



**AUGUST 2009**

## **PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT**

**LANGSTAFF ROAD UNDERPASS  
HIGHWAY 427 EXTENSION  
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE  
MINISTRY OF TRANSPORTATION, ONTARIO  
W.O. 05-20012**

**Submitted to:**  
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**REPORT**

**GEOCRES No: 30M13-169**

**Report Number:** 06-1111-012-7

**Distribution:**

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**PRELIMINARY FOUNDATION REPORT  
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# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
LANGSTAFF ROAD UNDERPASS  
HIGHWAY 427 EXTENSION  
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE  
W.O. 05-20012**



## PRELIMINARY FOUNDATION REPORT LANGSTAFF ROAD UNDERPASS - HIGHWAY 427 EXTENSION

### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the proposed 6.6 km long extension of Highway 427 from Highway 7 northward to Major Mackenzie Drive in the City of Vaughan, Ontario. The terms of reference for the foundation engineering services are provided in the Request for Proposal for MTO Assignment No. 2005-E-0028, dated December 21, 2005.

This report addresses the preliminary foundation investigation carried out for the proposed Langstaff Road underpass structure over the new Highway 427, and the immediate approach embankments for Langstaff Road underpass. The approximate location of this site on the Highway 427 Extension alignment is shown on Figure 1.

The work was carried out in accordance with Golder's Supplemental Speciality Quality Control Plan for foundation engineering services for this project dated April 4, 2006.

### 2.0 SITE DESCRIPTION

The proposed Langstaff Road underpass structure is located approximately 1.4 km north of Zenway Boulevard in the City of Vaughan, Ontario. The proposed structure is located approximately 1 km west of Highway 27 and approximately 1.1 km east of Huntington Road (see Figure 1).

In general, the topography in the area of the overall project limits consists of flat-lying to gently sloping land which is associated with the two creek crossings: Rainbow Creek and West Robinson Creek. The terrain consists of densely treed areas and farmland. Along Zenway Boulevard, Langstaff Road and Rutherford Road there is some residential, commercial and/or light industrial development.

Langstaff Road is fairly flat in vicinity of the proposed Highway 427 underpass, with the ground surface of the road typically at about Elevation 187.5 m. A hydro corridor, running in a north-south direction, is located east of the proposed structure site.

### 3.0 INVESTIGATION PROCEDURES

The field work for the Langstaff Road underpass structure investigation was carried out in March and April, 2009 during which time a total of three boreholes were advanced. The boreholes, designated as Boreholes S12, S13, and S14/S14A, were advanced at the locations shown on Drawing 1.

The field investigation for the boreholes was carried out along Langstaff Road using a truck-mounted CME 55 drill rig supplied by Walker Drilling Ltd. of Utopia, Ontario. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). The boreholes were terminated after penetrating either at least 3 m into hard or very dense soil having Standard Penetration Test (SPT) 'N' values of greater than 100 blows per 0.3 m of penetration, or into shale bedrock, where encountered. The boreholes were advanced to depths ranging from 23.1 m to 27.5 m below existing ground surface.

The groundwater conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in Borehole S12 to permit monitoring of the groundwater level at the site. The piezometer consisted of 51 mm diameter PVC pipe, with a slotted screen sealed at a selected depth within the borehole. A sand filter pack surrounds the screen, and above the screen the borehole and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are described on the Record of Borehole Sheets in Appendix A. The boreholes



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in which no standpipe piezometers were installed were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and geotechnical classification testing (water content, Atterberg limits and grain size distribution tests). All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate.

Prior to drilling, the boreholes were located in the field using the Highway 427 Extension alignment centreline stakes installed by MRC and a Global Positioning System (GPS) unit. The as-drilled borehole locations and ground surface elevations were surveyed by MRC. The borehole locations shown on Drawing 1 and on the borehole records are given relative to MTM NAD 83 northing and easting coordinates, and the ground surface elevations are referenced to geodetic datum.

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

The Highway 427 Extension area lies within the Peel Plain physiographic region, as delineated in *The Physiography of Southern Ontario*<sup>1</sup>. A surficial till sheet, which generally follows the surface topography, is 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. 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. The study area is underlain by Ordovician shales of the Georgian Bay Formation.

### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced for this investigation and the results of the laboratory tests carried out on selected soil samples are provided in Appendices A and B, respectively. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change.

The interpreted stratigraphic conditions along Langstaff Road at the proposed underpass site are shown on Drawing 1. This stratigraphic profile represents a simplification of the subsurface conditions as encountered in the boreholes. Variation in the stratigraphic boundaries and properties of the soil deposits will occur between and beyond the borehole locations.

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<sup>1</sup> Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.



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In general, the subsurface conditions in the area of the proposed underpass consist of asphalt underlain by up to 1.5 m of silty sand and gravel fill and clayey silt fill. The fill is underlain by a till deposit consisting of clayey silt and silty clay with layers of sand and silty sand deposits. The till deposit overlies shale bedrock.

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

### 4.2.1 Asphalt

Approximately 100 mm of asphalt was encountered immediately below ground surface in all boreholes drilled for this site.

### 4.2.2 Fill

In all boreholes drilled for this site, the asphalt is underlain by fill that extends to depths of between 0.8 m and 1.5 m below ground surface (between Elevations 186.1 m and 186.9 m). The composition of the fill varies; the upper 0.2 m to 0.4 m consists of silty sand containing some gravel. The silty sand fill is underlain by clayey silt fill which varies in thickness from about 0.5 m to 1.1 m. The clayey silt fill contains trace to some sand and trace gravel, as well as rootlets. One measured Standard Penetration Test (SPT) 'N' value in the silty sand was 29 blows per 0.3 m of penetration, indicating that the silty sand fill has a compact relative density. The SPT 'N' values measured in the clayey silt fill were 13, 15 and 41 blows per 0.3 m of penetration, indicating that the clayey silt fill has a stiff to hard consistency. The measured water content on a sample of the clayey silt fill was 19 percent.

### 4.2.3 Silty Clay to Clayey Silt Till

In all boreholes drilled for this site, the fill is underlain by a cohesive till deposit that varies in composition from silty clay to clayey silt. Boreholes S13 and S14/S14A fully penetrated the till deposit, which was found to have an overall thickness of approximately 23 m. The base of the till deposit was encountered in Boreholes S13 and S14A at Elevations 163.9 m and 165.9 m, respectively, although the deposit base may be lower than this in Borehole S12 where it was not fully penetrated. The silty clay portions were encountered in Boreholes S12 and S13 within the upper 4.6 m below ground surface, with the base encountered at about Elevation 182.9 m to 183.1 m. Silty clay till was also encountered near the base of the till deposit in Boreholes S12 and S14/14A at depths of 18.9 m and 18.4 m, respectively. The surface of the lower silty clay till was encountered at Elevations 168.6 m and 169.3 m and extended to Elevations 166.6 m and 165.9 m (equating to thicknesses of about 2.0 m and 3.4 m).

The silty clay portion of the cohesive till contains trace sand and gravel whereas the clayey silt portion contains some sand and trace gravel. The results of a grain size distribution test completed on one selected sample of the clayey silt portion of the till deposit is provided on Figure B1 in Appendix B. Within the cohesive till deposit layers of sand to silty sand were encountered; these layers varied in thickness from about 0.7 m to 2.1 m (see Section 4.2.4 for more details).

Till deposits in southern Ontario typically contain cobbles and/or boulders. Cobbles and/or boulders have been inferred to be present within the till deposits at this site, based on grinding of the augers during borehole drilling, as summarized below:





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Borehole No.	Depth of Grinding of Augers	Elevation of Inferred Cobbles / Boulders
S12	11.4 m	176.1 m
	17.6 to 18 m	169.9 to 169.5 m
	22.1 to 22.5 m	165.4 to 165.0 m
S13	5.2 m	182.5 m
	8.4 m	179.3 m
	21 m	166.7 m
	22 m	165.7 m

Atterberg limits testing was carried out on four samples of the silty clay portion of the till deposit. The plastic limits varied from 16 to 19 percent, the liquid limits varied from 37 to 42 percent, and the plasticity indices varied from 20 to 24 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that this portion of the till deposit is a silty clay of medium plasticity. Measured water contents on samples of the silty clay till ranged from about 12 to 25 percent.

Atterberg limits testing was carried out on nine samples of the clayey silt portion of the till deposit. The measured plastic limits varied from 11 to 16 percent, the liquid limits varied from 16 to 33 percent and the plasticity indices varied from 4 to 17 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that this portion of the till deposit is a clayey silt of low plasticity. Measured water contents on samples of the clayey silt till ranged from about 9 to 14 percent with one value of 25 percent.

The SPT 'N' values measured within the silty clay to clayey silt till deposit typically ranged from 17 blows to greater than 100 per 0.3 m of penetration, indicating a very stiff to hard consistency. SPT 'N' values as low as 12 blows per 0.3 m of penetration were recorded in the near-surface portions of the till deposit (from ground surface to a depth of about 1.5 m), indicating that this portion of the till has a stiff consistency.

### 4.2.4 Sand to Silty Sand Layers

In Borehole S13, a layer of silty sand containing trace gravel was encountered within the clayey silt till deposit. The surface of the layer was encountered at a depth of 13.7 m, and the layer extended from Elevation 174.0 m to 172.5 m and therefore has a thickness of about 1.5 m. At approximately 2.5 m below the base of this silty sand layer, a continuous layer of sand to silty sand was encountered within the clayey silt till deposit in all of the boreholes drilled for this site. The surface of the sand to silty sand layer was encountered between Elevations 170.0 m and 170.1 m (at depths of between 17.4 m and 17.7 m). The thickness of sand to silty sand layer varied from about 0.7 m to 2.1 m and the base of this layer was encountered between Elevations 167.9 and 169.3 m.

The sand contains trace gravel and trace silt and the silty sand contains trace to some silt, trace gravel and trace clay. The results of grain size distribution tests completed on two selected samples of the sand portion of the deposit are provided on Figure B3 in Appendix B. Measured water contents on samples of the sand to silty sand deposit varied from about 13 to 15 percent.

During drilling within the sand layer in Boreholes S12 and S13, "blowing" sand was encountered, in which the sand penetrated up inside the hollow stem augers. In Borehole S12 the sand came 1.2 m up inside the hollow stem augers. Since Borehole S12 was drilled first "blowing" of the sands was prevented in Borehole S13 by maintaining a hydrostatic head of water inside the augers during drilling, thereby counterbalancing the water pressures at the base of the borehole. "Blowing" sand was not encountered in Borehole S14/14A.

The SPT 'N' values measured in the sand and silty sand deposit were 12 and 22 blows per 0.3 m of penetration, indicating that the sand and silty sand has a compact relative density.



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### 4.2.5 Sand and Silt Till

A sand and silt till deposit was encountered underlying the silty clay to clayey silt till deposit in Borehole S14A at a depth of depth of 21.8 m (Elevation 165.9 m). The sand and silt till deposit is approximately 1.9 m thick and extended to Elevation 164.0 m where it overlies shale bedrock.

The sand and silt till contains some gravel and trace clay. A grain size distribution test was completed on one selected sample of the sand and silt till deposit and the result is provided on Figure B4 in Appendix B. The measured water content on one sample of the sand and silt till deposit was 9 percent.

The measured SPT 'N' value in the sand and silt till deposit was 70 blows per 0.3 m of penetration, indicating that the sand and silt till deposit has a very dense relative density.

### 4.2.6 Shale Bedrock

Bedrock was encountered and split-spoon samples were recovered from Boreholes S13 and S14/S14A. The bedrock surface in these boreholes was at a depth of 23.8 m and 23.7 m below ground surface (Elevation 163.9 m and 164.0 m, respectively), as summarized in the table below. The bedrock samples consisted of light grey to dark grey shale. Based on available bedrock geology maps, the bedrock at this site is understood to be part of the Georgian Bay Formation.

Borehole No.	Depth to Bedrock Surface	Bedrock Surface Elevation
S13	23.8 m	163.9 m
S14A	23.7 m	164.0 m

## 4.3 Groundwater Conditions

The water level in the open Boreholes S12 and S13 was between about Elevation 179.9 m and 181.5 m (at a depth of about 5.2 and 7.8 m below ground surface) as noted during and upon completion of drilling operations, although the level had not stabilized. In Borehole S14, which was drilled to a depth of 18.9 m, the water level in the open borehole on completion of drilling was at a depth of about 17.7 m, which corresponds to about Elevation 170.0 m. Borehole S14A which is about 1 m east of Borehole S14 was drilled to a depth of 24.5 m and the water level in the open borehole upon completion of drilling was at 21.8 m depth (Elevation 165.9 m).

A standpipe piezometer was installed in Borehole S12 sealed into the sand layer within the cohesive till deposit to permit monitoring of the groundwater level at this site. Details of the piezometer installation are shown in the borehole records in Appendix A. The groundwater level measured in the piezometer installation, some six weeks following borehole completion, is summarised below.

Borehole No.	Ground Surface Elevation	Groundwater Depth	Groundwater Elevation	Date of Measurement
S12	187.5 m	6.9 m	180.6 m	May 13, 2009
		6.7 m	180.8 m	June 15, 2009
		6.3 m	181.3 m	July 9, 2009



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The groundwater levels in the area should be expected to be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

### 5.0 CLOSURE

The field technicians directing the drilling program were Messrs. Chris Radway and Jordan Black. This Foundation Investigation Report was prepared by Ms. Sandra McGaghran, P.Eng., a geotechnical engineer with Golder and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

  
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**PRELIMINARY FOUNDATION REPORT  
LANGSTAFF ROAD UNDERPASS - HIGHWAY 427 EXTENSION**

# **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
LANGSTAFF ROAD UNDERPASS  
HIGHWAY 427 EXTENSION  
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE  
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### 6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

This section of the report provides foundation design recommendations for the preliminary design of the proposed Langstaff Road underpass structure on the Highway 427 NBL and SBL mainline alignment. The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the preliminary design of the project, and for which special provisions are expected to be required as the project proceeds through detail design and into contract preparation. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Further borehole investigation and analysis will be required during the detail design phase of the project, once the configuration of the proposed underpass structure is finalized, to confirm and expand on the preliminary foundation recommendations provided in this report.

#### 6.1 General

The Langstaff Road underpass is proposed to consist of a two-span structure with a centre pier in the median of Highway 427. Based on the preliminary General Arrangement (GA) Drawing provided by MRC on May 15, 2009, the span length between the abutments and the centre pier is approximately 45 m.

According to the preliminary GA Drawing, the finished grade of Langstaff Road will be at about Elevation 193.8 m, which is approximately 8 m above the proposed Highway 427 grade (Elevation 185.8 m). The existing ground surface along Langstaff Road is at about Elevation 187.5 m; therefore, the east and west approach embankments will be about 6.3 m high relative to the existing ground surface. The existing grade is proposed to be lowered by about 1.7 m to construct Highway 427 at Elevation 185.8 m.

### 6.2 Foundation Recommendations

#### 6.2.1 Foundation Options

Based on the proposed vertical elevations and subsurface soil conditions, the following foundation options are considered feasible for the Langstaff Road underpass:

- **Spread footings founded on the very stiff to hard clayey silt to silty clay till:** Considering that the grade at the east and west abutments is to be raised by about 6.3 m, this option may not be economical at the abutments given the resulting height of abutment walls. However, this option is feasible at the centre pier, where an approximately 1.7 m deep cut is proposed to construct the Highway 427 NBL, SBL and median at about Elevation 185.8 m. Based on the results from Borehole S13 drilled adjacent to the centre pier, spread footings founded at a minimum depth of 1.4 m below the proposed cut grade would be supported on the very stiff to hard clayey silt till.
- **Spread footings “perched” on a granular pad within the approach embankment fill:** This option could be adopted to support the abutments for an open structure, with 2 horizontal to 1 vertical (2H:1V) foreslopes in front of the abutment footings.



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- **Steel H-piles driven to found within the till deposit or bedrock:** Steel H-piles founded within the hard clayey silt to silty clay till (having SPT 'N' values greater than 100 blows per 0.3 m of penetration) could be adopted at the centre pier and west abutment. In Borehole S14A drilled near the east abutment, soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration was not encountered, and steel H-piles at this location could be supported on the bedrock. This option could be adopted to support the abutments and centre pier in either a conventional or an integral abutment-type structure. Given that the site soils will not present long-term settlement issues, the site is considered suitable for the use of integral abutments. Alternatively, an open bridge configuration could be adopted, in conjunction with 2H:1V foreslopes in front of the abutment pile caps.
- **Caissons founded within the till deposit or bedrock:** This option could be adopted to support the abutments and center pier in either a conventional or a semi-integral abutment-type structure.

At the abutments, either “perched” footings or steel H-piles are preferred over spread footings founded on the native soils due the resulting height of the abutment walls. At the centre pier, the grade of the Highway 427 will be lowered by about 1.7 m compared to the current grade, which will permit spread footings to be founded on very stiff to hard clayey silt to silty clay till at a depth of 1.4 m below the cut grade. Spread footings are therefore preferred for support of the centre pier if sufficient geotechnical resistance can be achieved; otherwise, support of the centre pier on deep foundations will be required to achieve a higher capacity. The use of piles is preferred from a foundations perspective over caissons as “blowing” sands were encountered at a depth of about 18.3 m (Elevation 169.4 m) in Boreholes S12 and S13 at the west abutment and the centre pier, respectively. The caissons would require a temporary or permanent steel liner and also may require a hydrostatic head be maintained inside the steel liner while advancing the caisson through the “blowing” sand layer. Therefore considering these special construction procedures, piles are preferred over caissons from a foundation and constructability perspective.

Recommendations for preliminary design of spread footings, steel H-pile and caisson foundations are presented in the following sections. A summary comparison of the advantages, disadvantages and relative costs associated with each of the feasible foundation options is presented in Table 1 following the text of this report.

### 6.2.2 Spread Footings on Native Soils

The following sections provide geotechnical resistances for spread footings founded on the very stiff to hard silty clay to clayey silt till.

#### 6.2.2.1 Founding Elevations

The abutments and center pier may be supported on spread footings placed below the fill and stiff silty clay till on very stiff to hard clayey silt to silty clay till. Currently, it is proposed that the existing grade is to be lowered by about 1.7 m to construct Highway 427. Founding elevation recommendations are provided in the table below based on a Highway 427 grade of Elevation 185.8 m, with a minimum embedment depth of 1.4 m to provide adequate protection against frost penetration (per OPSD 3090.101). In the event that the Highway 427 cut grade is raised during detail design, preliminary recommendations for maximum (highest) founding elevations are also provided in the following table; these founding levels should be checked to ensure that a minimum of 1.4 m of frost protection is provided (OPSD 3090.101).





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Foundation Element	Borehole No.	Founding Stratum	Maximum Founding Elevation Based on 1.7 m Cut	Maximum Founding Elevation Based on Shallower Cut
West Abutment	S12	Hard Silty Clay Till	184.4 m	186.0 m
Center pier	S13	Very Stiff to Hard Silty Clay Till	184.4 m	186.0 m
East Abutment	S14	Very Stiff to Hard Clayey Silt Till	184.4 m	186.0 m

### 6.2.2.2 Geotechnical Resistances

A factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 350 kPa (for 25 mm of settlement) may be used for preliminary design purposes, assuming 3 m wide footings.

The ULS and SLS resistances and settlement are dependent on the footing size, configuration and applied loads. The geotechnical resistances should, therefore, be reviewed during detail design, once further drilling has been carried out at the foundation elements to delineate the thickness of the fill and confirm the founding level, once the final geometry of the foundations has been established.

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 Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*, using the curves for cohesive soils.

### 6.2.2.3 Resistances to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the very stiff to hard native clayey silt to silty clay till should be calculated in accordance with Section 6.7.5 of the *CHBDC*. A coefficient of friction,  $\tan \phi'$ , of 0.55 can be used for cast-in-place concrete footings on the properly prepared clayey silt till subgrade. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating horizontal resistance.

## 6.2.3 “Perched” Spread Footings

The following sections provide geotechnical resistances for spread footings at the abutments that are “perched” within the approach embankment fill on a compacted granular pad.

### 6.2.3.1 Founding Elevations

“Perched” abutment footings founded on Ontario Provincial Standard Specification (OPSS) 1010 Granular ‘A’ pads should be provided with a minimum of 1.4 m of soil cover for frost protection (OPSD 3090.101).

For this option, subexcavation of any existing topsoil, organic or loosened/softened material is recommended within the embankment footprint below the perched abutment, to minimize settlement due to the embankment loading. Based on the results of the preliminary investigation it is not anticipated that the compact to dense



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Langstaff Road fill will require subexcavation. However, further investigation is recommended during the detail design stage to confirm the required thickness of such subexcavation. If required, the area to be subexcavated, should be defined by a line extending from the toe of the OPSS 1010 Granular 'A' pad, outward and downward at 1 horizontal to 1 vertical (1H:1V). The subexcavation should be replaced with compacted OPSS 1010 Granular 'B'. The Granular 'A' pad should be a minimum of 2 m thick and should extend at least 1 m beyond the plan limits of the footing. The Granular 'A' pad should be constructed in accordance with MTO Special Provision SP105S10.

### 6.2.3.2 Geotechnical Resistances

Assuming the above subexcavation depths and filling procedures, a factored geotechnical resistance at ULS of 850 kPa may be used for preliminary design. The geotechnical resistance at SLS may be taken as 350 kPa. These geotechnical resistances will have to be reviewed during detail design, after further drilling has been carried out at the foundation elements to confirm the extent of subexcavation that is required, and once the final geometry of the foundations and approach embankments has been established.

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 Section 6.7.4 of the *CHBDC* and its *Commentary*, using the curves for non-cohesive soils.

### 6.2.3.3 Resistances to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the compacted Granular 'A' pad should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction,  $\tan \phi'$ , can be taken as 0.70. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating horizontal resistance.

### 6.2.4 Steel H-Piles

Preliminary geotechnical recommendations for steel H-pile foundations driven to found within the till deposits and on the shale bedrock are provided in the subsections that follow.

For the installation of steel H-piles, consideration will have to be given to the potential presence of cobbles and/or boulders within the till. It is recommended that the piles be stiffened with driving shoes/flange plates for protection during driving, in accordance with OPSS 903.07.05.04 and OPSD 3000.100. Pile installation and driving shoes should be in accordance with Special Provision SP903S01.

Battered steel H-piles driven to bedrock should be fitted with rock points to facilitate proper seating on the bedrock. The driving procedures to enable pile seating depend on the type of pile driving rig used. Generally, the procedures will involve a reduction in hammer energy once abrupt peaking is met to ease the pile point into the rock.

#### 6.2.4.1 Founding Elevations

Steel H-piles driven to found within the hard silty clay to clayey silt till deposit may be used for support of the west abutment and the center pier. In Borehole S14A, drilled in vicinity of the east abutment, "refusal" (i.e. soil





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having SPT 'N' values greater than 100 blow per 0.3 m of penetration) was not encountered, and it is recommended that piles for this foundation element be founded on the shale bedrock. The table below summarizes the estimated pile tip elevations for preliminary design purposes, based on assumed penetration of approximately 1.5 m into soil having SPT 'N' values of greater than 100 blows per 0.3 m of penetration.

Foundation Unit	Borehole No.	Founding Stratum	Estimated Pile Tip Elevation
West Abutment	S12	Hard Silty Clay Till	167.1 m
Center pier	S13	Hard Clayey Silt Till	164.5 m
East Abutment	S14/S14A	Shale Bedrock	163.9 m

### 6.2.4.2 Geotechnical Axial Resistances

For HP 310x110 piles driven to the estimated tip elevations provided in Section 6.2.4.1 above (taken as approximately 1.5 m into soil having SPT 'N' values greater than 100, or on the surface of the shale bedrock, as applicable), the factored axial geotechnical resistance at ULS and the axial geotechnical resistance at SLS (for 25 mm of settlement) are given below. For piles driven to bedrock at the east abutment, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, and a result SLS condition do not apply. The geotechnical resistances given for shale bedrock below will need to be confirmed at the detail design stage

Foundation Unit	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
West Abutment	Hard Silty Clay Till	1,600 kN	1,400 kN
Centre Pier	Hard Clayey Silt Till	1,400 kN	1,200 kN
East Abutment	Shale Bedrock	2,000 kN	N/A

At the proposed east and west abutment area it is estimated that up to about 25 mm of settlement will occur, under the proposed loading from the approach embankment. For preliminary design purposes it is recommended that a downdrag load of 100 kN be included, although further investigation and assessment will be required during detail design stage. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.

The pile capacity values provided above will have to be reviewed and modified if necessary during detail design, further to additional subsurface investigations at the locations of each bridge foundation element.

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 size and length of pile. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an ultimate capacity equal to the final recommended factored ULS capacity divided by a resistance factor of 0.5 applicable to the use of the Hiley formula.



### 6.2.4.3 Resistances to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

The resistance to lateral loading in front of the pile, as well as pile group action for lateral loading if the pile spacing in the direction of loading is less than six to eight pile diameters, should be accounted for and assessed during the detail design phase of the project. For preliminary design, a factored lateral geotechnical resistance at ULS of 200 kN may be used and a lateral geotechnical resistance at SLS of 110 kN (for 10 mm of lateral displacement at the pile cap level) may be used for a single vertical HP 310x110 pile embedded in very stiff silty clay till to clayey silt till. These values are 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*.

### 6.2.4.4 Frost Protection

All pile caps should be provided with a minimum of 1.4 m of soil cover for frost protection (OPSD 3090.101).

## 6.2.5 Caissons

Consideration could be given to the use of caissons socketted into the hard silty clay to clayey silt till for support of the foundation elements for the west abutment and pier, and caissons founded on the shale bedrock for support of the east abutment. Consideration could also be given to extending the caissons at the west abutment and centre pier to the shale bedrock, if higher geotechnical resistances are desired. Preliminary geotechnical recommendations for caisson foundations are provided in the sub-sections that follow.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, particularly while advancing the caissons through the “blowing” sand layers. If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner would be required, possibly in conjunction with drilling fluids when drilling through sand layers, to support the soils during construction, to prevent the movement of “blowing” sands from coming up into the liners, and to permit inspection and cleaning of the caisson base.

### 6.2.5.1 Founding Elevations

The recommended pile tip elevations as given in Section 6.2.4.1 may also be used for preliminary design for the founding elevations for caissons.

### 6.2.5.2 Geotechnical Resistances

The table below provides preliminary recommendations for factored axial geotechnical resistance at ULS and axial geotechnical resistance at SLS (for 25 mm of settlement) for caissons at the west abutment and pier founded within the hard silty clay to clayey silt till at the elevations given in Section 6.2.4.1. Preliminary recommendations for factored axial geotechnical resistance at ULS for caissons founded on the shale bedrock at the elevations given in Section 6.2.4.1 at the east abutment location are given below. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, and a result



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SLS condition do not apply. The same geotechnical resistances can be used for caissons founded on bedrock at the west abutment and centre pier, although further investigation would be required during detail design to confirm the bedrock elevation at the west abutment location. These preliminary recommendations should be verified by bedrock coring at the detail design stage if caissons foundations on bedrock are adopted.

Foundation Unit	Founding Stratum	Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
West Abutment	Hard Silty Clay Till	0.9 m	4,400 kN	3,600 kN
		1.2 m	7,900 kN	6,600 kN
		1.5 m	12,400 kN	10,300 kN
Pier	Hard Clayey Silt Till	0.9 m	3,200 kN	2,700 kN
		1.2 m	5,600 kN	4,700 kN
		1.5 m	8,800 kN	7,300 kN
East Abutment	Shale Bedrock	0.9 m	5,000 kN	N/A
		1.2 m	9,000 kN	
		1.5 m	14,000 kN	

### 6.2.5.3 Resistances to Lateral Loads

For preliminary design purposes, a maximum factored lateral resistance at ULS of 400 kN and a maximum lateral resistance at SLS (for 10 mm of horizontal deflection at pile cap level) of 250 kN 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* together with lateral caisson load test data. Values for alternative caisson diameters can be developed if larger diameter caisson foundations are adopted for support of foundation elements at this site.

### 6.2.5.4 Frost Protection

The caisson caps should be provided with a minimum of 1.4 m of soil cover for frost protection (OPSD 3090.101).

## 6.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. 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.



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- Select, free draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 1010 Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. 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 OPSS 3101.150 and OPSS 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 6.6. Compaction equipment should be used in accordance with MTO's Special Provision SP105S10. Other surcharge loadings should be accounted for in the design as required.
- The granular fill may be placed either in a zone with the width equal to at least 1.4 m behind the back of the walls (see Case A in Figure C6.20(a) of the *Commentary* to the *CHBDC*), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the *CHBDC*).
- For Case A, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill :

	Earth Fill
Soil unit weight:	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50

- For Case B, where the pressures are based on OPSS 1010 granular fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
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 should be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as for a rigid frame structure), at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

### 6.3.1 Seismic Considerations

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the *CHBDC*. Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6.4 of



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*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. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

Where	K	is either the static active earth pressure coefficient ( $K_a$ ) or the static at rest earth pressure coefficient ( $K_o$ );
	$K_{AE}$	is the seismic active earth pressure coefficient;
	$\gamma'$	is the effective unit weight of the soil ( $\text{kN/m}^3$ )
		<ul style="list-style-type: none"> <li>taken as soil unit weights given above for fill materials</li> <li>taken as <math>20 \text{ kN/m}^3</math> for the native materials</li> </ul>
	d	is the depth below the top of the wall (m); and
	H	is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1 and the site specific zonal acceleration ratio for the Vaughan area is 0.05. For the thicknesses and type of competent overburden soils at this site, a site coefficient of 1.0 and an amplification factor of 1.33 are recommended. Therefore, the recommended ground surface acceleration is  $0.067g$ .

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of  $A = 0.067$ . These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary*, and 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 A	CASE B	
	Earth Fill	Granular 'A'	Granular 'B' Type II
Yielding Wall	0.29	0.26	0.26
Non-Yielding Wall	0.33	0.29	0.29

Note : These *CHBDC* seismic  $K_{AE}$  values include the effect of wall friction ( $\delta = \phi'/2$ ) and are not greater than the static values of  $K_a$  and  $K_o$  reported above for the very low zonal acceleration ratio for this site.

## 6.4 Approach Embankments

The construction of the Langstaff Road underpass structure will require placement of up to about 6.3 m of fill within the limits of the east and west approach embankments.

Based on the results of the boreholes drilled at this site, the approach embankments will be founded on very stiff to hard silty clay to clayey silt till at the east and west approach embankments.



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### 6.4.1 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be an appropriate subgrade for the proposed approach embankments; however, to improve the embankment performance, it is recommended that prior to the placement of any fill, all topsoil, organic matter, and any softened/loosened native soils should be stripped from below the approach embankment areas. Further investigation of the Langstaff Road fill will be required during detail design to confirm whether the fill can be left in place. Embankment fill should be placed and compacted in accordance with MTO's SP 206S03 and SP 105S10. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection must be in accordance with OPSS 572.

### 6.4.2 Approach Embankment Stability

Static and seismic slope stability analyses of the proposed approach embankments were carried out with the commercially available program SLOPE-W (produced by Geo-Slope International Ltd.), to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered appropriate for the embankments at this site.

The soil parameters used in the analysis, as given in the following table, were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPTs) and geotechnical classification testing. The groundwater table was taken at Elevation 180.6 m in the analyses.

Soil Type	Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength (kPa)	Cohesion, c' (kPa)	Angle of Internal Friction, $\phi'$ (degrees)
New Earth or Granular Fill	21	--	--	34
Compact Silty Sand Fill Very Stiff Clayey Silt Fill	20	--	--	30
Very Stiff to Hard Silty Clay to Clayey Silt Till	21	--	--	32
Hard Clayey Silt Till	21	--	--	34

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 6.3 m high approach embankments with side slopes maintained at 2H:1V will have a factor of safety of greater than 1.3 against deep-seated slope instability. The result of an example static stability analysis is provided on Figure 2.

Under seismic loading conditions with a horizontal peak ground acceleration (HPGA) equal to 0.067g, the factor of safety is greater than 1.2. The result of an example seismic slope stability analysis is shown on Figure 3.





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### 6.4.3 Approach Embankment Settlement

Settlement of the approach embankments at the site will occur due to compression of the new embankment fill itself, as well as compression of the underlying native soils. Provided that the embankment material consists of clean earth fill or granular fill, the settlement of the 6.3 m high approach embankment fill itself is expected to be less than about 25 mm, and this settlement will occur relatively quickly during and immediately following construction.

The settlement of the foundation soils under the approach embankment loading is anticipated to be approximately 25 mm; the majority of this settlement will occur during or immediately following construction of the approach embankments. This compression has been estimated using the elastic deformation moduli given in the table below, based on correlations with the measured SPT 'N' values.

Soil Deposit	Bulk Unit Weight	Elastic Modulus	Consolidation Parameters
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m <sup>3</sup>	--	--
Very stiff to hard clayey silt till	21 kN/m <sup>3</sup>	75 MPa	--

## 6.5 Detail Design and Construction Considerations

### 6.5.1 Additional Investigation Requirements

As noted previously, additional borehole investigation, laboratory testing and analysis will be required during detail design, once the layout of the proposed underpass foundation elements is finalized, to confirm the preliminary foundation recommendations presented herein, including founding elevations and subexcavation requirements, geotechnical resistances, settlement, and dewatering.

In particular, it is recommended to extend all boreholes to the shale bedrock to provide depth and elevation of the bedrock surface if consideration is given to extending the piles or caissons to bedrock. It is also recommended that further investigation of the Langstaff Road fill will be required during detail design to confirm whether the fill can be left in place.

### 6.5.2 Excavation

Depending on the foundation option adopted, excavations for the underpass foundations are expected to extend to depths of up to about 1.5 m below the proposed Highway 427 cut grade, and will be made through very stiff to hard clayey silt till to silty clay till, which is considered Type 3 soil according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation work should be carried out in accordance with the requirements of the OHSA, with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

### 6.5.3 Groundwater and Surface Water Control for Foundation Excavation

The groundwater level was measured in a standpipe piezometer at the site at about 6.9 m below ground surface. Therefore, it is expected that for spread footings founded on the very stiff to hard silty clay to clayey silt till the excavations for the west abutment will be about 5.4 m above the groundwater level. Some groundwater may be "perched" within the existing granular fill at the site, and therefore some water inflow should be expected into the foundation excavations, particularly during wet months; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavation.



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### 6.5.4 Subgrade Preparation

The soils exposed at the footing or pile cap subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a working mat of mass concrete be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade.

### 6.5.5 Obstructions During Pile Driving / Caisson Installation

It is anticipated that cobbles and/or boulders will be encountered within the till deposits, as noted in some of the boreholes at this site, and may affect the installation of steel H-piles and/or caissons. It is recommended that flange plate reinforcement or driving shoes be used on all steel H-Piles to facilitate driving into the very dense sand and silt till. In addition, as part of the detail design and contract preparation, it is recommended that consideration be given to including a Non-Standard Special Provision in the contract documents to warn the contractor of the possible presence of cobbles and/or boulders within the overburden soils.

## 7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Sandra McGaghran, P.Eng. and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

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### REFERENCES

Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06. 2006. CSA Special Publication, S6.1 06. Canadian Standard Association.

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

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

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

NAVFAC Design Manual DM 7.2. Soil Mechanics, Foundation and Earth Structures. U.S. Navy. 1982. Alexandria, Virginia.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, Second Edition, John Wiley and Sons, New York.



## PRELIMINARY FOUNDATION REPORT LANGSTAFF ROAD UNDERPASS - HIGHWAY 427 EXTENSION

**TABLE 1  
COMPARISON OF FOUNDATION ALTERNATIVES – LANGSTAFF ROAD UNDERPASS STRUCTURE  
W.O. 05-20012**

Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on very stiff to hard silty clay to clayey silt till	Feasible for support of piers	<ul style="list-style-type: none"> <li>Relative ease of construction; and,</li> <li>Negligible post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Some differential settlement possible between pier supported on shallow foundations and abutments supported on deep foundations;</li> <li>Groundwater control required (can be controlled by pumping from sumps depending on the time of year); and,</li> <li>Lowest bearing capacities of the four options.</li> </ul>	<ul style="list-style-type: none"> <li>Lower relative cost than piled foundations</li> </ul>	<ul style="list-style-type: none"> <li>Low risk of up to about 10 mm to 15 mm of differential settlement between abutments and centre pier; and,</li> <li>Moderate risk of softening of subgrade soil due to ponded water, but can be mitigated with concrete working mat</li> </ul>
Spread Footings "perched" in Approach Embankment Fill	Feasible for support of abutments	<ul style="list-style-type: none"> <li>Negligible post-construction settlement; and,</li> <li>Footing subgrade will not be disturbed by groundwater</li> </ul>		<ul style="list-style-type: none"> <li>Low cost option</li> </ul>	<ul style="list-style-type: none"> <li>Must ensure proper compaction of Granular 'A' pad to minimise post-construction settlement.</li> </ul>
Steel H-pile Foundations driven to found within hard silty clay to clayey silt till	Feasible for support of west abutment and centre pier	<ul style="list-style-type: none"> <li>Sub-excavation is not required,</li> <li>Higher bearing capacity,</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Downdrag loading must be taken into account in the design of piled foundations at the abutments; and</li> <li>Piles may encounter obstructions (cobbles and boulders) during driving</li> </ul>	<ul style="list-style-type: none"> <li>More costly than spread footings; and</li> <li>Installation costs could be impacted by presence of obstructions</li> </ul>	<ul style="list-style-type: none"> <li>Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements; and,</li> <li>Low to moderate risk of encountering obstructions that could impact pile installation</li> </ul>
Steel H-pile foundations driven to found on shale bedrock	Feasible for support of east abutment	<ul style="list-style-type: none"> <li>Sub-excavation is not required,</li> <li>Higher bearing capacity compared with piles driven to found in hard silty clay to clayey silt till,</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Downdrag loading must be taken into account in the design of piled foundations at the abutments; and</li> <li>Piles may encounter obstructions (cobbles and boulders) during driving</li> </ul>	<ul style="list-style-type: none"> <li>More costly than spread footings; and,</li> <li>Installation costs could be impacted by presence of obstructions</li> </ul>	<ul style="list-style-type: none"> <li>Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements; and,</li> <li>Low to moderate risk of encountering obstructions</li> </ul>
Caissons Foundations founded on hard silty clay to clayey silt till	Feasible at the piers and abutments	<ul style="list-style-type: none"> <li>Sub-excavation is not required</li> <li>Highest bearing capacity of the four options,</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or semi-integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Caissons could encounter obstructions (cobbles and boulders) during installation</li> <li>Need for temporary or permanent liners;</li> <li>Cleaning of the base below the water table could be difficult; and</li> <li>Caissons may encounter obstructions (cobbles and boulders) during installation</li> </ul>	<ul style="list-style-type: none"> <li>More costly option than steel H-piles</li> <li>Installation cost could be impacted by presence of cobbles and boulders</li> </ul>	<ul style="list-style-type: none"> <li>Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements</li> <li>Low to moderate risk of disturbance of water-bearing sand and silt till soils, requiring special construction procedures including use of temporary or permanent liners; and,</li> <li>Low to moderate risk of encountering obstructions that could impact caisson installation</li> </ul>
Caissons founded on shale bedrock.	Feasible at the east abutments.	<ul style="list-style-type: none"> <li>Sub-excavation is not required;</li> <li>Highest bearing capacity compared with caissons founded in hard clayey silt till;</li> <li>Negligible post-construction settlement;</li> <li>Can be used for support of conventional or semi-integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Need for temporary or permanent liners during installation through water-bearing sand and silt till to shale bedrock; and,</li> <li>Caissons may encounter obstructions (cobbles and boulders) during installation.</li> </ul>	<ul style="list-style-type: none"> <li>More costly than spread footings or steel H-piles; and,</li> <li>Additional cost associated with specialised drilling equipment and temporary or permanent liners.</li> </ul>	<ul style="list-style-type: none"> <li>Low to moderate risk of disturbance of water-bearing sand and silt till soils, requiring special construction procedures including use of temporary or permanent liners; and,</li> <li>Low to moderate risk of encountering obstructions that could impact caisson installation.</li> </ul>



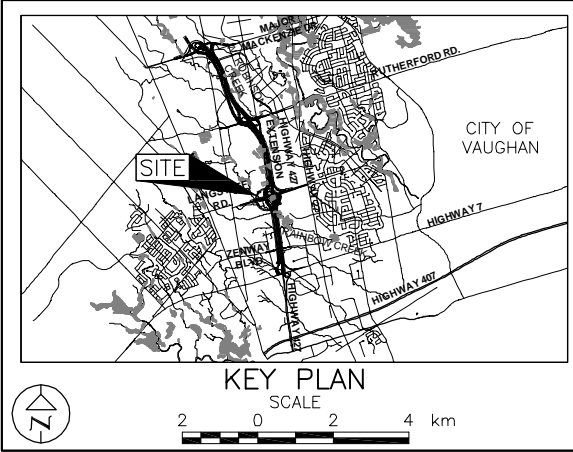
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WO No. 05-20012

HIGHWAY 427 EXTENSION  
LANGSTAFF ROAD UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



- LEGEND**
- Borehole - Current Investigation
  - Seal
  - Piezometer
  - N Standard Penetration Test Value
  - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
  - WL in piezometer, measured on May 13, 2009
  - WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
S12	187.5	4849869.7	293699.8
S13	187.7	4849885.0	293730.1
S14	187.7	4849893.4	293775.6
S14A	187.7	4849893.4	293776.6

**NOTES**

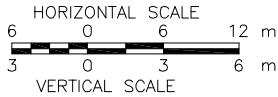
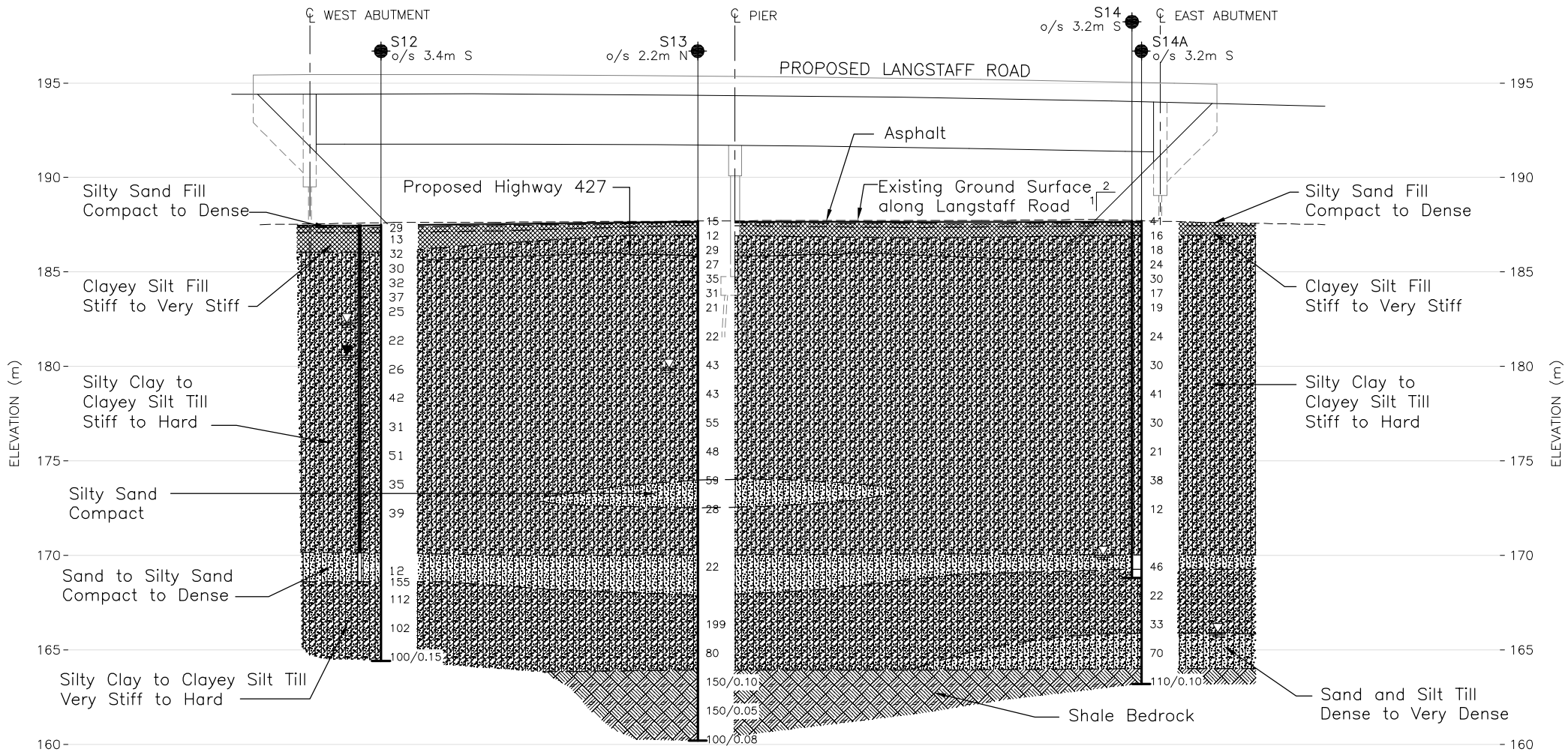
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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

The complete Preliminary 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**

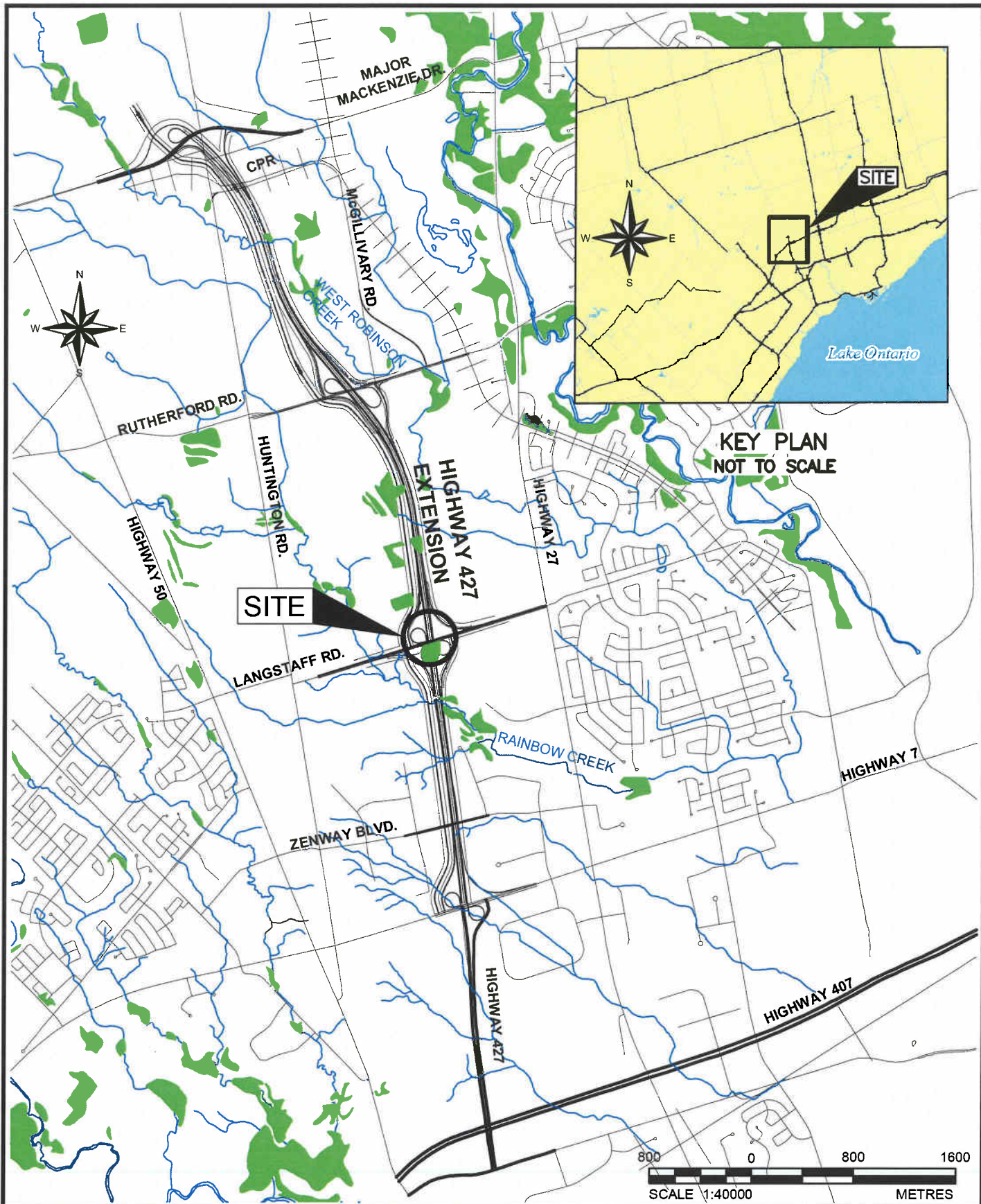
Base plans provided in digital format by MRC, (Drawing file "langstaff\_ga.dwg", received May 15, 2009).



NO.	DATE	BY	REVISION
Geocres No. 30M13-169			
HWY. 427	PROJECT NO. 06-1111-0012-7		DIST.
SUBM'D. CR/JEB	CHKD. SMM	DATE: 5-Aug-2009	SITE:
DRAWN: JFC/JM	CHKD. SMM	APPD. LCC	DWG. 1



PLOT DATE: August 5, 2009  
 FILENAME: T:\Projects\2006\06-1111-012 (MRC, Vaughan)\-GB- (LANGSTAFF ROAD)\061111012GB0F1.dwg



SCALE	AS SHOWN
DATE	Aug. 5, 2009
DESIGN	PKS
CAD	JFC
CHECK	SMM
REVIEW	LCC

TITLE

## SITE LOCATION PLAN LANGSTAFF ROAD UNDERPASS

FILE No. 061111012GB0F1.dwg

PROJECT No. 06-1111-012-7

REV. B

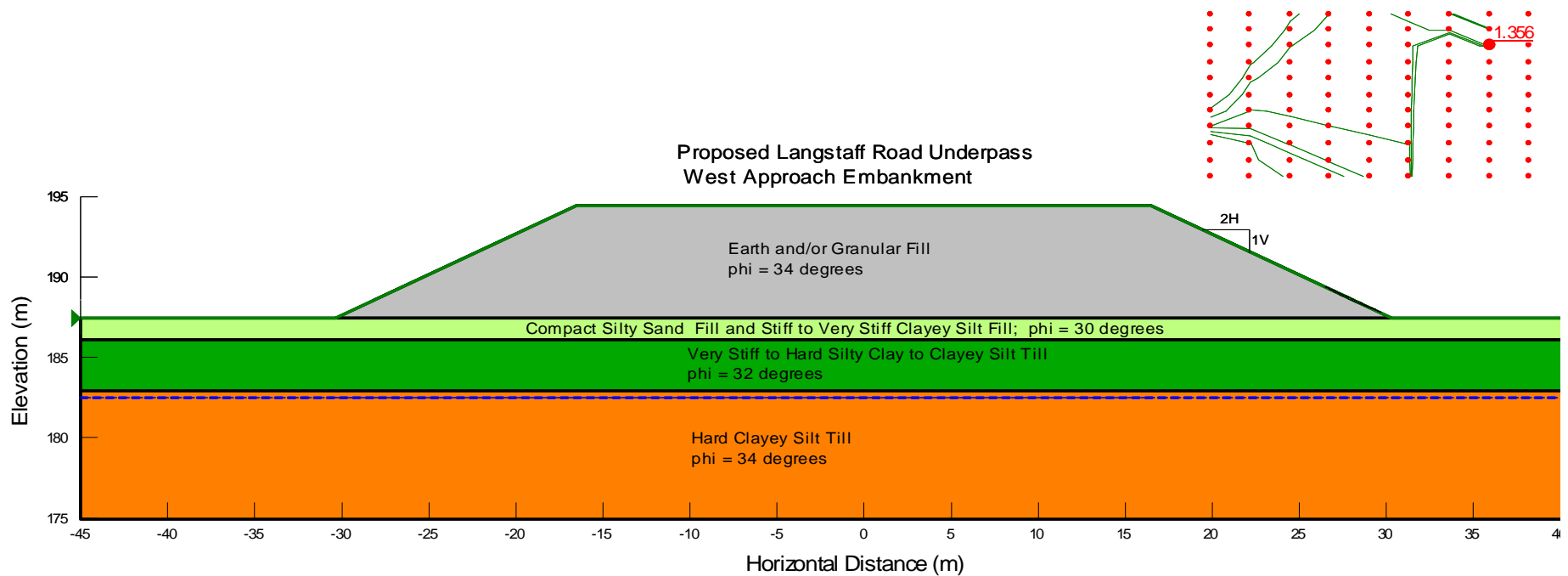
HIGHWAY 427 EXTENSION

FIGURE

1

**HIGHWAY 427 EXTENSION - LANGSTAFF ROAD UNDERPASS  
WEST APPROACH EMBANKMENT- STATIC GLOBAL STABILITY**

**FIGURE 2**



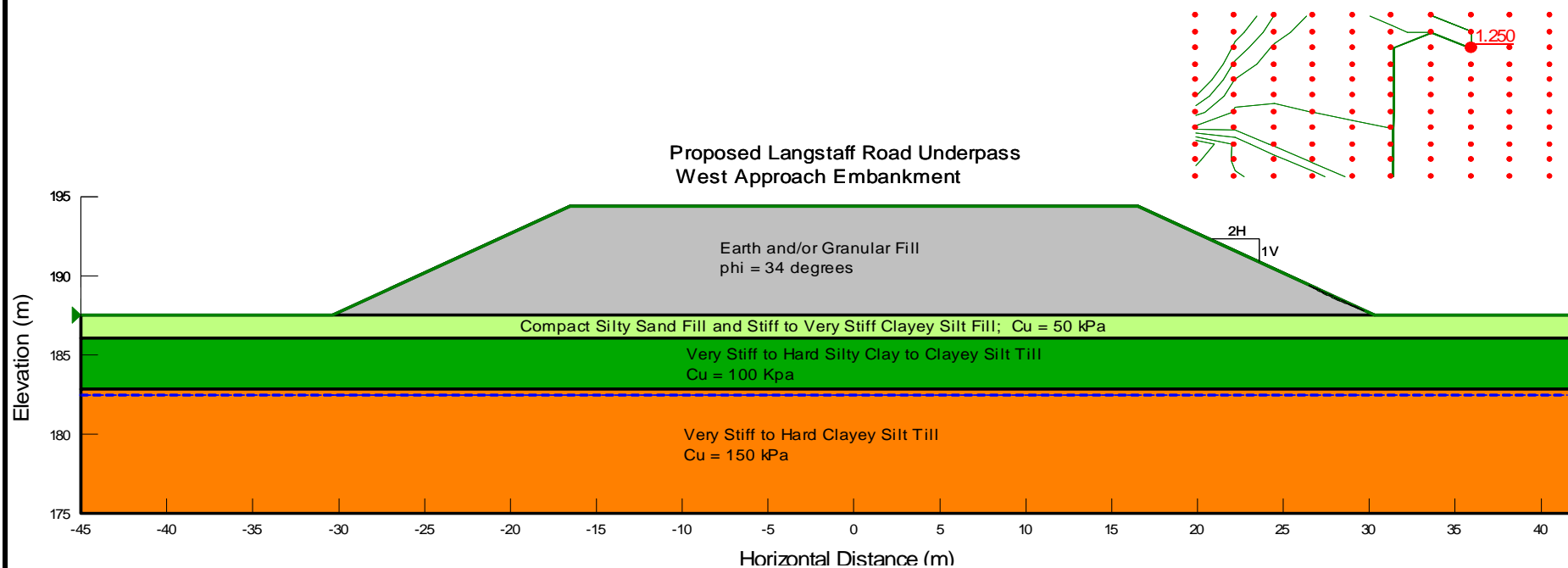
Date: June 2009  
Project: 06-1111-012-7

**Golder Associates**

Drawn: SMM  
Checked: LCC

**HIGHWAY 427 EXTENSION - LANGSTAFF ROAD UNDERPASS  
WEST APPROACH EMBANKMENT- SEISMIC GLOBAL STABILITY**

**FIGURE 3**



Date: June 2009  
Project: 06-1111-012-7

**Golder Associates**

Drawn: SMM  
Checked: LCC



**PRELIMINARY FOUNDATION REPORT  
LANGSTAFF ROAD UNDERPASS - HIGHWAY 427 EXTENSION**

# **APPENDIX A**

## **Record of Boreholes**



## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - \mu$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
$\mu$	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
$\gamma'$	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  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by 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
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

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

Notes: 1  $\tau = c' + \sigma' \tan \phi'$   
2 shear strength = (compressive strength)/2





## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

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

### III. SOIL DESCRIPTION

#### (a) 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

#### (b) Cohesive Soils Consistency

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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)
$\gamma$	unit weight

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

PROJECT 06-1111-012

**RECORD OF BOREHOLE No S12**

1 OF 2 **METRIC**

W.O. 05-20012

LOCATION N 4849869.7 : E 293699.8

ORIGINATED BY CR

DIST Central HWY 427

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY PKS/VA

DATUM Geodetic

DATE March 26, 2009

CHECKED BY SMN *SMN*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED						
187.5	GROUND SURFACE						20 40 60 80 100		10 20 30						
0.0	ASPHALT														
0.4	Silty sand, some gravel (FILL) Compact Brown to grey Moist		1	SS	29					○					
	Clayey silt, trace gravel, trace sand, containing rootlets (FILL) Stiff to very stiff Brown Moist		2	SS	13						○				
186.1															
1.5	SILTY CLAY, trace gravel, trace sand (TILL) Hard Brown Moist		3	SS	32					○	—				
			4	SS	30					○					
			5	SS	32					○					
			6	SS	37					○					
182.9	CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL) Very stiff to hard Grey Moist		7	SS	25					○					
4.6															
			8	SS	22					○				1 18 58 23	
			9	SS	26					○	—				
			10	SS	42					○					
			11	SS	31					○					
	Augers grinding at 11.4 m depth		12	SS	51					○					
			13	SS	35						—				

Augers grinding at 11.4 m depth

Continued Next Page

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

PROJECT <u>06-1111-012</u>		<b>RECORD OF BOREHOLE No S12</b>		2 OF 2 <b>METRIC</b>	
W.O. <u>05-20012</u>	LOCATION <u>N 4849869.7 ; E 293699.8</u>	ORIGINATED BY <u>CR</u>			
DIST <u>Central</u> HWY <u>427</u>	BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>	COMPILED BY <u>PKS/VA</u>			
DATUM <u>Geodetic</u>	DATE <u>March 26, 2009</u>	CHECKED BY <u>SMM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
	— CONTINUED FROM PREVIOUS PAGE —							● QUICK TRIAXIAL	× REMOULDED						
	CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL) Very stiff to hard Grey Moist		14	SS	39		172								
							171								
170.1							170								
17.4	SAND, trace gravel, trace silt, trace clay Compact Grey Wet Augers grinding between 17.6 m and 18.0 m depth		15	SS	12		169							2 91 4 3	
168.6							168								
18.9	SILTY CLAY, some sand, trace gravel (TILL) Hard Grey Wet		16	SS	155		167								
			17	SS	112										
166.6							166								
20.9	CLAYEY SILT, trace gravel, trace sand, containing cobbles (TILL) Hard Grey Wet  Augers grinding between 22.1 m and 22.5 m depth		18	SS	102		165								
164.4			19	SS	00/0.1										
23.1	END OF BOREHOLE														
	NOTES:  1. A 50 mm diameter monitoring well was installed at a depth of 18.9 m (Elev. 168.6 m).  Water level measurements  Date            Depth    Elev.  On Completion    5.2 m    182.3 m May 13, 2009    6.9 m    180.6 m June 15, 2009    6.7 m    180.8 m July 09, 2009    6.3 m    181.3 m  2. At 18.3 m depth (Elev. 169.2 m) 1.2 m of sand was up inside the augers during drilling due to "blowing" sands.														

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

PROJECT 06-1111-012

**RECORD OF BOREHOLE No S13**

1 OF 3 **METRIC**

W.O. 05-20012

LOCATION N 4849885.0 ; E 293730.1

ORIGINATED BY CR

DIST Central HWY 427

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY PKS/VA

DATUM Geodetic

DATE March 30 &amp; 31, 2009



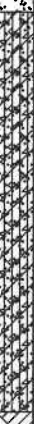

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
187.7	GROUND SURFACE													
-0.6	ASPHALT		1	SS	15		187							
0.2	Silty sand, some gravel (FILL)													
186.9	Compact Brown Moist		2	SS	12		186							
0.8	Clayey silt, some sand, trace gravel (FILL)													
	Very stiff Brown Moist		3	SS	29		185							
	SILTY CLAY, trace sand, trace gravel (TILL)													
	Stiff to hard Brown Moist		4	SS	27		184							
			5	SS	35		183							
			6	SS	31		182							
183.1	CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL)		7	SS	21		181							
4.6	Very stiff to hard Grey Moist						180							
	Augers grinding at 5.2 m depth		8	SS	22		179							
			9	SS	43		178							
			10	SS	43		177							
			11	SS	55		176							
			12	SS	48		175							
174.0	Silty SAND, trace gravel						174							
13.7	Very dense Grey Wet		13	SS	59		173							

Continued Next Page

+ 3, X 3; Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>06-1111-012</u>		<b>RECORD OF BOREHOLE No S13</b>		2 OF 3 <b>METRIC</b>	
W.O. <u>05-20012</u>		LOCATION <u>N 4849885.0 : E 293730.1</u>		ORIGINATED BY <u>CR</u>	
DIST <u>Central</u> HWY <u>427</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>		COMPILED BY <u>PKS/VA</u>	
DATUM <u>Geodetic</u>		DATE <u>March 30 &amp; 31, 2009</u>		CHECKED BY <u>SMM</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
172.5	15.2		14	SS	28									
	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Wet													
170.0	17.7			15	SS	22								
	SAND, trace to some silt, trace gravel Compact Grey Wet													
167.9	19.8													
	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Wet													
	Augers grinding at 21.0 m depth													
	Augers grinding at 22.0 m depth													
			16	SS	199									
			17	SS	80									
163.9	23.8		18	SS	50/0.10									
	SHALE (BEDROCK) Grey													
			19	SS	50/0.05									
			20	SS	00/0.00									
160.2	27.5													
	END OF BOREHOLE													

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE



+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div><div></div><div>20 40 60 80 100</div></div>	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
187.7	GROUND SURFACE												
-0.0	ASPHALT												
0.3	Silty sand, some gravel (FILL)		1	SS	41								
186.9	Dense Brown Moist		2	SS	16								
0.8	Clayey silt, some sand, trace gravel (FILL)		3	SS	18								
	Hard Brown Moist		4	SS	24								
	CLAYEY SILT, some sand, trace gravel (TILL)		5	SS	30								
	Stiff to hard Brown Moist		6	SS	17								
			7	SS	19								
			8	SS	24								
			9	SS	30								
			10	SS	41								
			11	SS	30								
			12	SS	21								
			13	SS	38								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

PROJECT 06-1111-012		<b>RECORD OF BOREHOLE No S14</b>		2 OF 2 <b>METRIC</b>	
W.O. 05-20012		LOCATION N 4849893.4 :E 293775.6		ORIGINATED BY CR	
DIST Central HWY 427		BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY PKS/VA	
DATUM Geodetic		DATE April 2, 2009		CHECKED BY SMM <i>SM</i>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
— CONTINUED FROM PREVIOUS PAGE —								○ UNCONFINED    + FIELD VANE	● QUICK TRIAXIAL    × REMOULDED						
	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Brown Moist		14	SS	12		172								
								171							
170.0								170							
17.7	Silty SAND, trace to some silt, trace gravel, trace clay Dense Grey Wet														
169.3															
18.4	SILTY CLAY, some sand, trace gravel (TILL) Hard Grey Wet		15	SS	46		169								
168.8															
18.9	END OF BOREHOLE														
NOTES:  1. Water level in open borehole at a depth of 17.7 m below ground surface (Elev. 170.0 m) upon completion of drilling.  2. An additional borehole was drilled adjacent to Borehole S14; See Record of Borehole S14A for details.  3. Borehole backfilled with bentonite.															



PROJECT <u>06-1111-012</u>		<b>RECORD OF BOREHOLE No S14A</b>		1 OF 2 <b>METRIC</b>	
W.O. <u>05-20012</u>	LOCATION <u>N 4849893.4 ; E 293776.6</u>	ORIGINATED BY <u>JEB</u>			
DIST <u>Central</u> HWY <u>427</u>	BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>	COMPILED BY <u>PKS/VA</u>			
DATUM <u>Geodetic</u>	DATE <u>April 13, 2009</u>	CHECKED BY <u>SMM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		
								○ UNCONFINED + FIELD VANE	WATER CONTENT (%)					
187.7 0.0	GROUND SURFACE						20 40 60 80 100	● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	10 20 30				
	Unsampled, see Record of Borehole S14 for stratigraphy above a depth of 19.8 m.						187							
							186							
							185							
							184							
							183							
							182							
							181							
							180							
							179							
							178							
							177							
							176							
							175							
						174								
						173								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



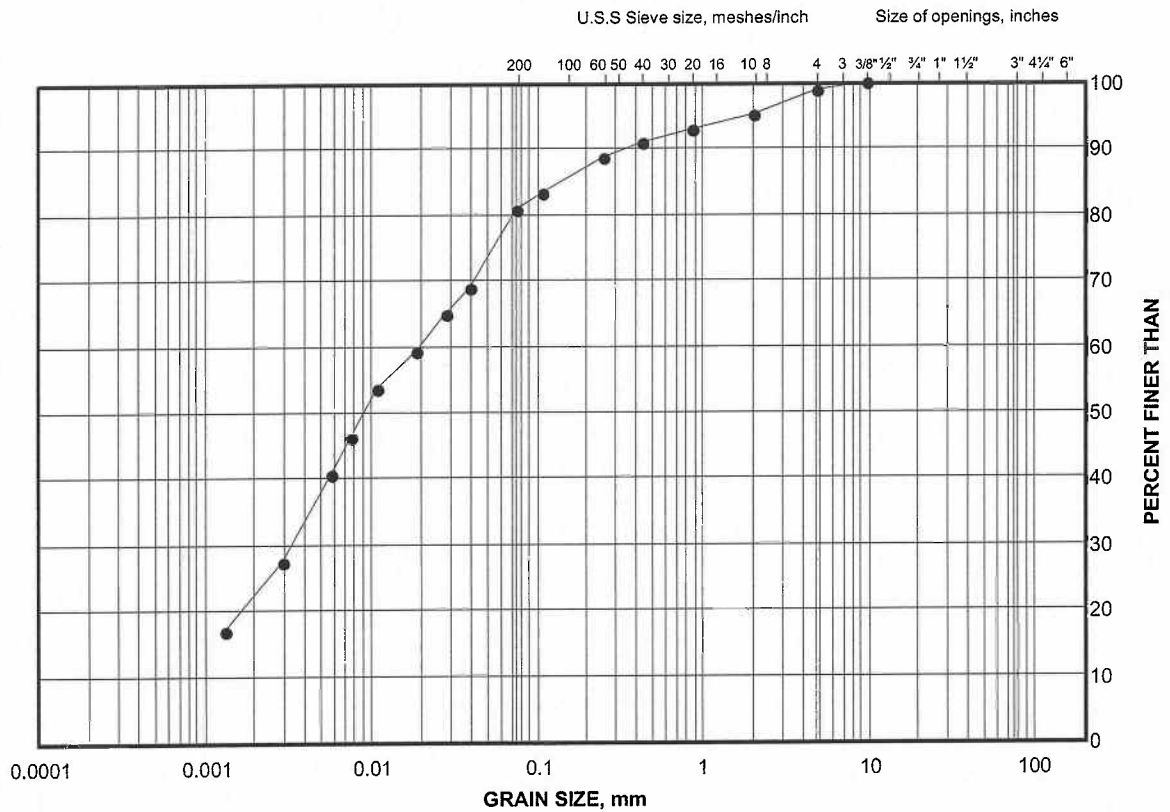
# **APPENDIX B**

## **Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Till

FIGURE B1



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

## LEGEND

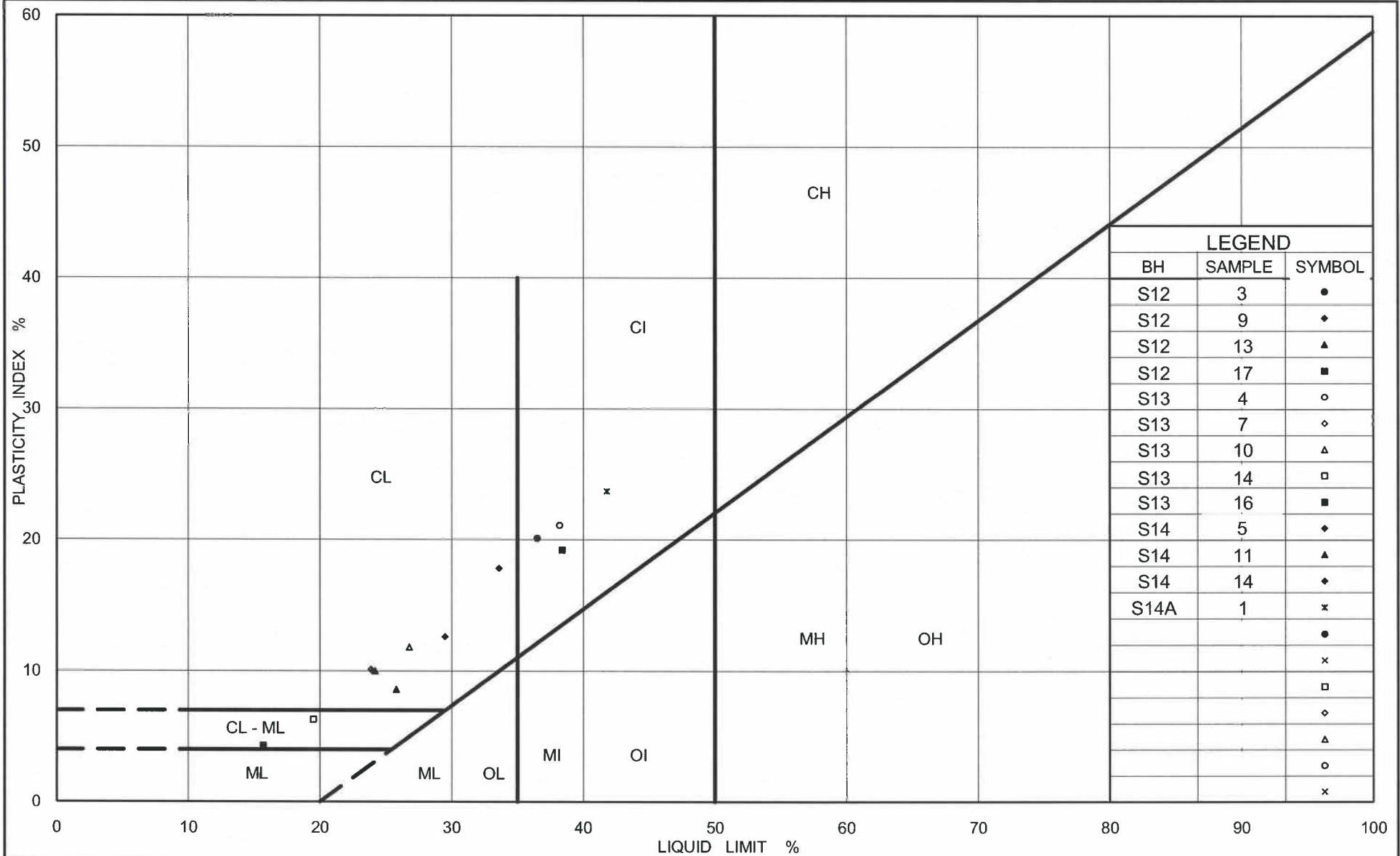
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S12	8	181.1

Project Number: 06-1111-012-7

Checked By: SM

Golder Associates

Date: 08-Jun-09



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## PLASTICITY CHART

### Silty Clay Till to Clayey Silt Till

Figure No. B2

Project No. 06-1111-012-7

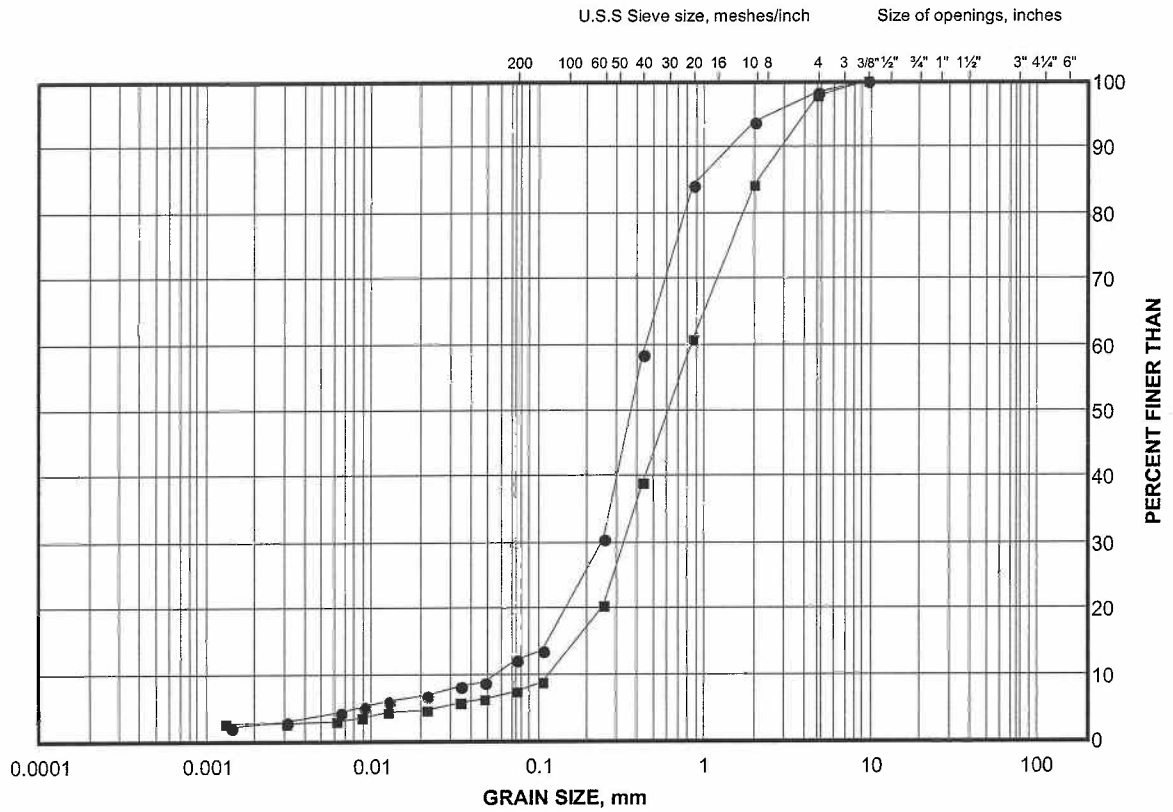
Checked By:

*SM*

# GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand

FIGURE B3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S13	15	153.9
■	S12	15	168.9

Project Number: 06-1111-012-7

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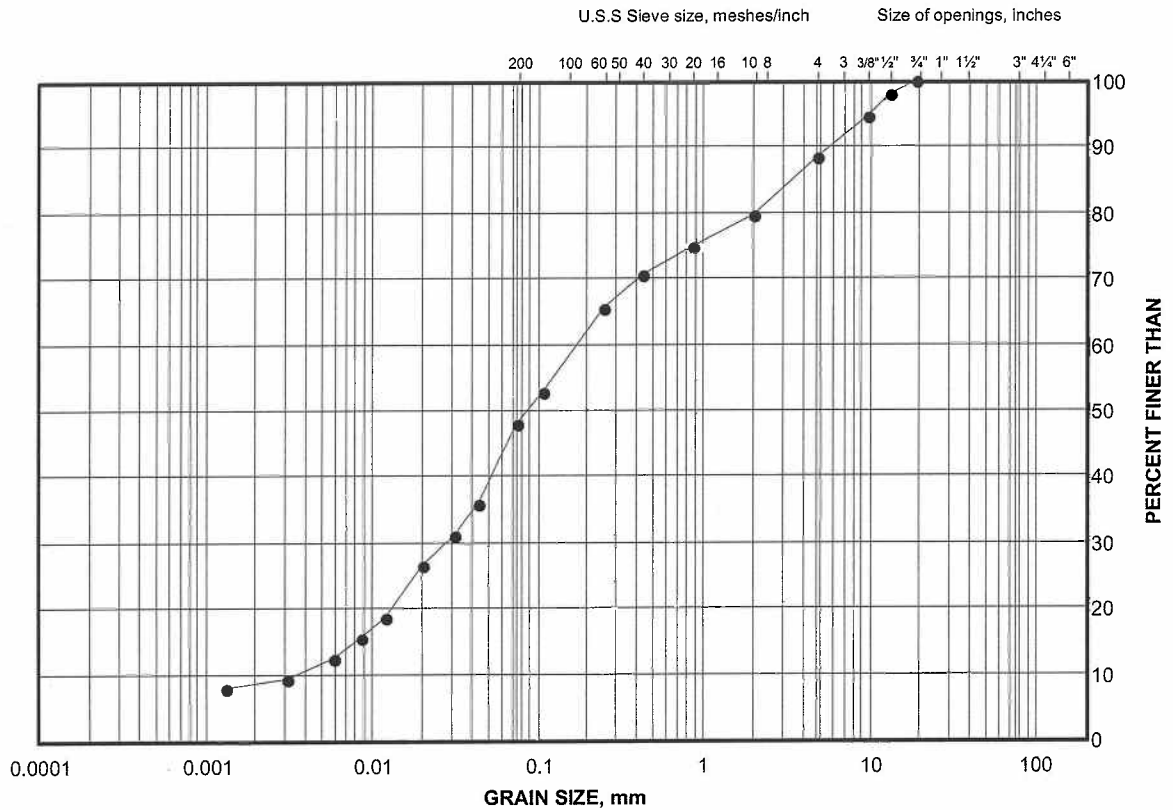
Date: 08-Jun-09



# GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Silt Till

FIGURE B4



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S14A	3	164.5

Project Number: 06-1111-012-7

Checked By: sm

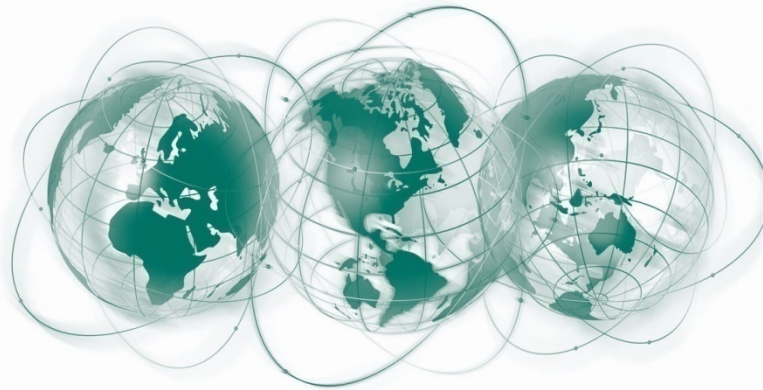
Golder Associates

Date: 08-Jun-09

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