic Stability Analyses

6.6.1 Analysis Methods

The liquefaction potential of granular soils under seismic loading is assessed using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary* (2001) based on publications by Seed and Idriss (1971) and Seed et al. (1984), which compares the cyclic resistance ratio (CRR) of the soils to the cyclic stress ratio (CSR) caused by an earthquake. The CRR is determined based on correlations with the normalized penetration resistance and fines content of soil together with the characteristic earthquake magnitude for liquefaction assessment (which is indirectly related to the number of significant stress cycles or duration of strong shaking). The CRR is corrected for earthquake magnitude and overburden stress effects. The CSR at a given depth is related to the peak ground acceleration, the ratio of the total to effective overburden stress at that depth, and soil flexibility. A factor of 0.65 is used to convert the maximum CSR to an equivalent CSR of uniform cycles (Section C4.6.2 of the *CHBDC Commentary*).

In general, geologically young, loose, saturated deposits of sand, silty sand, and non-plastic silt are susceptible to liquefaction.

Liquefaction-Induced Settlements and Lateral Movements

Where liquefaction is identified to be a problem using the methods described above, vertical settlements of the soil under the earthquake loading may occur. The anticipated post-earthquake settlements are estimated using a relationship developed by Tokimatsu and Seed (1987) where the anticipated post-earthquake volume change is related to the SPT “N” values and CSR.

The lateral movements can be estimated using relationships proposed by Makdisi and Seed (1978). If unacceptable lateral movements are anticipated, soil improvement methods should be considered and could include densification, removal and re-compaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled.

Embankment Stability under Seismic Conditions

If liquefaction of the subsoils under an embankment loading is not anticipated, the stability of the embankment slope may be assessed using conventional pseudo-static methods of slope stability analysis under earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate; however, a factor of safety less than 1.0 does not indicate full-scale failure of the embankment slope due to the application of the peak ground acceleration in one direction for a short period of time. In this case, other methods, such as the Newmark sliding block method may be used to assess the magnitude of the ground movement.

Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual shear strength parameters in the liquefied layers using the correlation proposed by Seed and Harder (1990) which is correlated to SPT “N” values. If, under these conditions, the embankment is estimated to have a factor of safety of less than 1.0 under static conditions (i.e., without inertia effects), the embankment is considered to be susceptible to a flow slide (characterized by very large lateral and vertical displacements of the embankment). If under residual strength conditions, the static factor of safety is greater than 1.0, lateral displacements may still occur, and these are estimated using the Newmark method, which relates the horizontal acceleration necessary to induce a factor of safety equal to 1.0 in the embankment (i.e. yield acceleration) to the anticipated displacements. If the yield acceleration is greater than the maximum acceleration for the site, then no remedial measures are required. If the yield acceleration is less than the maximum acceleration and the computed movements are unacceptable, soil improvement methods may be necessary at the site.

6.6.2 Results of Liquefaction and Seismic Stability Analyses

The liquefaction susceptibility of the soil deposits underlying Hogg’s Hollow bridge site and the consequent stability of the Highway 401 embankments and the West Don River valley slopes under seismic loading conditions have been assessed. The peak zonal acceleration used for the Hogg’s Hollow bridge site (Toronto) is 0.07 g, which is based on a zonal acceleration of 0.05 g multiplied by an amplification factor of 1.4 for the types of soils found at the site. This amplification factor was estimated in accordance with Section 4.1.8.4 and Table 4.1.8.4A of the *National Building Code of Canada* (National Research Council Canada, 2005) and *NEHRP* (FEMA, 1994) for Class D soils. Typically, for free-draining soils, the seismic loading is applied to the long-term (drained) conditions.

Using the methods outlined above, the soils at this site have a very low risk of liquefaction. This assessment corresponds to a characteristic earthquake of magnitude 7.0 representing approximately 10 to 15 effective cycles of loading, which has been established based on historical earthquake data and de-aggregation of seismic risk carried out for other projects in the general region, taking into consideration that smaller magnitude events (i.e. less than magnitude 5.0) do not contribute to liquefaction damage.

A factor of safety of greater than 1.0 against embankment instability under seismic conditions is obtained for the Hogg's Hollow bridge approach embankments and river valley slopes. The results of the slope stability analysis under an earthquake-induced peak ground acceleration equal to 0.07g, using the commercially available program SLOPE/W (Version 6.20) produced by Geo-Slope International Ltd. and employing the Morgenstern-Price method of analysis, are shown on Figures 11 and 12 for the west and east slopes of the West Don River valley, respectively.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select free-draining granular fill meeting the specifications of OPSS 1010 Granular A or Granular B Type II (but with less than 5 per cent passing the 200 sieve) should be used as backfill behind the walls. This fill should be placed and compacted in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I, Figure C6.20(a) of the *Commentary on CHBDC*) or within a wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II, Figure C6.20(b) of the *Commentary on CHBDC*).
- For Case I, the pressures are based on the existing and new embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade material for new portions of the approach embankments:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows in accordance with Section C6.9.1 of the *Commentary to CHBDC*:
 - rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
 - horizontal translation of 0.001 times the height of the wall; or
 - a combination of both.

6.7.1 Seismic Considerations

Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6.4 of *CHBDC*, as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure, as given in the following equation and illustrated on Figure 13 following the text of this report.

$$K \gamma' d + (K_{AE} - K) \gamma' (H-d)$$

- where
- K = either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
 - K_{AE} = the seismic active earth pressure coefficient determined in accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*;
 - γ' = the effective unit weight of the soil (kN/m^3), as given in Section 6.5 for the fill materials, and taken as 21 kN/m^3 for the till deposit and its interlayers;
 - d = the investigated depth below the top of the wall (m); and
 - H = the total height of the wall above the underside of footing or toe (m).

Using the amplified zonal acceleration ratio of 0.07g obtained for this site (refer to Section 6.5), the seismic lateral earth pressure coefficients (K_{AE}) for both yielding and non-yielding walls, considering earth and granular fills, were determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary*, and these are presented below. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is essentially flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill	Granular A	Granular B Type II
Yielding wall ¹	0.30	0.27	0.27
Non-yielding wall	0.35	0.31	0.31

¹ The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.07. The vertical component of the earthquake acceleration K_v is taken as $2/3 K_h$.

6.8 Construction Considerations

6.8.1 Open-Cut Excavations

The foundation excavations will extend through existing fill and into the clayey silt till deposit. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill materials are classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

6.8.2 Temporary Excavation Protection

Temporary protection will be required on Highway 401 to facilitate construction of the new abutments and retaining walls. These temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of temporary shoring systems on Highway 401 should meet Performance Level 2 as specified in SP 105S19, provided that any utilities that may be present adjacent to the temporary shoring systems can tolerate this level of deformation.

Temporary protection may also be required adjacent to Piers 1 and 9, depending on the scheme that is adopted for the northward widening of these two foundation elements. If temporary excavation support will be required at these locations, it is recommended that the structural team complete an assessment of the existing structure and foundations to determine which of Performance Levels 1a or 1b, as set out in SP105S19, is appropriate for this structure site.

6.8.3 Groundwater Control for Foundation Excavations

The granular (sand and gravel) fill may be water-bearing, particularly during wet periods of the year, with groundwater "perched" on top of the underlying, less permeable clayey silt till deposit. It is anticipated that the groundwater seepage into the foundation or pile cap excavations can be adequately controlled by pumping from properly filtered sumps.

6.8.4 Subgrade Protection

As discussed in Sections 6.3.1 and 6.4.1, the clayey silt till that will form the subgrade for shallow foundations will be susceptible to softening and degradation on exposure to water and construction traffic. If spread footings are adopted for any of the foundation elements for the structure widening or retaining wall replacements, it is recommended that a 100 mm thick layer of lean mix concrete or mass concrete be placed on the footing subgrade to form a working mat, to protect the subgrade from degradation. This subgrade protection can be illustrated on the General Arrangement and Foundation Layout drawings; alternatively, a Non-Standard Special Provision can be included in the Contract Documents. A sample Non-Standard Special Provision to address subgrade protection is provided in Appendix A.

6.8.5 Ground and Groundwater Control for Caisson Installation

As discussed in Sections 6.3.3 and 6.4.3, running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur where water-bearing cohesionless soils are present at/near the caisson base. If caisson foundations are adopted for support of any of the foundation elements associated with the core lane structure widening and retaining wall replacement, temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; a sample NSSP is included in Appendix A.

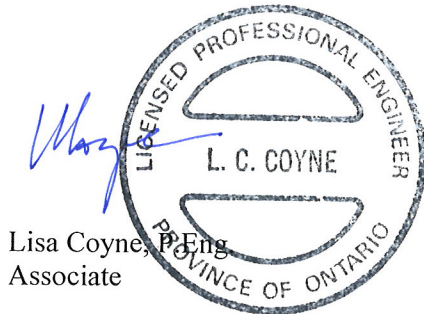
6.8.6 Vibration Monitoring During Pile Installation

Where driven steel H-piles or caissons are adopted for the structure widening, the installation of such deep foundations will cause vibrations. It is recommended that an NSSP be included in the Contract Documents to address vibration monitoring during pile driving or caisson installation; a sample NSSP is included in Appendix A, based on a peak particle velocity of 50 mm/s. However, it is recommended that the structural team assess the tolerable vibration levels for the existing Hogg's Hollow core and collector structures; the maximum peak particle velocity in the NSSP can then be modified accordingly.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate with Golder, with technical input from Mr. Sen Hu, P.Eng., a geotechnical engineer and seismic specialist with Golder, and Mr. Murty Devata, P.Eng., a specialist foundations consultant with Golder. Mr. Jorge Costa, P.Eng., a Principal and Designated MTO Contact for Golder, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.



Lisa Coyne, P.Eng.
Associate



Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Contact

SH/MSD/LCC/JMAC/lcc

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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
HOGG'S HOLLOW CORE BRIDGE STRUCTURES – ABUTMENTS AND PIERS
G.W.P. 5-98-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
Spread/strip footings on generally very stiff to hard clayey silt till	<ul style="list-style-type: none"> • Feasible for support of abutment widening or replacement, and widening of Piers 1 and 9 	<ul style="list-style-type: none"> • Potentially easier and faster construction 	<ul style="list-style-type: none"> • Relatively low geotechnical resistance values • Potential for up to about 15 mm of differential settlement between existing structure and new or widened replacement portions • Significant excavation and temporary excavation support required, particularly for new abutments and widening of Piers 1 and 9 (due to presence of valley slope and adjacent foundation elements) 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques 	<ul style="list-style-type: none"> • Foundation costs less expensive than deep foundations. • However, higher costs due to excavation and temporary excavation support requirements
Steel H-piles driven to found within 100-blow clayey silt till, sand and gravel, sand, or sandy silt	<ul style="list-style-type: none"> • Feasible for support of abutment widening or replacement • Feasible for northward widening of Piers 1 and 9; however, see “Disadvantages” and “Constructability/Practicability” for installation limitations 	<ul style="list-style-type: none"> • Minimize differential settlement between existing and new/widened portions of structure • Higher geotechnical resistance than for shallow foundations • Readily installed for abutment widening or replacement, though interference with existing piles must be considered 	<ul style="list-style-type: none"> • Head room beneath the Highway 401 westbound core and collector structures will limit the use of full-size, conventional pile driving equipment for Pier 1 and 9 sites • Mini-rigs can be operated in 2.1 m to 4.3 m of headroom, but are only able to develop a factored axial geotechnical resistance of up to 600 kN per pile, and can achieve a batter of only 1H:6V • Full-size conventional pile driving rig can operate from existing Highway 401 bridge deck, but it may be difficult to achieve 1H:3V pile batter based on horizontal clearance from the bridge decks, and would require closure of two lanes (or equivalent area) • Piles may “hang up” on boulders in glacially-derived soils 	<ul style="list-style-type: none"> • Conventional construction methods for H-pile foundations for abutment widening or replacement • Special equipment/procedures would be required for pile driving at Piers 1 and 9, whether using mini-rigs below the bridge deck or full-size rigs on the bridge deck; preliminary discussions with piling contractors indicate that both approaches are constructable but the mini-rigs will likely not be able to achieve sufficient capacity, and the full-size rig on the bridge deck may not be able to achieve sufficient batter 	<ul style="list-style-type: none"> • Less expensive than caissons

TABLE 1 (Continued)
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
HOGG'S HOLLOW CORE BRIDGE STRUCTURES – ABUTMENTS AND PIERS
G.W.P. 5-98-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
Caissons bored to found within 100-blow clayey silt till, sand and gravel, sand, or sandy silt	<ul style="list-style-type: none"> Feasible for support of abutment widening or replacement, and widening of Piers 1 and 9 	<ul style="list-style-type: none"> Minimize differential settlement across abutments and between foundation elements Higher bearing resistances Readily installed for abutment widening or replacement, and widening of Piers 1 and 9; discussions with deep foundation contractors have indicated that it will be feasible to operate a caisson rig from the bridge deck for installation at Piers 1 and 9 Where a single caisson is used for support of the northward widening of the piers, no pile cap will be required (i.e. caisson and structural column can be continuous) 	<ul style="list-style-type: none"> Head room beneath the Highway 401 westbound core and collector structures will not allow a full-size, conventional caisson rig to operate from the existing ground surface at Piers 1 and 9 Temporary or permanent liners required due to soil conditions (cohesionless soils that are potentially water-bearing) Potential for interference from cobbles and boulders 	<ul style="list-style-type: none"> As noted under "Advantages", based on discussions with deep foundation contractors, the caisson rig could operate from the existing bridge deck to install caissons for support of the Pier 1 and 9 widenings Careful construction techniques will be required during withdrawal of temporary piles to avoid "necking" of the caissons; alternatively, permanent liners could be used 	<ul style="list-style-type: none"> More expensive than steel H-piles, plus the cost of permanent liners if adopted; however, it is understood that a single 1.2 m diameter caisson would be required for support of the northward pier widenings (compared to a group of battered piles)
Micropiles	<ul style="list-style-type: none"> Feasible for support of northward widening Piers 1 and 9; feasible but not considered necessary for support of abutment widening or replacement where standard foundations are suitable and constructable 	<ul style="list-style-type: none"> Small micropile rig can operate under existing bridge deck; however, this advantage is offset by the fact that conventional foundations can be installed using a full-size pile driving or caisson rig operating from the bridge deck 	<ul style="list-style-type: none"> More expensive to construct than other foundation types, plus the cost of site-specific design 	<ul style="list-style-type: none"> Constructable, but less practical than more economical standard foundations which are also constructable at this site 	<ul style="list-style-type: none"> More expensive than other foundation types

TABLE 2
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
HOGG'S HOLLOW CORE BRIDGE STRUCTURES – RETAINING WALLS
G.W.P. 5-98-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
Spread/strip footings on generally very stiff to hard clayey silt till	<ul style="list-style-type: none"> Feasible for support of retaining wall replacement 	<ul style="list-style-type: none"> Potentially easier and faster construction 	<ul style="list-style-type: none"> Relatively low geotechnical resistance values Significant excavation and temporary excavation support required, including removal of existing retaining wall pile caps and a portion of the existing piles 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> Foundation costs less expensive than deep foundations. However, higher costs due to excavation and temporary excavation support requirements
Steel H-piles driven to found within 100-blow clayey silt till, sand and gravel, sand, or sandy silt	<ul style="list-style-type: none"> Feasible for support of retaining wall replacement 	<ul style="list-style-type: none"> Higher geotechnical resistance than for shallow foundations Readily installed for retaining wall replacements, though interference with existing piles must be considered 	<ul style="list-style-type: none"> Piles may “hang up” on boulders in glacially-derived soils; also potential for interference with existing piles Existing pile cap may require removal, depending on configuration of new pile cap 	<ul style="list-style-type: none"> Conventional construction methods for H-pile foundations for retaining wall replacement 	<ul style="list-style-type: none"> Less expensive than caissons
Caissons bored to found within 100-blow clayey silt till, sand and gravel, sand, or sandy silt	<ul style="list-style-type: none"> Feasible for support of retaining wall replacement 	<ul style="list-style-type: none"> Higher bearing resistances Readily installed for retaining wall replacement 	<ul style="list-style-type: none"> Existing pile cap may require removal, depending on configuration of new pile cap Temporary or permanent liners required due to soil conditions (cohesionless soils that are potentially water-bearing) Potential for interference from cobbles and boulders 	<ul style="list-style-type: none"> Careful construction techniques will be required during withdrawal of temporary piles to avoid “necking” of the caissons; alternatively, permanent liners could be used 	<ul style="list-style-type: none"> More expensive than steel H-piles, plus the cost of permanent liners if adopted

TABLE 2 (Continued)
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
HOGG'S HOLLOW CORE BRIDGE STRUCTURES – RETAINING WALLS
G.W.P. 5-98-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
Soldier pile and concrete panel walls	<ul style="list-style-type: none"> Feasible for construction of retaining wall replacements, though soil anchor installation likely not possible in the limited space between the new retaining walls and the adjacent collector structure wing walls/ retaining walls 	<ul style="list-style-type: none"> This type of wall generally most advantageous in “top-down” construction applications (i.e., as part of a cut widening); less advantageous for embankment widening required on this project 	<ul style="list-style-type: none"> Potential for interference with cobbles and boulders during soldier pile installation Existing pile cap may require removal, depending on soil anchor geometry Soil anchor installation likely not possible as there will be less than 1.5 m of working room between the face of the new retaining wall and that of the existing collector structure wing walls/ retaining walls 	<ul style="list-style-type: none"> Soil anchors are likely not constructable in the limited space available between the new and existing walls; it may be possible to install an anchor at the top of the soldier pile from the deck of the adjacent collector structure, though collector lane closures would be required during installation, grouting and anchor stressing 	<ul style="list-style-type: none"> Less expensive than caisson-supported gravity wall
RSS walls	<ul style="list-style-type: none"> Feasible for replacement of retaining walls in northwest, southwest, northeast and southeast quadrants of core structures 	<ul style="list-style-type: none"> Conventional construction methods Ease of construction compared to cast-in-place wall; less vertical excavation when working within the relatively limited space between the core and collector bridge structures Existing pile caps/footings for existing retaining walls can remain in place below reinforced soil mass following removal of existing retaining wall stems 	<ul style="list-style-type: none"> None 	<ul style="list-style-type: none"> Conventional construction methods; constructible within limited working space between existing core and collector bridge structures 	<ul style="list-style-type: none"> Less expensive than all other foundation/ retaining wall options

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Consistency

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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-1			1 OF 1 METRIC														
W.P. 5-98-00			LOCATION N 4845882.4 ; E 312074.8			ORIGINATED BY SB														
DIST Central HWY 401			BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Manual Hammer			COMPILED BY MK														
DATUM Geodetic			DATE April 25, 2007			CHECKED BY LCC														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR	SA	SI	CL
171.6	GROUND SURFACE							20	40	60	80	100								
0.0	Asphalt																			
171.1	Sand and gravel (FILL)																			
0.8	Sand, some gravel (FILL)						171													
	CLAYEY SILT with sand, trace to some gravel (TILL) Stiff to hard Brown to grey-brown Moist		1	SS	22		170													
			2	SS	12															
			3	SS	26		169										5	33	43	19
			4	SS	55		168													
			5	SS	92															
			6	SS	75/0.15		167										2	40	38	18
165.5							166													
6.1	Sandy SILT, trace clay and gravel (TILL) Very dense Brown Moist to wet		7	SS	73/0.15		165													
164.3																				
7.3	CLAYEY SILT, trace sand and gravel (TILL) Hard Grey Wet		8	SS	73		164													
							163													
			9	SS	43		162													
							161													
			10	SS	30		160													
159.7																				
11.9	Sandy SILT, trace gravel and clay Very dense Grey Moist		11	SS	67/0.15															
159.1																	6	28	57	9
12.5	END OF BOREHOLE																			
NOTES:																				
1. Water level measured in open borehole at a depth of 9.6m (Elevation 162.0m) upon completion of drilling.																				

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-2			1 OF 1 METRIC															
W.P. 5-98-00			LOCATION N 4845523.4 ; E 311722.8			ORIGINATED BY SB															
DIST Central HWY 401			BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Manual Hammer			COMPILED BY MK															
DATUM Geodetic			DATE April 26, 2007			CHECKED BY LCC															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR	SA	SI	CL	
168.5	GROUND SURFACE							20	40	60	80	100									
0.0	Asphalt																				
0.2	Sand and gravel (FILL)																				
167.7							168														
0.8	SAND, some silt, trace to some gravel (FILL) Compact Brown Moist		1	SS	17													11	71	13	5
167.0							167														
1.5	CLAYEY SILT with sand, trace gravel (TILL) Hard Brown Moist		2	SS	62																
			3	SS	82		166											6	49	33	12
			4	SS	67/0.15		165														
			5	SS	82		164														
164.2							163														
4.3	Gravelly SAND, some silt Very dense Brown Moist		6	SS	96		162														
			7	SS	75/0.15		161											28	55	14	3
							160														
161.2							159														
7.3	SAND and GRAVEL, some silt Very dense Brown Moist		8	SS	105		158														
			9	SS	105		157														
			10	SS	122																
156.3																					
12.3	CLAYEY SILT, some sand, trace gravel (TILL) Hard Brown Moist END OF BOREHOLE		11	SS	95/0.15																
	NOTES: 1. Open borehole dry upon completion of drilling.																				

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ

PROJECT <u>06-1111-060</u>		RECORD OF BOREHOLE No 07-3		1 OF 2 METRIC	
W.P. <u>5-98-00</u>		LOCATION <u>N 4845828.9 ; E 312064.6</u>		ORIGINATED BY <u>GPD</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Automatic Hammer</u>		COMPILED BY <u>MK</u>	
DATUM <u>Geodetic</u>		DATE <u>April 26, 2007</u>		CHECKED BY <u>LCC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								20 40 60 80 100	20 40 60 80 100	W _P	W	W _L			
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
171.3	GROUND SURFACE														
0.0	Asphalt														
0.2	Concrete														
170.5	Sand and gravel (FILL)														
0.8	Clayey silt, trace sand and gravel (FILL) Firm Brown Moist		1	SS	7										
169.8	CLAYEY SILT with sand to some sand, trace to some gravel (TILL) Very stiff to hard Brown Moist		2	SS	22										
1.5			3	SS	19										
			4	SS	15										
			5	SS	21										
			6	SS	46										
			7	SS	37										
			8	SS	73										
			9	SS	35										
			10	SS	19										
159.1	SAND and SILT, trace clay and gravel Very dense Brown Moist		11	SS	117										
12.2															
157.6	CLAYEY SILT, some sand, trace gravel (TILL) Hard Brown Moist		12	SS	62/0.15										
13.7															

Continued Next Page

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

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ



PROJECT		RECORD OF BOREHOLE No 07-3				2 OF 2 METRIC									
W.P.		LOCATION				ORIGINATED BY									
DIST		BOREHOLE TYPE				COMPILED BY									
DATUM		DATE				CHECKED BY									
06-1111-060		N 4845828.9 ; E 312064.6				GPD									
5-98-00		Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Automatic Hammer				MK									
Geodetic		April 26, 2007				LCC									
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							
--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100				
156.1															
15.2	SILT, trace clay		13	SS	113										
155.6	Very dense														
15.7	Grey Moist														
	END OF BOREHOLE														
NOTES: 1. Water level measured in open borehole at a depth of 14.6m (Elevation 156.7m) upon completion of drilling.															

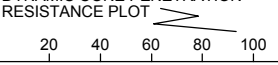

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-4			1 OF 2 METRIC																											
W.P. 5-98-00			LOCATION N 4845839.5 ; E 312028.7			ORIGINATED BY GPD																											
DIST Central HWY 401			BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Automatic Hammer			COMPILED BY MK																											
DATUM Geodetic			DATE April 27, 2007			CHECKED BY LCC																											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION	20	40	60	80	100	W _p	W	W _L	UNCONFINED	FIELD VANE	QUICK TRIAXIAL	REMOULDED	20	40	60	80	100	10	20	30	γ	GR	SA	SI	CL
171.0		GROUND SURFACE																															
0.0		Asphalt																															
170.6		Concrete																															
170.2		Sand and gravel, trace silt (FILL) Brown																															
0.8		Clayey silt with sand to some sand, trace gravel (FILL) Very stiff Brown Moist		1	SS	17		170																									
				2	SS	27		169																									
				3	SS	25		168																									
168.0								168																									
3.1		CLAYEY SILT with sand to some sand, trace gravel (TILL) Hard to very stiff Brown, becoming grey below a depth of 6.1 m Moist		4	SS	35		167																									
				5	SS	41		166																									
				6	SS	48		165																									
				7	SS	35		164																									
				8	SS	41		163																									
				9	SS	23		162																									
				10	SS	20		161																									
158.8								160																									
12.2		SAND and SILT, trace gravel and clay Very dense Brown Moist		11	SS	87		159																									
				12	SS	83		158																									
								157																									

Continued Next Page

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

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ

PROJECT <u>06-1111-060</u>			RECORD OF BOREHOLE No 07-4				2 OF 2 METRIC				
W.P. <u>5-98-00</u>			LOCATION <u>N 4845839.5 ; E 312028.7</u>				ORIGINATED BY <u>GPD</u>				
DIST <u>Central</u> HWY <u>401</u>			BOREHOLE TYPE <u>Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Automatic Hammer</u>				COMPILED BY <u>MK</u>				
DATUM <u>Geodetic</u>			DATE <u>April 27, 2007</u>				CHECKED BY <u>LCC</u>				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and SILT, trace gravel and clay Very dense Brown Moist		13	SS	102		155				
			14	SS	114		154				1 34 59 6
							153				
152.4	Becoming grey at 18.3 m depth		15	SS	65/0.15						
18.6	END OF BOREHOLE NOTES: 1. Open borehole dry upon completion of drilling.										

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-5			1 OF 2 METRIC																
W.P. 5-98-00			LOCATION N 4845582.7 ; E 311740.6			ORIGINATED BY SB																
DIST Central HWY 401			BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Manual Hammer			COMPILED BY MK																
DATUM Geodetic			DATE April 27, 2007			CHECKED BY LCC																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ			GR SA SI CL			
169.1	GROUND SURFACE							20 40 60 80 100														
0.0	Asphalt						169															
168.6	Concrete						168															
0.5	Sand and gravel, trace silt (FILL) Loose to dense Brown Moist		1	SS	6		167															
			2	SS	33		166															
			3	SS	27		165															
165.8	CLAYEY SILT with sand, trace to some gravel (TILL) Very stiff to hard Brown Moist		4	SS	31		164															
3.4			5	SS	23		163															
			6	SS	22		162															
			7	SS	21		161															
161.2	Gravelly SAND, some silt, trace clay Dense to very dense Brown Moist		8	SS	39		160															
7.9			9	SS	82		159															
			10	SS	44		158															
157.5	SAND, some silt, trace gravel Very dense Brown Moist		11	SS	80/0.15		157															
11.6							156															
155.4	CLAYEY SILT with sand, trace gravel (TILL) Hard Brown to grey Moist		12	SS	76/0.15		155															
13.7																						

Continued Next Page

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

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ



PROJECT		RECORD OF BOREHOLE No 07-5				2 OF 2 METRIC										
W.P. 06-1111-060		LOCATION N 4845582.7 ; E 311740.6				ORIGINATED BY SB										
DIST Central HWY 401		BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Manual Hammer				COMPILED BY MK										
DATUM Geodetic		DATE April 27, 2007				CHECKED BY LCC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
153.5			13	SS	110/0.20											
15.6	END OF BOREHOLE															
	NOTES: 1. Open borehole dry upon completion of drilling.															

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

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-7			1 OF 1 METRIC														
W.P. 5-98-00			LOCATION N 4845813.6 ; E 312047.5			ORIGINATED BY GPD														
DIST Central HWY 401			BOREHOLE TYPE Truck-Mount D-90, 108 mm Diameter Solid Stem Augers, Automatic Hammer			COMPILED BY MK														
DATUM Geodetic			DATE April 29, 2007			CHECKED BY LCC														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR	SA	SI	CL
171.0	GROUND SURFACE							20 40 60 80 100												
0.0	Asphalt																			
170.5	Concrete																			
0.5	Gravelly sand, trace silt (FILL) Compact Brown Moist		1	SS	14		170													28 62 7 3
169.5																				
1.5	Clayey silt with sand, trace gravel (FILL) Stiff to very stiff Brown Moist		2	SS	13		169													
			3	SS	15		168													1 32 47 20
			4	SS	19		167													
167.2	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Brown Moist		5	SS	32		166													
3.8			6	SS	61		165													
	Becoming grey at 6.1 m depth		7	SS	19		164													1 22 55 22
			8	SS	21		163													
							162													
161.3	Wet sand seams encountered below 9.4 m depth		9	SS	24															
9.8	END OF BOREHOLE																			
	NOTES: 1. Open borehole dry upon completion of drilling.																			

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ

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

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-9			1 OF 2 METRIC																				
W.P. 5-98-00			LOCATION N 4845857.2 ; E 312048.4			ORIGINATED BY GPD																				
DIST Central HWY 401			BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Manual Hammer			COMPILED BY MK																				
DATUM Geodetic			DATE June 5, 2007			CHECKED BY LCC																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL									
171.1	GROUND SURFACE						171																			
0.0	Asphalt						170																			
0.3	Concrete																									
	Sand and gravel, trace silt (FILL)		1	SS	24																					
	Compact Brown Moist																									
169.6							169																			
1.5	Clayey silt, some sand, trace gravel, containing sand layers (FILL)		2	SS	11																					
	Stiff to very stiff Brown Moist		3	SS	22																					
			4	SS	23																					
							167																			
166.5							166																			
4.6	CLAYEY SILT with sand to some sand, trace gravel (TILL)		5	SS	50/0.08																					
	Very stiff to hard Brown Moist						165																			
			6	SS	81																					
							164																			
			7	SS	95																					
							163																			
							162																			
	Becoming grey and wet at 9.1m depth.		8	SS	62																					
							161																			
			9	SS	24																					
							160																			
							159																			
158.9							158																			
12.2	Silty SAND to Sandy SILT, trace gravel		10	SS	70/0.15																					
	Very dense Brown Moist						157																			
			11	SS	84/0.15																					

Continued Next Page

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

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ



PROJECT		RECORD OF BOREHOLE No 07-9				2 OF 2 METRIC										
W.P. 06-1111-060		LOCATION N 4845857.2 ; E 312048.4				ORIGINATED BY GPD										
DIST Central HWY 401		BOREHOLE TYPE Truck-Mount D-25, 108 mm Diameter Solid Stem Augers, Manual Hammer				COMPILED BY MK										
DATUM Geodetic		DATE June 5, 2007				CHECKED BY LCC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
155.7			12	SS	82/0.15		156									
15.4	END OF BOREHOLE															
	NOTES: 1. Water level measured in open borehole at a depth of 11.9m (Elevation 159.2m) upon completion of drilling.															

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-12			1 OF 2 METRIC											
W.P. 5-98-00			LOCATION N 4845830.7 ; E 312009.1			ORIGINATED BY SB											
DIST Central HWY 401			BOREHOLE TYPE Track-Mount CME-55, 108 mm Diameter Solid Stem Augers, Automatic Hammer			COMPILED BY MWK											
DATUM Geodetic			DATE June 19, 2007			CHECKED BY LCC											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p	W	W _L	γ	GR SA SI CL
162.0	GROUND SURFACE																
0.0	Sand and gravel, containing concrete rubble (FILL) Very dense Brown Moist		1	SS	79		161										
160.5							160										
1.5	CLAYEY SILT with sand, trace gravel (TILL) Stiff to hard Brown and grey Moist		2	SS	12												
			3	SS	11		159										7 36 40 17
			4	SS	17												
			5	SS	55		158										
			6	SS	61		157										
							156										
			7	SS	64		155										5 32 49 14
							154										
			8	SS	58												
153.2							153										
8.8	SAND and SILT Very dense Brown Moist		9	SS	104/0.25		152										0 68 30 2
							151										
			10	SS	55												
149.8							150										
12.2	CLAYEY SILT, trace sand and gravel (TILL) Hard Brown, becoming grey below at depth of 13.7 m Moist		11	SS	43		149										
							148										
			12	SS	57												

Continued Next Page

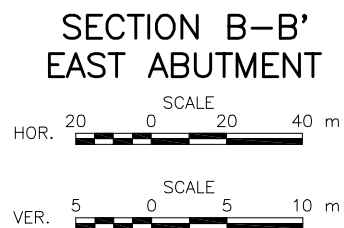
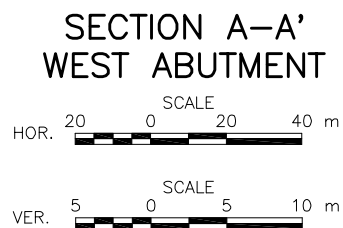
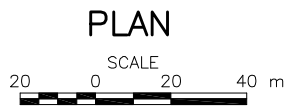
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ

PROJECT <u>06-1111-060</u>			RECORD OF BOREHOLE No 07-12			2 OF 2 METRIC														
W.P. <u>5-98-00</u>			LOCATION <u>N 4845830.7 ; E 312009.1</u>			ORIGINATED BY <u>SB</u>														
DIST <u>Central</u> HWY <u>401</u>			BOREHOLE TYPE <u>Track-Mount CME-55, 108 mm Diameter Solid Stem Augers, Automatic Hammer</u>			COMPILED BY <u>MWK</u>														
DATUM <u>Geodetic</u>			DATE <u>June 19, 2007</u>			CHECKED BY <u>LCC</u>														
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa												
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>													
	CLAYEY SILT, trace sand and gravel (TILL) Hard Brown, becoming grey below at depth of 13.7 m Moist		13	SS	114												0 9 76 15			
			14	SS	100/0.10															
143.4			15	SS	92/0.15															
18.6	END OF BOREHOLE NOTES: 1. Open borehole dry upon completion of drilling.																			

PROJECT 06-1111-060			RECORD OF BOREHOLE No 07-13			1 OF 1 METRIC		
W.P. 5-98-00			LOCATION N 4845597.3 ; E 311748.0			ORIGINATED BY SB		
DIST Central HWY 401			BOREHOLE TYPE Track-Mount D-50, 108 mm Diameter Solid Stem Augers, Automatic Hammer			COMPILED BY MK		
DATUM Geodetic			DATE June 26, 2007			CHECKED BY LCC		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED
161.5	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	Sandy SILT, trace gravel and clay Loose to compact Brown Moist		1	SS	13		161	
			2	SS	9		160	
159.2	Silty SAND, trace to some gravel, trace clay Loose Brown Moist		3	SS	6		159	
2.3			4	SS	4		158	
			5	SS	8		157	
156.9	CLAYEY SILT, trace to some sand, trace gravel (TILL) Very stiff to hard Brown Moist		6	SS	21		156	
4.6			7	SS	42		155	
			8	SS	52		154	
			9	SS	70		152	
			10	SS	110		151	
			11	SS	131		149	
147.5	END OF BOREHOLE		12	SS	104/0.15		148	
14.0	NOTES: 1. Open borehole dry upon completion of drilling.							

MIS-MTO 001 061111060.GPJ GAL-MISS.GDT 11/8/07 DD/RJ



CONT No.
WP No. 5-98-00

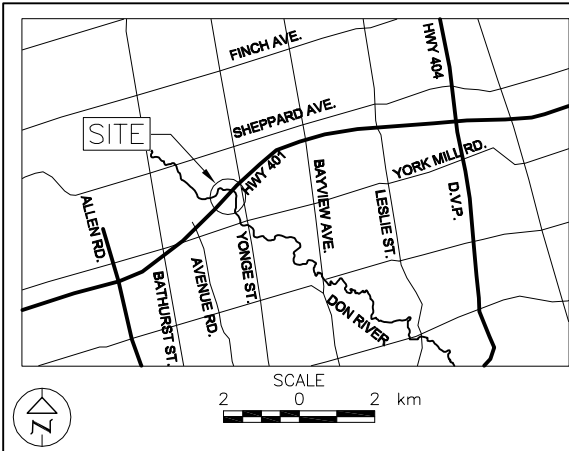


HIGHWAY 401
Hogg's Hollow Bridge and Retaining Walls
BOREHOLE LOCATIONS
AND SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
07-1	171.6	4845882.4	312074.8
07-2	168.5	4845523.4	311722.8
07-3	171.3	4845828.9	312064.6
07-4	171.0	4845839.5	312028.7
07-5	169.1	4845582.7	311740.6
07-6	169.0	4845555.9	311758.7
07-7	171.0	4845813.6	312047.5
07-8	169.5	4845548.6	311702.2
07-9	171.1	4845857.2	312048.4
07-12	162.0	4845830.7	312009.1
07-13	161.5	4845597.3	311748.0

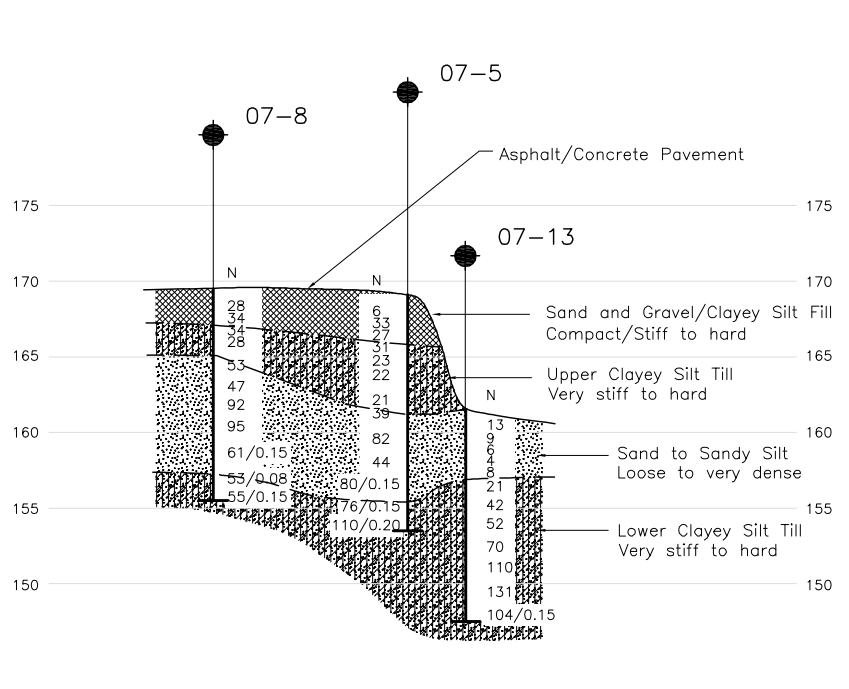
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

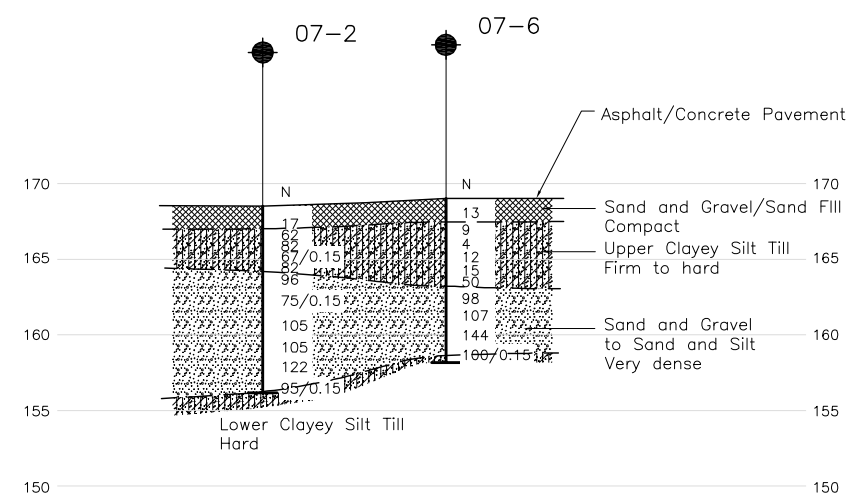
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

Base plans provided in digital format by Morrison Hershfield Limited, received Aug. 09, 2007.

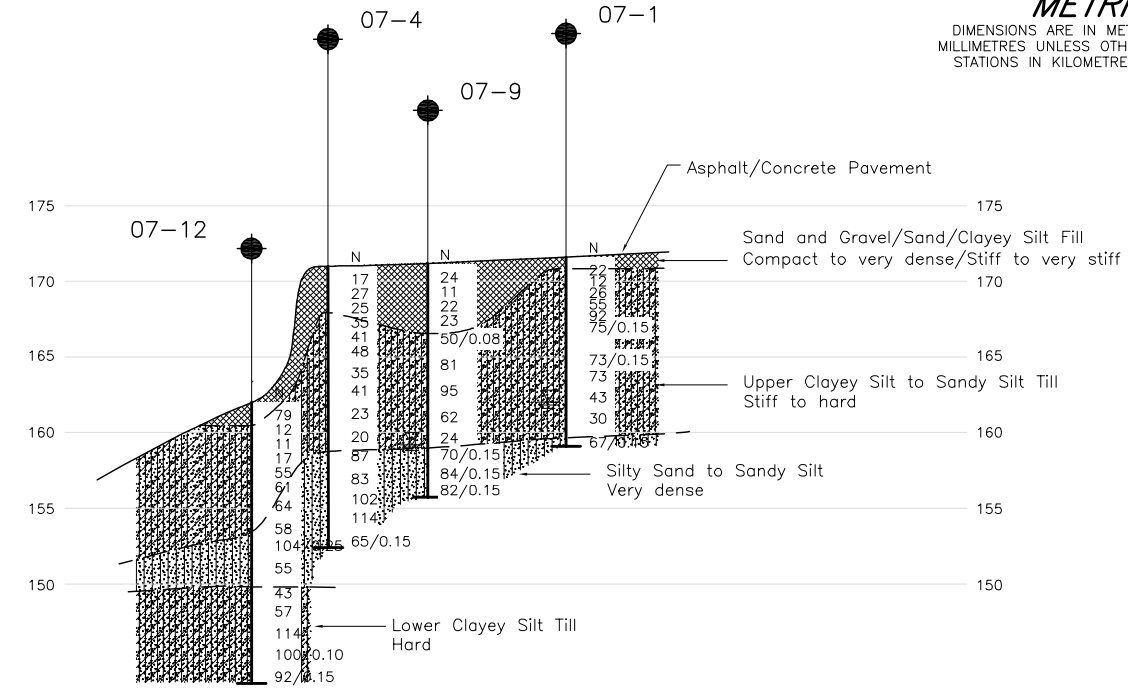
NO.	DATE	BY	REVISION		
Geocres No. 30M11-224					
HWY. 401		PROJECT NO. 06-1111-060-1		DIST.	
SUBM'D. PKS	CHKD. LCC		DATE: 01/09/08		SITE:
DRAWN: RJ/DD	CHKD. PKS		APPD. LCC		DWG. 1



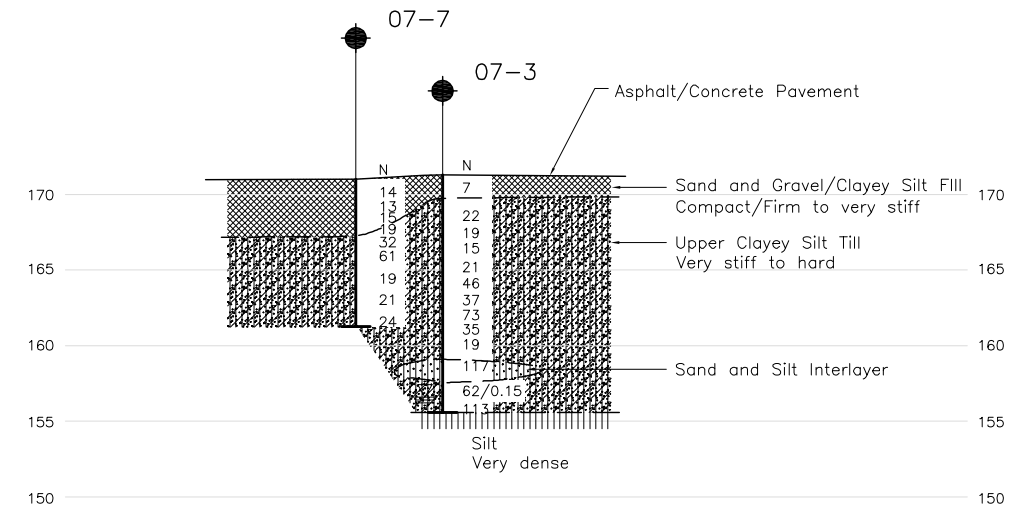
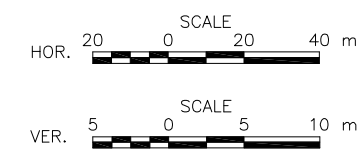
SECTION C-C'
NORTHWEST RETAINING WALL
AND PIER 1



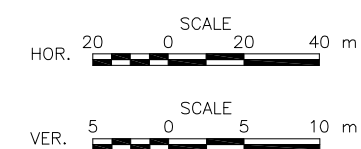
SECTION D-D'
SOUTHWEST RETAINING WALL



SECTION E-E'
NORTHEAST RETAINING WALL
AND PIER 9



SECTION F-F'
SOUTHEAST RETAINING WALL



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

CONT No.
WP No. 5-98-00

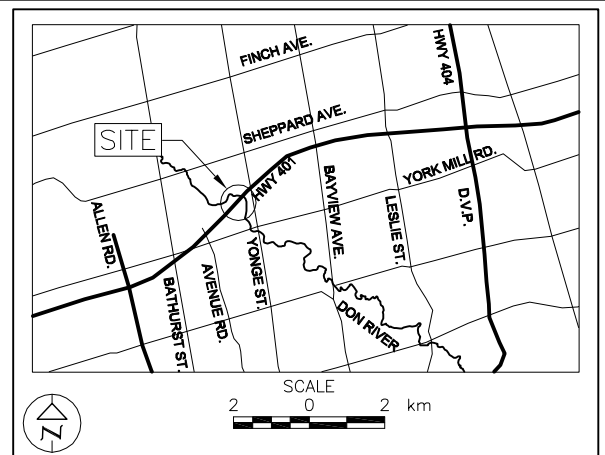
HIGHWAY 401
Hogg's Hollow Bridge and Retaining Walls
SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole - Current Investigation		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL upon completion of drilling		

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
07-1	171.6	4845882.4	312074.8
07-2	168.5	4845523.4	311722.8
07-3	171.3	4845828.9	312064.6
07-4	171.0	4845839.5	312028.7
07-5	169.1	4845582.7	311740.6
07-6	169.0	4845555.9	311758.7
07-7	171.0	4845813.6	312047.5
07-8	169.5	4845548.6	311702.2
07-9	171.1	4845857.2	312048.4
07-12	162.0	4845830.7	312009.1
07-13	161.5	4845597.3	311748.0

NOTES

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

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

REFERENCE

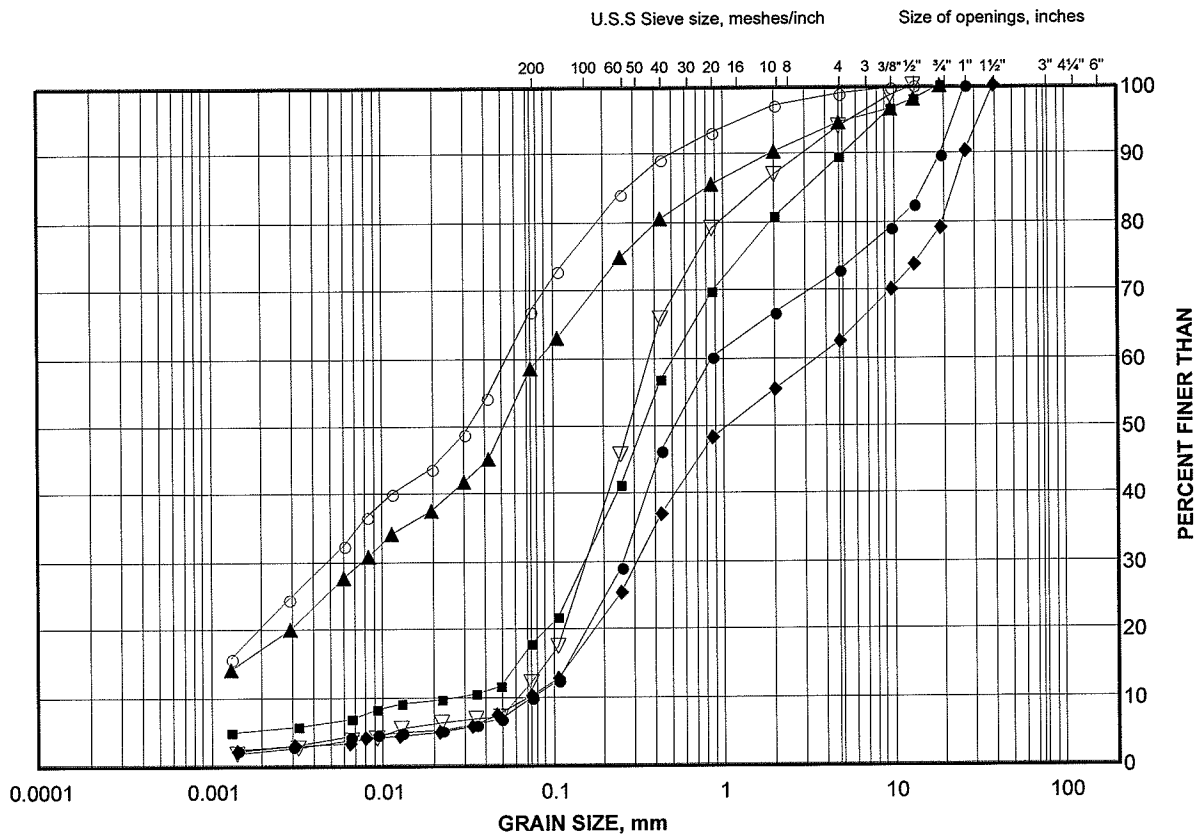
Base plans provided in digital format by Morrison Hershfield Limited, received Aug. 09, 2007.

NO.	DATE	BY	REVISION
Geocres No. 30M11-224			
HWY. 401	PROJECT NO. 06-1111-060-1		DIST.
SUBM'D. PKS	CHKD. LCC	DATE: 01/09/08	SITE:
DRAWN: RJ/DD	CHKD. PKS	APPD. LCC	DWG. 2

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel Fill and Clayey Silt Fill

FIGURE 1



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

LEGEND

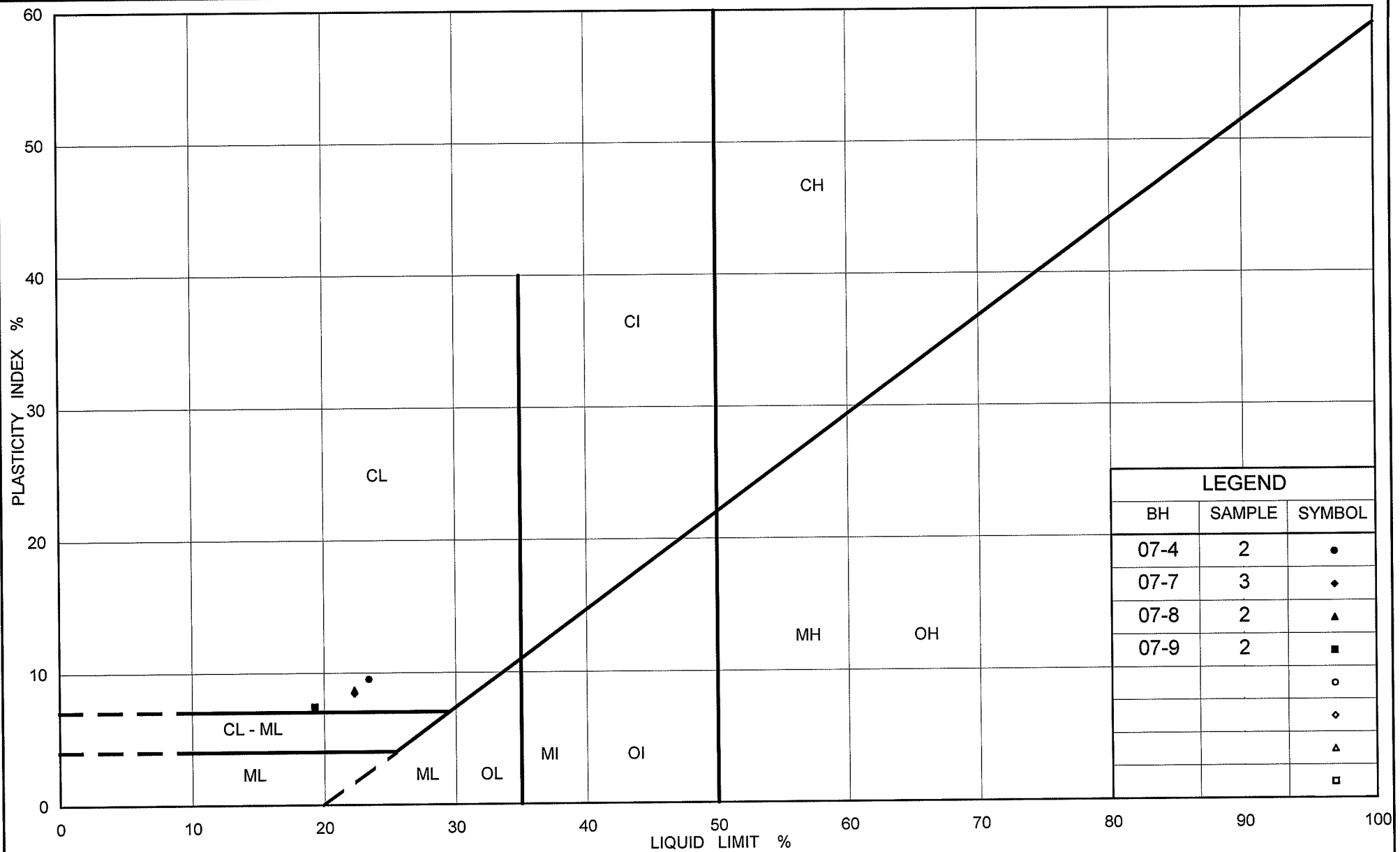
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-7	1	169.9
■	07-2	1	167.5
◆	07-5	2	167.3
▲	07-4	2	169.2
▽	07-9	3	168.5
○	07-7	3	168.4

Project Number: 06-1111-060-1

Checked By: *[Signature]*

Golder Associates

Date: 09-Jan-08



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt Fill

Figure No. 2

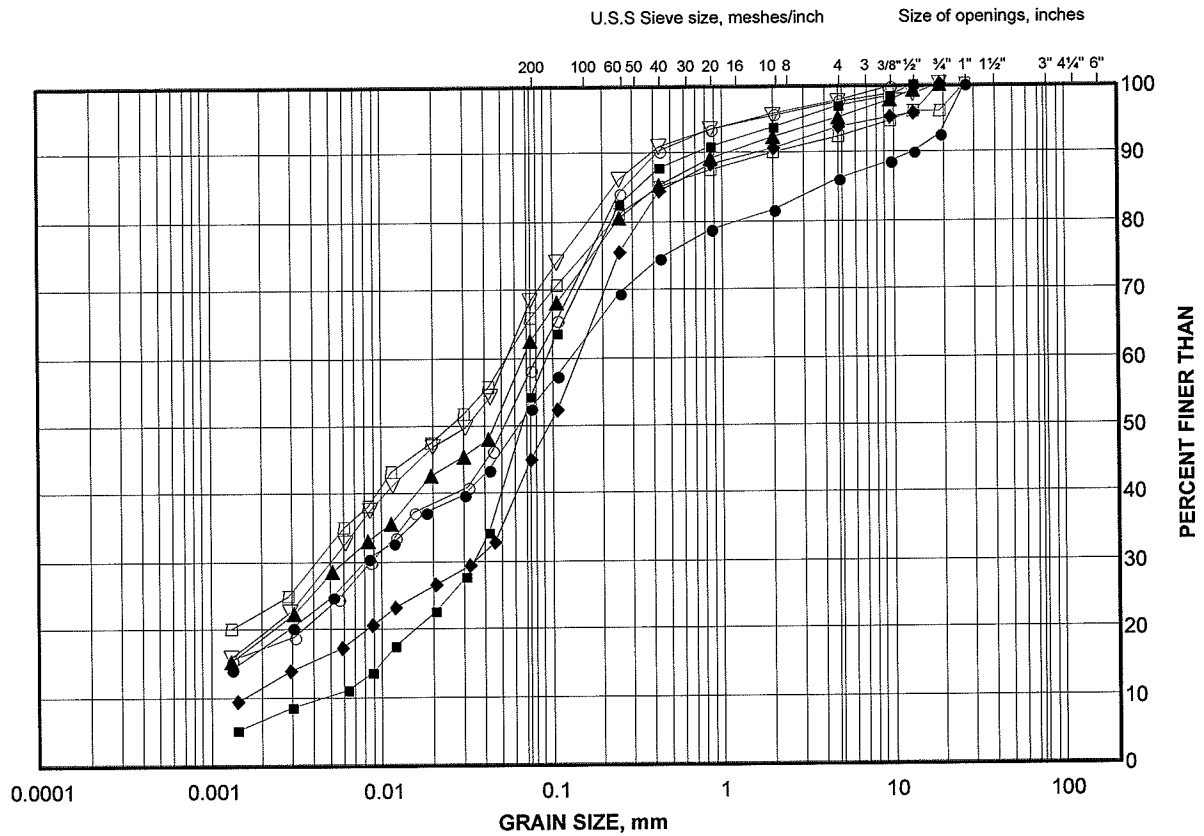
Project No. 06-1111-060-1

Checked By: *Maye*

GRAIN SIZE DISTRIBUTION TEST RESULTS

Upper Clayey Silt Till (Including Interlayers)

FIGURE 3A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-3	10	160.3
■	07-3	11	159.0
◆	07-2	3	165.9
▲	07-1	3	169.0
▽	07-3	6	166.4
○	07-1	6	166.9
□	07-4	8	163.1

Project Number: 06-1111-060-1

Checked By: *W. Boyce*

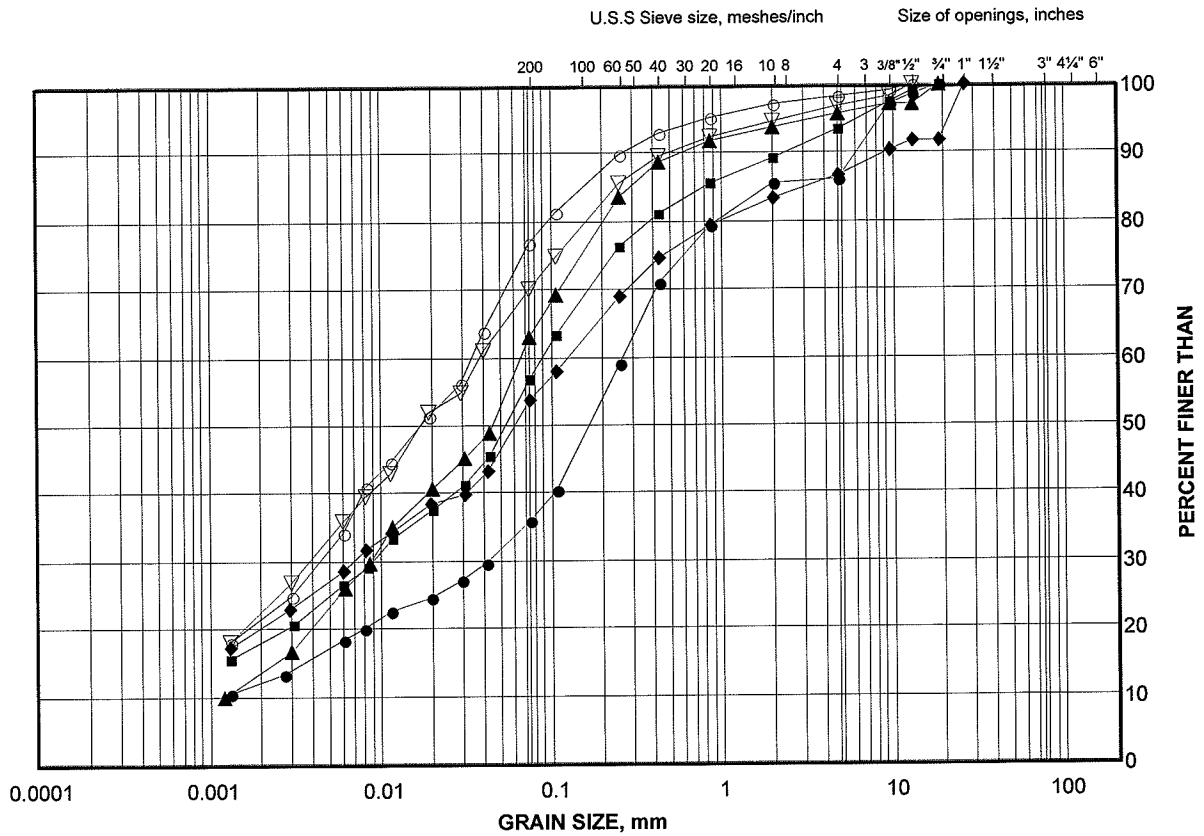
Golder Associates

Date: 13-Nov-07

GRAIN SIZE DISTRIBUTION TEST RESULTS

Upper Clayey Silt Till (Including Interlayers)

FIGURE 3B



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

LEGEND

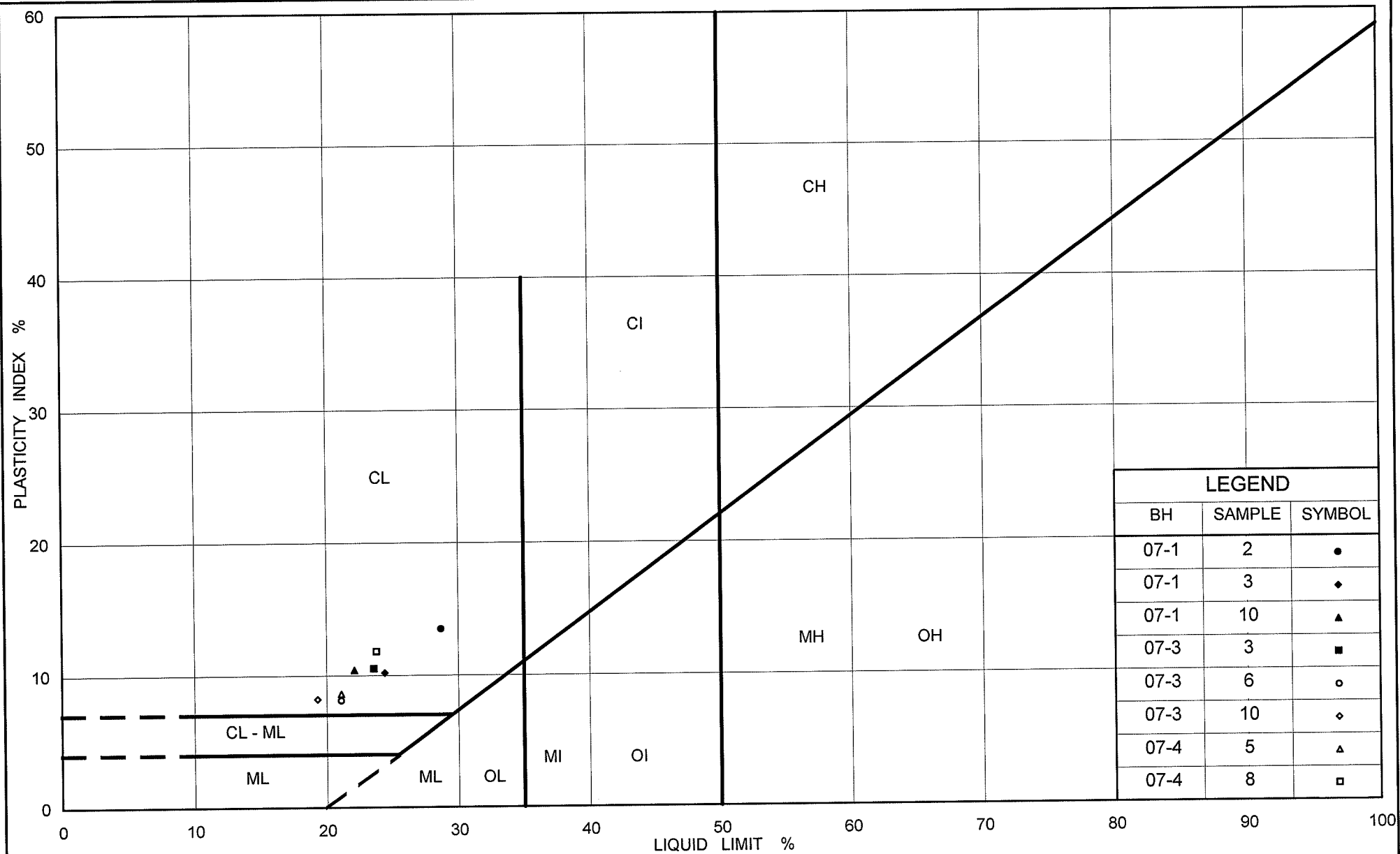
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-6	2	167.2
■	07-12	3	159.4
◆	07-5	6	164.2
▲	07-12	7	155.6
▽	07-9	7	163.3
○	07-7	7	164.6

Project Number: 06-1111-060-1

Checked By: *Moyle*

Golder Associates

Date: 13-Nov-07



Ministry of Transportation

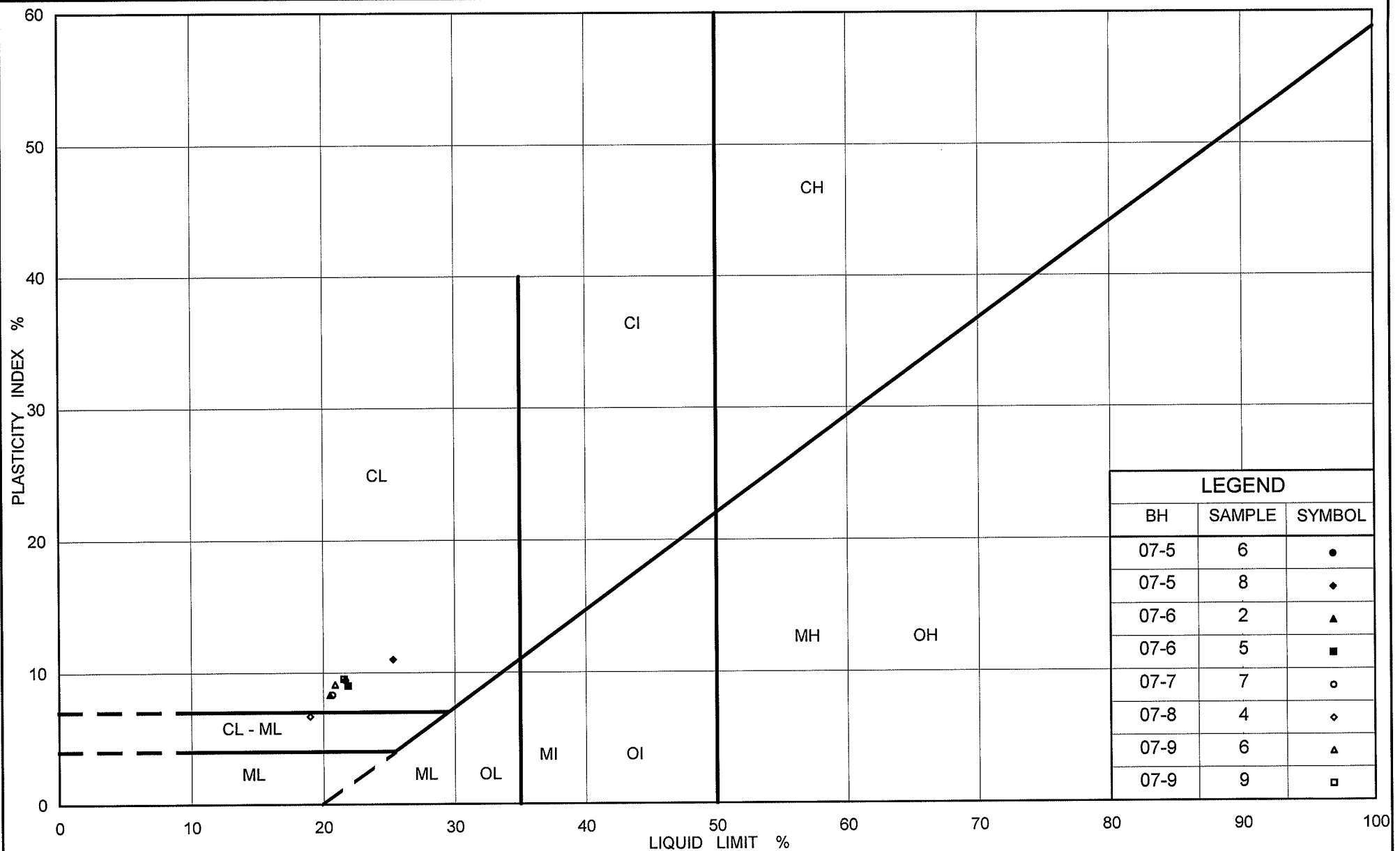
Ontario

PLASTICITY CHART Upper Clayey Silt Till

Figure No. 4A

Project No.06-1111-060-1

Checked By: *W. H. H. H.*



Ministry of Transportation

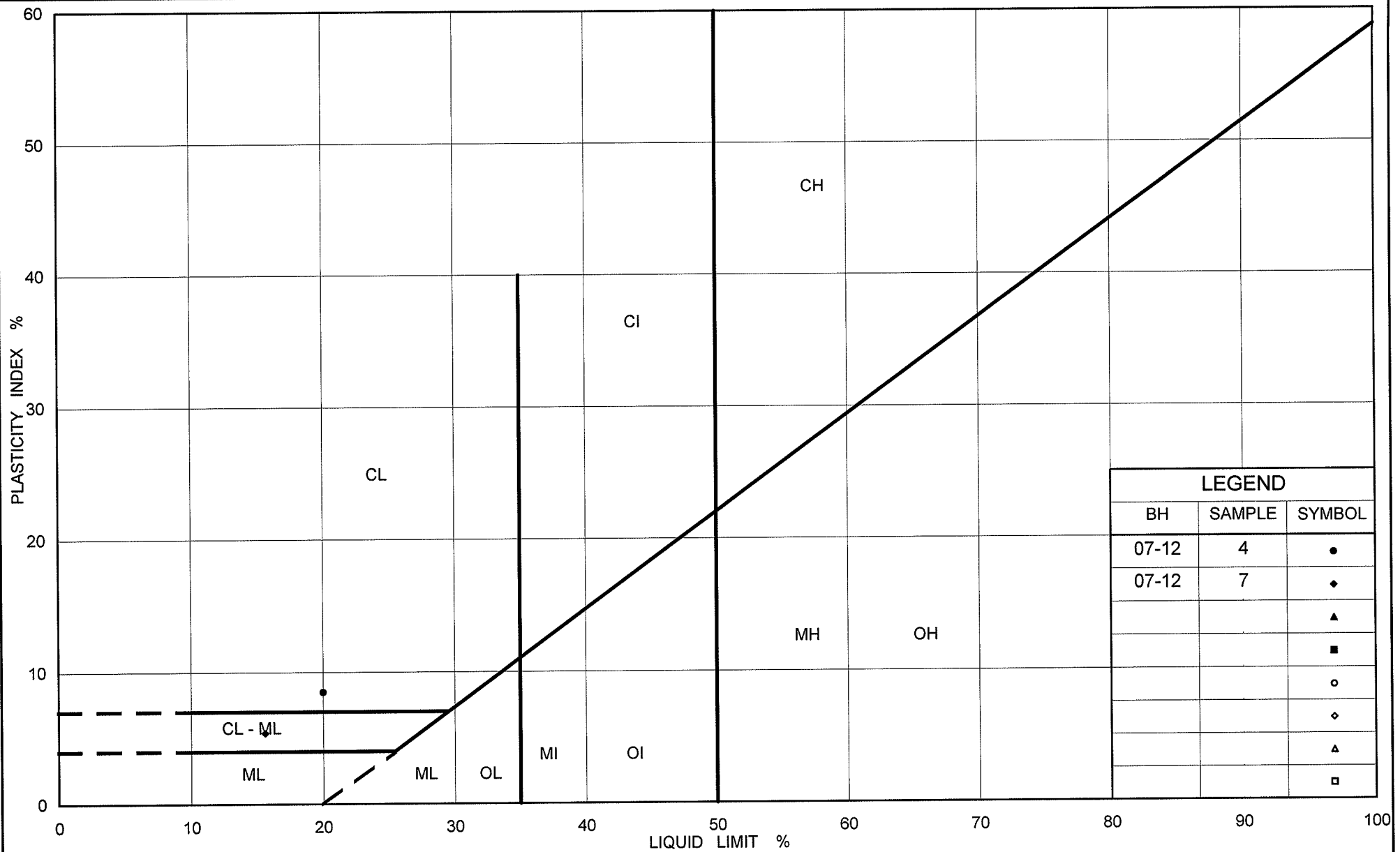
Ontario

PLASTICITY CHART Upper Clayey Silt Till

Figure No. 4B

Project No.06-1111-060-1

Checked By: *W. J. J.*



Ministry of Transportation

Ontario

PLASTICITY CHART Upper Clayey Silt Till

Figure No. 4C

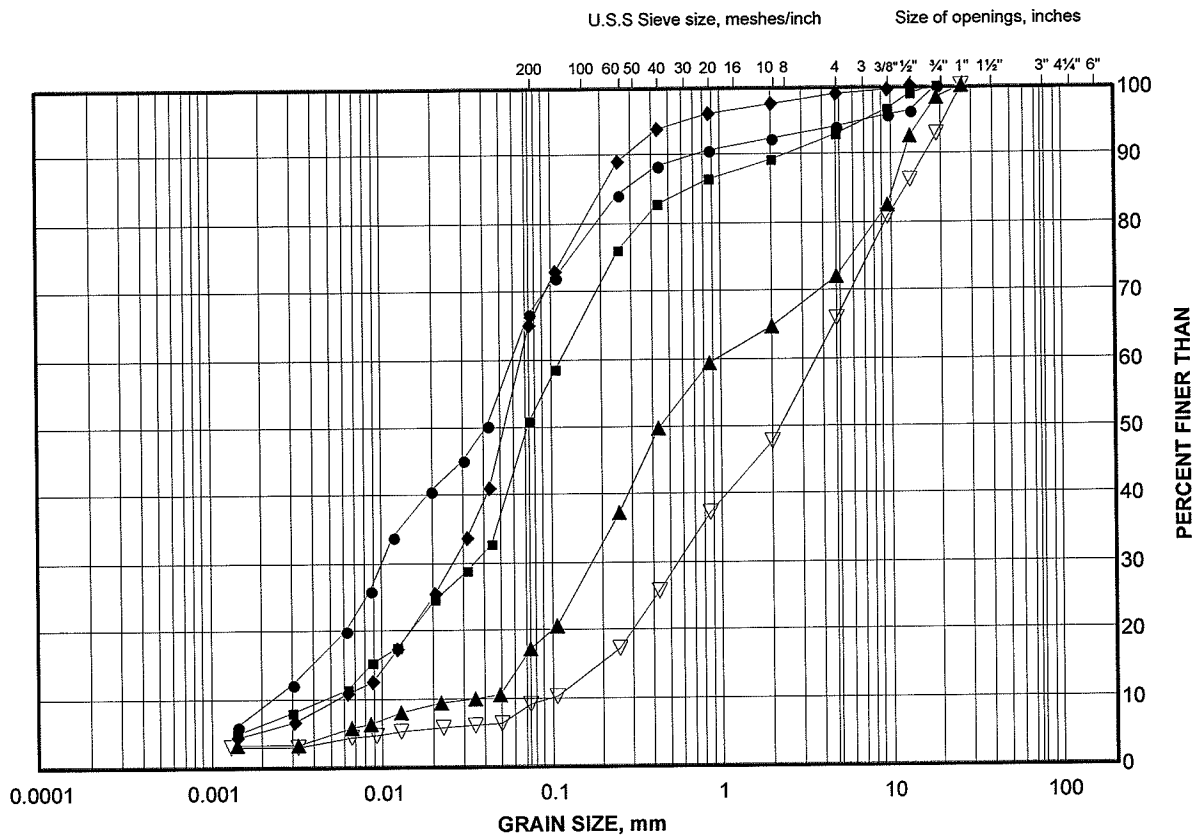
Project No.06-1111-060-1

Checked By: *W. H. H. H.*

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Sandy Silt

FIGURE 5A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-1	11	159.3
■	07-4	12	157.1
◆	07-4	14	154.0
▲	07-2	7	162.3
▽	07-2	9	159.2

Project Number: 06-1111-060-1

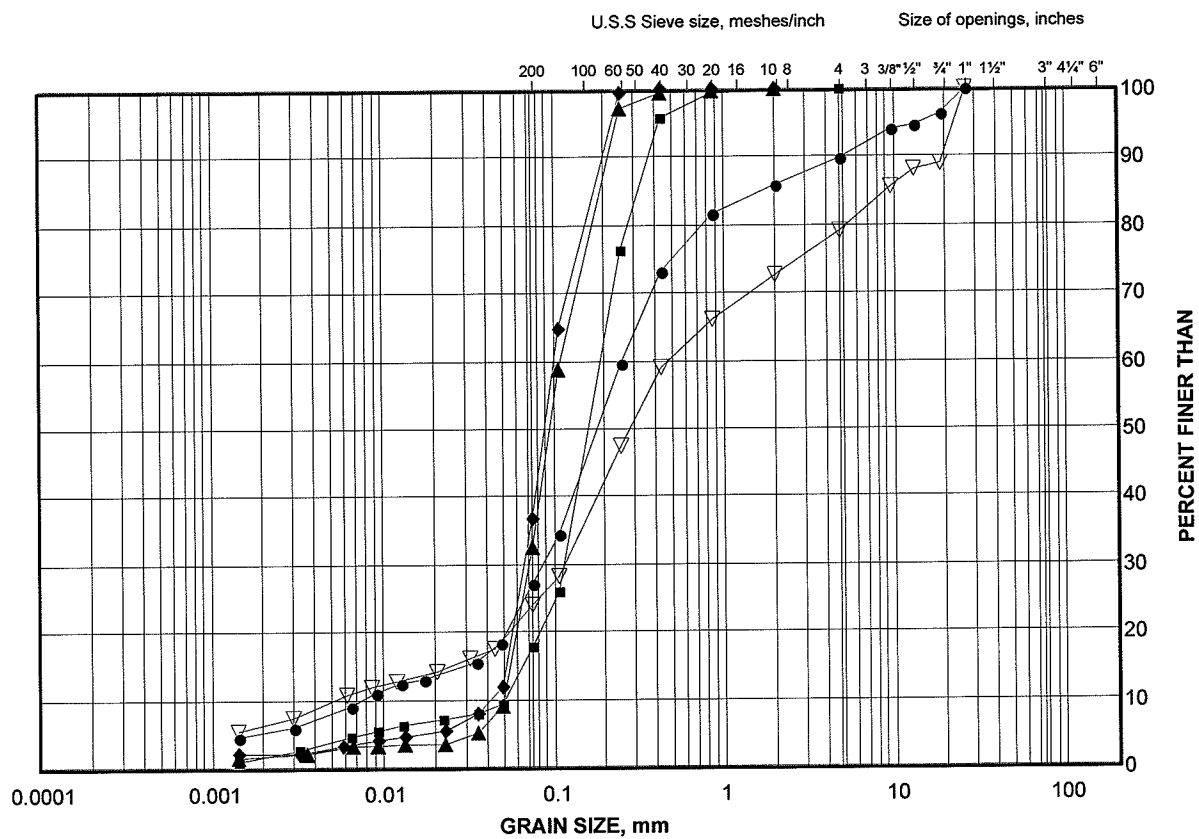
Checked By: *Uloye*

Golder Associates

Date: 12-Nov-07

Sand and Gravel to Sandy Silt

FIGURE 5B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-13	3	158.9
■	07-8	7	161.6
◆	07-6	7	162.7
▲	07-12	9	152.6
▽	07-5	9	159.7

Project Number: 06-1111-060-1

Checked By: Chavez

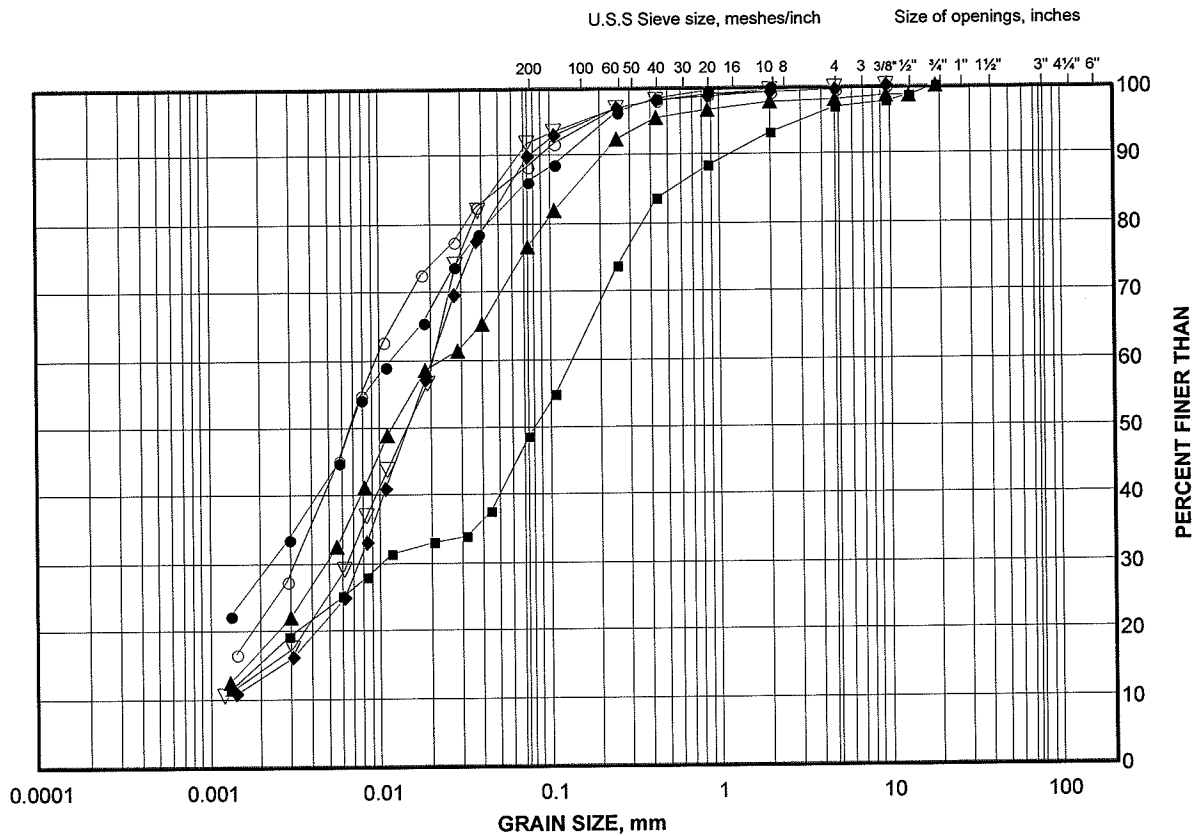
Golder Associates

Date: 12-Nov-07

GRAIN SIZE DISTRIBUTION TEST RESULTS

Lower Clayey Silt Till

FIGURE 6



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

LEGEND

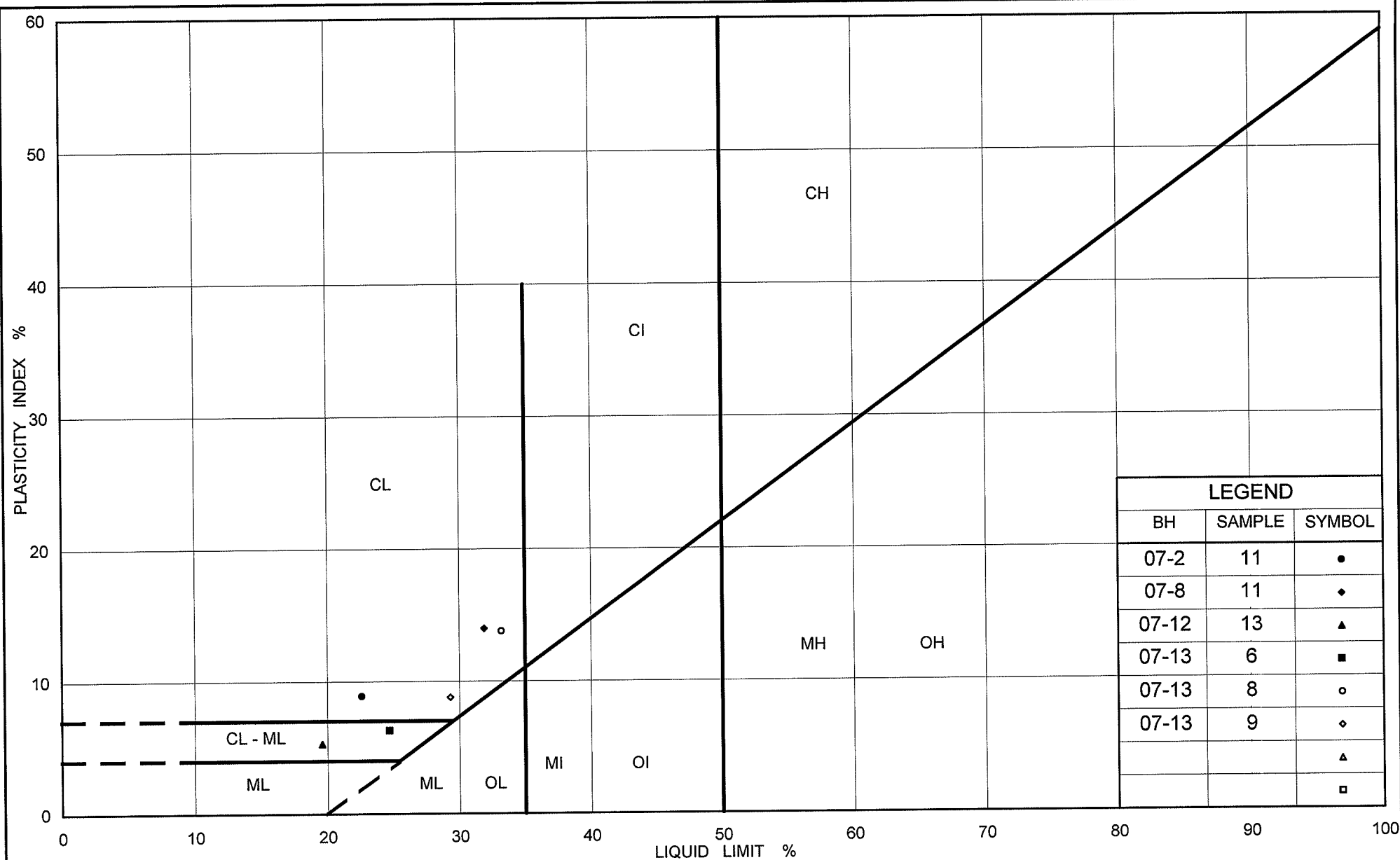
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-8	10	157.2
■	07-6	10	158.3
◆	07-13	11	149.2
▲	07-5	12	155.3
▽	07-12	13	146.5
○	07-13	7	155.1

Project Number: 06-1111-060-1

Checked By: *Wozje*

Golder Associates

Date: 12-Nov-07



Ministry of Transportation

Ontario

PLASTICITY CHART Lower Clayey Silt Till

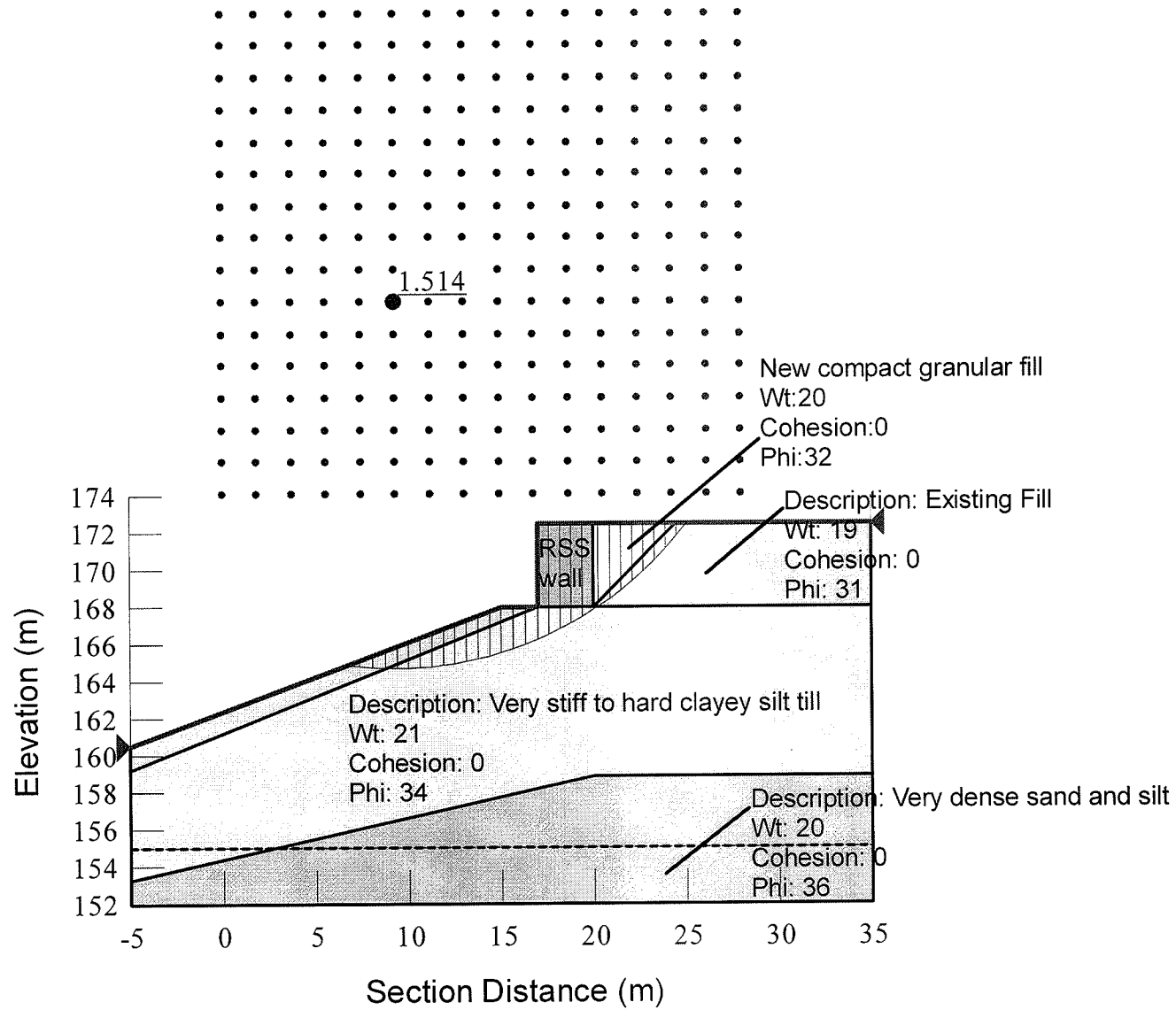
Figure No. 7

Project No. 06-1111-060-1

Checked By: *Wozje*

STATIC GLOBAL STABILITY - RSS WALLS

FIGURE 8



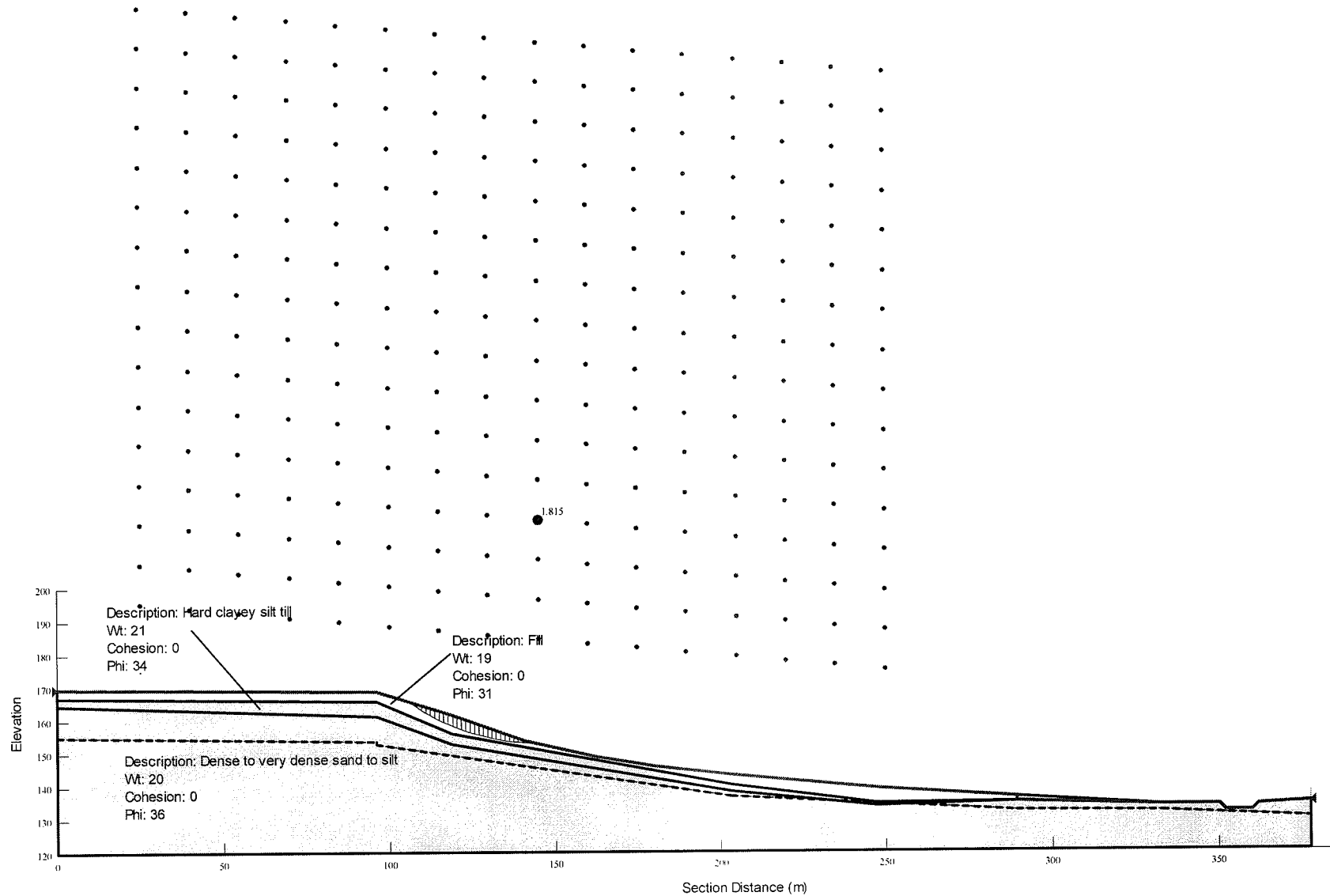
Date: October 2007
Project: 06-1111-060-1

Golder Associates

Drawn: SH
Checked: LCC *ll*

STATIC GLOBAL STABILITY - WEST SLOPE OF WEST DON RIVER VALLEY

FIGURE 9



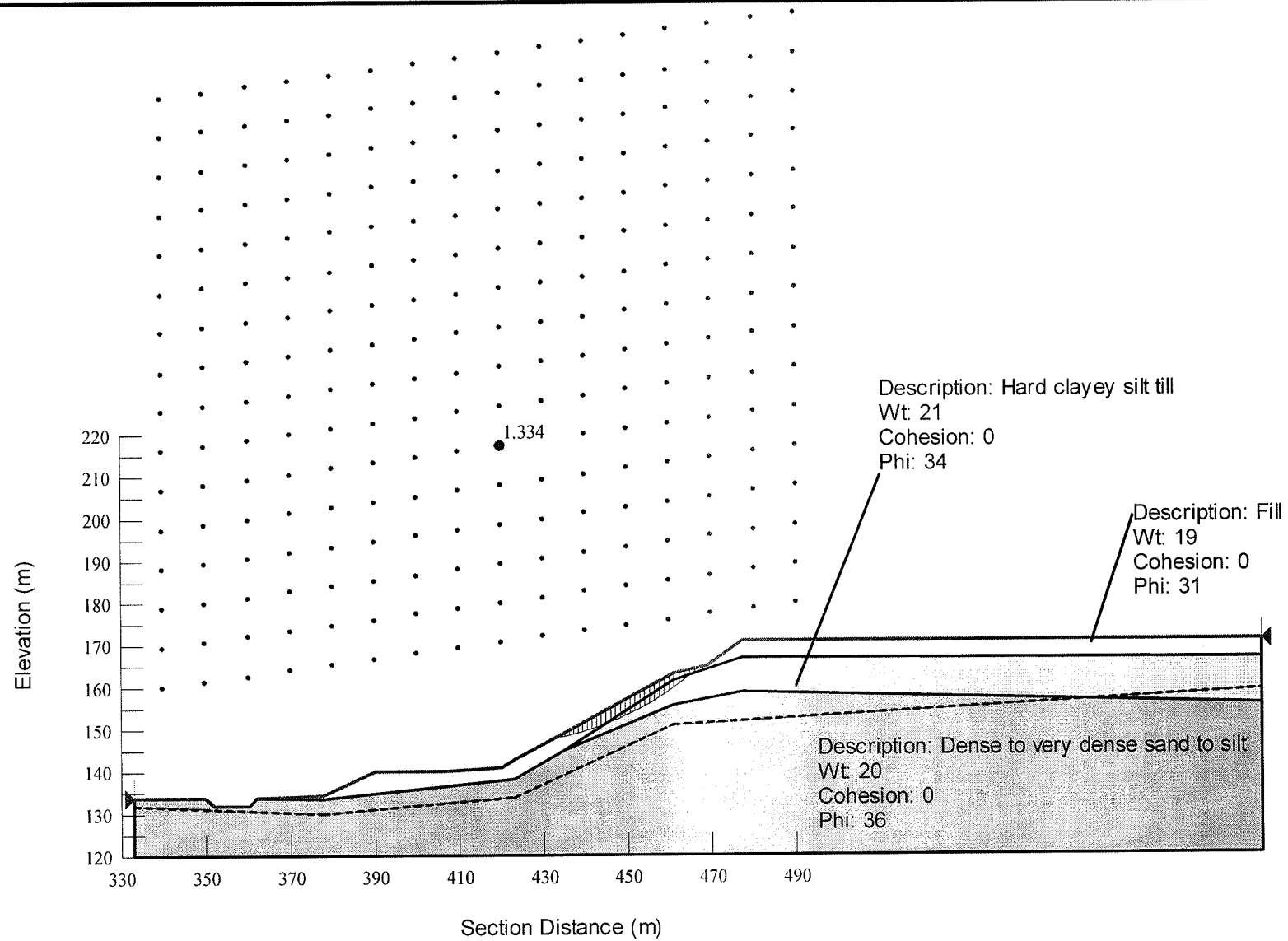
Date: October 2007
Project: 06-1111-060-1

Golder Associates

Drawn: SH
Checked: LCC *SH*

STATIC GLOBAL STABILITY - EAST SLOPE OF EAST DON RIVER VALLEY

FIGURE 10



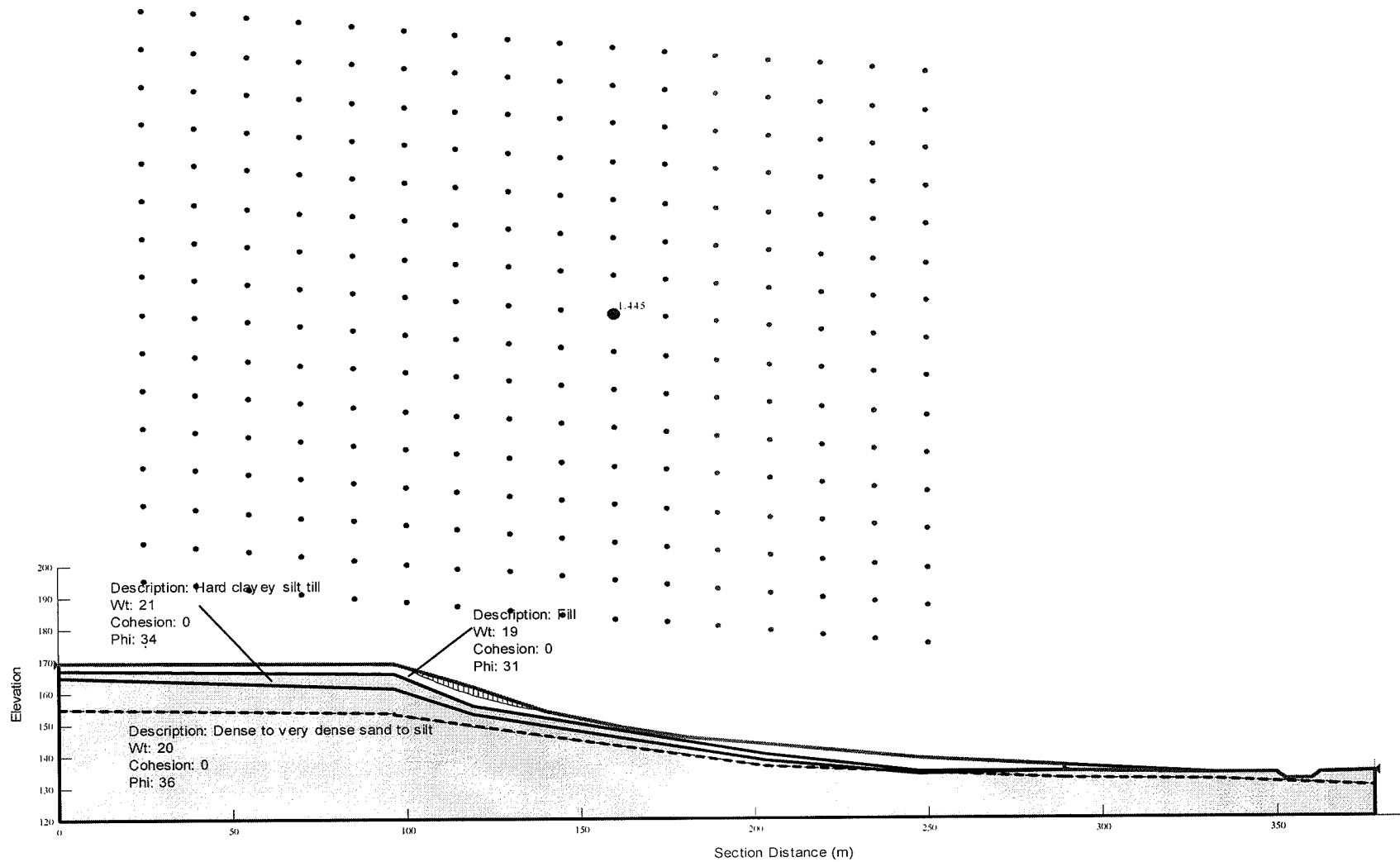
Date: October 2007
Project: 06-1111-060-1

Golder Associates

Drawn: SH
Checked: LCC *vl*

SEISMIC GLOBAL STABILITY - WEST SLOPE OF WEST DON RIVER VALLEY

FIGURE 11



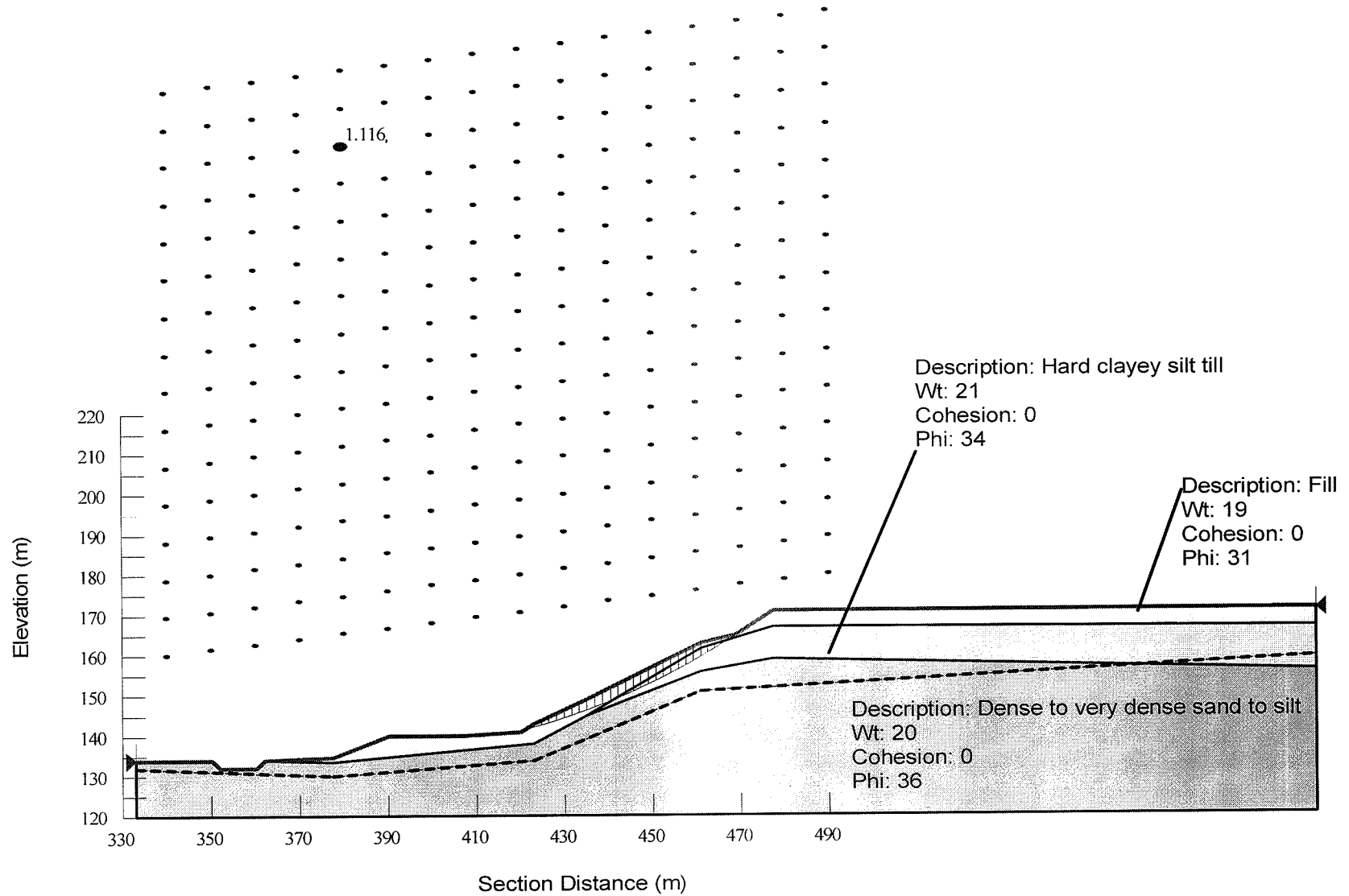
Date: October 2007
Project: 06-1111-060-1

Golder Associates

Drawn: SH
Checked: LCC *SH*

SEISMIC STABILITY - EAST SLOPE OF EAST DON RIVER VALLEY

FIGURE 12

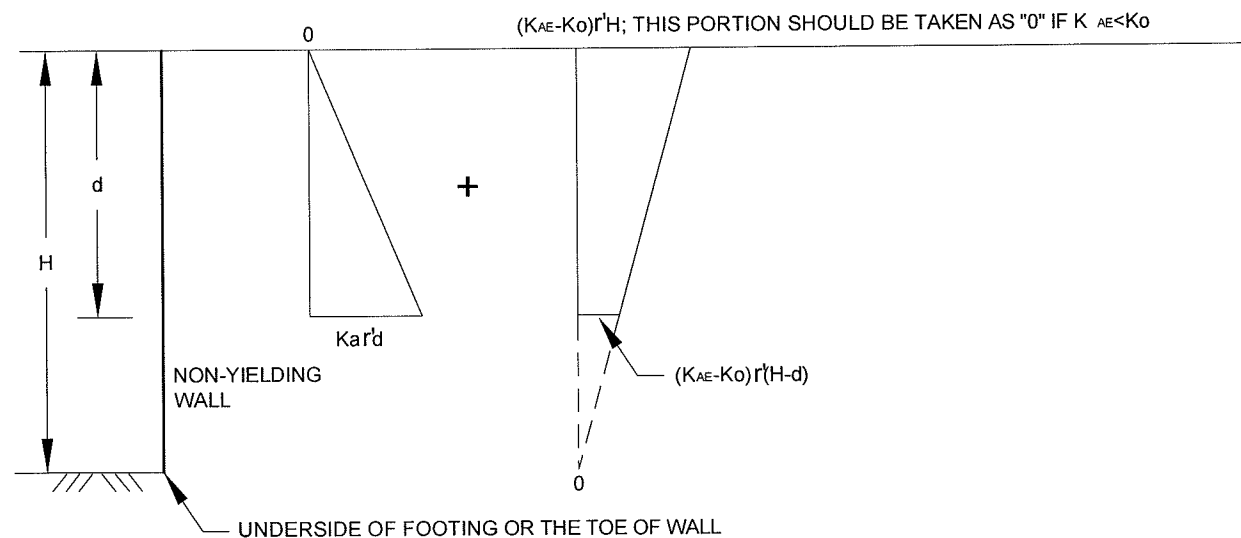
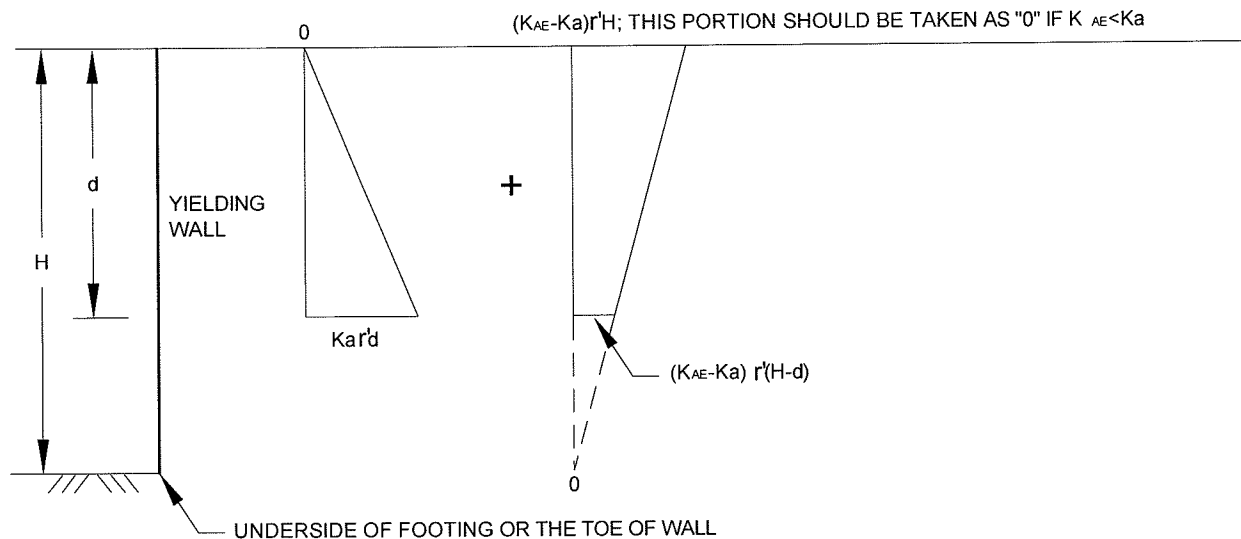


Date: October 2007
Project: 06-1111-060-1

Golder Associates

Drawn: SH
Checked: LCC *ll*

PLOT DATE: November 13, 2007
FILENAME: T:\Projects\2006\06-1111-060 (MH, Hoggs Hollow, Toronto)\-DA- Phase 5400 - Retaining Walls\06111060DA013.dwg



- K_a = STATIC ACTIVE EARTH PRESSURE COEFFICIENT
 K_o = STATIC AT REST EARTH PRESSURE COEFFICIENT
 K_{AE} = SEISMIC ACTIVE EARTH PRESSURE COEFFICIENT
 r' = EFFECTIVE UNIT WEIGHT OF SOIL
 H = THE TOTAL HEIGHT OF WALL ABOVE UNDERSIDE OF THE FOOTING OR THE TOE OF WALL
 d = THE INVESTIGATED DEPTH BELOW THE TOP OF WALL



SCALE N.T.S.
DATE Nov. 13, 2007
DESIGN SH
CAD DD

TITLE

SCHEMATIC SEISMIC EARTH PRESSURE DISTRIBUTION BEHIND WALLS

FILE No. 061111060DA013.dwg

PROJECT No. 06-1111-060

REV.

CHECK SH
REVIEW LCC

HOGG'S HOLLOW BRIDGE AND
RETAINING WALLS, HIGHWAY 401

FIGURE

13

APPENDIX A

NON-STANDARD SPECIAL PROVISIONS

SUBGRADE PROTECTION – Item No.

Special Provision

Where shallow foundations are adopted, the clayey silt till subgrade will be susceptible to softening and degradation on exposure to water and construction traffic. If the concrete for the footings cannot be poured within four hours after inspection and approval of the subgrade, a working mat of lean concrete or mass concrete, with a minimum thickness of 100 mm, should be placed on the foundation subgrade.

Lean concrete shall have a compressive strength of at least 5 MPa, and be placed in accordance with OPSS 904.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

**CONTROL OF OVERBURDEN SOILS AND GROUNDWATER DURING CAISSON
INSTALLATION - Item No.**

Special Provision

Caisson excavations for the bridge abutments or pier widenings will be advanced through fill of varying composition, clayey silt till, and sand and gravel, sand, and sandy silt soils; lenses or layers of cohesionless soils should also be expected to be present within the till deposit. Where cohesionless soil deposits, layers or lenses are encountered, appropriate construction procedures and equipment will be required to minimize ground loss during drilling and concrete placement.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during pile or caisson installation works.

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

Submission Requirements

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibrations monitoring specialist.
- Proposed instrumentation.
- Proposed location of instruments on the existing Hogg's Hollow core and collector structures.
- Proposed frequency of readings.
- Proposed methods for adjusting piling or caisson installation methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Monitoring

The Contractor shall take readings during driving of each pile or installation of each caisson. The measured vibrations shall not exceed 50 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven or caisson installed, prior to continuing with the subsequent piles/caissons. As a minimum, the pile or caisson number, location, set criteria (if applicable) and driving/installation log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with subsequent piles/caissons with readings taken during installation of each pile/caisson. The results of subsequent piles/caissons should be submitted to the Contract Administrator after each pile/caisson has been installed.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations are within acceptable levels.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION