



August 2009

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Relocation of Pier 4
Main Street Underpass (Site 36-0043)
Highway 403 Bridge Rehabilitations
From Highway 6 Westerly to Aberdeen Avenue
City of Hamilton, Ontario
GWP 2172-06-00, Agreement No. 2006-E-0081
Ministry of Transportation - Central Region**

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REPORT



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FOUNDATION INVESTIGATION AND DESIGN REPORT RELOCATION OF PIER 4

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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

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APPENDIX A

Records of Previous Boreholes (Geocres No. 30M5-29)

APPENDIX B

Records of Previous Boreholes (Geocres No. 30M5-269)



**FOUNDATION INVESTIGATION AND DESIGN REPORT
RELOCATION OF PIER 4**

PART A

FOUNDATION INVESTIGATION REPORT

**RELOCATION OF PIER 4
MAIN STREET UNDERPASS (SITE 36-0043)
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00, AGREEMENT NO. 2006-E-0081
MINISTRY OF TRANSPORTATION - CENTRAL REGION**



FOUNDATION INVESTIGATION AND DESIGN REPORT RELOCATION OF PIER 4

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Morrison Hershfield Limited (Morrison Hershfield) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out additional foundation engineering services as part of the detail design work for GWP 2172-06-00. The main focus of the project is the rehabilitation of eight underpasses. This report describes the subsurface conditions at the proposed new location of Pier 4 at the Highway 403/Main Street underpass (Site 36-0043).

The purpose of the foundation investigation is to determine the subsurface conditions at the locations of the proposed works by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and Golder Associates' letters 08-1132-013-1-L01 and L02, dated March 12, 2009 and March 18, 2009, respectively. The work was carried out in accordance with our Project Specific Supplementary Specialty Plan for Foundations Engineering Specialty dated January 28, 2008.

Morrison Hershfield provided Golder Associates with preliminary drawings for this project in digital format.



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2.0 SITE DESCRIPTION

The overall project limits for GWP 2172-06-00 extend from the Highway 6 (North) Interchange westerly to approximately 1 kilometre west of the Aberdeen Avenue Interchange. The site is situated in Hamilton, Ontario. The site location is shown on the Key Plan, Figure 1.

Highway 403 at the Main Street underpass is a divided highway with two lanes in each direction and is co-signed with Highway 6 within the project limits. This section of Highway 403 traverses an urban section of the City of Hamilton. Highway 403 was constructed in the 1960s through the Chedoke Creek Valley. The Chedoke Creek Valley is approximately 150 metres wide at the structure location. In order to accommodate Highway 403, Chedoke Creek was realigned and segments channelized or rerouted through culverts. Ground surface elevations within the project area vary between 83.8 metres west of the structure and 91.4 metres east of the structure.

The Main Street Underpass site is bordered by a residential area to the northwest, Cathedral Park to the northeast and commercial and industrial properties to the south. The five span Main Street underpass structure conveys eastbound traffic on Main Street West (Highway 8) over the Chedoke Creek Valley, Ramp 'H' and the Highway 403 main lanes.

2.1 Site Geology

The project area is situated in the physiographic region of southern Ontario known as the Iroquois Plain¹. In the Lake Ontario lakehead region, the Iroquois sand plain exists as a narrow plain between Lake Ontario and the Niagara escarpment or locally known in Hamilton as the "mountain". The Iroquois Plain represents the lake bottom of former Lake Iroquois.

The surficial soils along Highway 403 in the Aberdeen Avenue Interchange area are primarily composed of lacustrine and outwash sands.² Paleozoic shale and dolomite outcrops immediately southwest of the Aberdeen Avenue Interchange.

The surface of the bedrock is reported to be at approximately elevation 60 metres in this area.³ The bedrock is reported to be red shale and mudstone with minor interbeds of silty limestone and dolomite of the Queenston Formation.⁴ The Queenston shale is irregularly interlayered with occasional beds or pockets of olive green calcareous siltstone. A 1960 study conducted by the Department of Highways found that the top 3 to 4 metres is weathered.⁵ The shale is highly fissile, susceptible to weathering under certain conditions and breaks easily parallel to the bedding planes.

¹ L.J. Chapman and D.F. Putnam: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2, 1984.

² Karrow, P.F., 1987: Quaternary Geology of the Hamilton Area, Southern Ontario. Ontario Geological Survey Map 2509 (Revised). Scale 1:50,000.

³ Karrow, P.F., 1958: Bedrock Topography of the Hamilton Area, Southern Ontario. Ontario Geological Survey Map 2034. Scale 1:63,360.

⁴ Karrow, P.F., 1969: Bedrock Geology Toronto-Windsor Area. Geological Survey of Canada Map 1263 A. Scale 1:250,000.

⁵ Geotitles Report No. (30MS-95) by Department of Highways, Ontario entitled "Engineering Study, Properties of Queenston Shale, Proposed Chedoke Expressway, Hamilton Area, Ontario" dated August 19, 1980.



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3.0 INVESTIGATION PROCEDURES

This report utilizes information from the original geotechnical investigation for the Highway 403 Main Street overpass structure (Geocres No. 30M5-29), our 2008 investigation for temporary bridge deck support structures for this project (Geocres No. 30M5-269) and a 2009 investigation that was carried out concurrently with fieldwork for the staging area and high fill detour for the Aberdeen Avenue underpass rapid bridge replacement. Two boreholes, numbered 112 and 113, were drilled to depths of 3.0 to 5.0 metres, respectively, on May 13, 2009.

The boreholes at Main Street were advanced using a hand auger due to access and headroom constraints that precluded use of conventional drill rigs. Penetration testing was carried out at regular intervals of depth with a 38 millimetre inside diameter split spoon sampler and a non-standard 31.7 kilogram manual hammer. The driving resistances have been interpreted to the approximate N values shown on the Records of Boreholes.

Groundwater conditions in the boreholes were observed during drilling. All of the boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 372/07.

The field work was supervised on a full-time basis by experienced members of our engineering staff who arranged for utility locates, directed the drilling, sampling and in-situ testing operations, logged the boreholes, cared for the samples obtained and surveyed the borehole elevations. The borehole elevations are referenced to benchmarks provided by Morrison Hershfield. It is understood that these elevations are referenced to geodetic datum.

The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' London laboratory for further examination and testing. The results of the field and laboratory testing are given on the Record of Borehole sheets.

The table below summarizes the locations, ground surface elevations and depths of the current boreholes:

Borehole	Location (m)		Ground Surface Elevation (m)	Depth (m)
	Northing	Easting		
112	4 791 249	272 825	87.68	3.05
113	4 791 271	272 843	87.71	5.03

Information from the current boreholes was supplemented with boreholes from previous geotechnical investigations conducted by others for the original design of Highway 403 (Geocres No. 30M5-29) and by Golder Associates for the design of temporary bridge deck support structures (Geocres No. 30M5-269). The selected boreholes and reports are:

- Records of Boreholes 1 and 4 from Geocres Report No. 30M5-29 entitled "Foundation Report on Hwy. 403 (Chedoke Expressway) and Main St. Crossing in Hamilton, Dist. 4., W.J. F-59-116, W.P. 180-60" dated May 26, 1960.
- Records of Boreholes 49 and 50 from Geocres Report No. 30M5-269, entitled "Foundation Investigation and Design Report, Temporary Bridge Support Structures, Highway 403 Rehabilitation from Highway 6 Westerly to Aberdeen Avenue, City of Hamilton, Ontario, Agreement No. 2006-E-0081, MTO Central Region", dated March 12, 2009.

The Record of Borehole sheets for the previous boreholes and associated Summary of Field and Laboratory



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Test sheets for boreholes 1 and 4 (30M5-29) are presented in Appendix A in their original format. The Record of Borehole sheets for boreholes 49 and 50 (Geocres 30M5-269) are provided in Appendix A.

The table below summarizes the locations, ground surface elevations and depths of the previous boreholes:

Borehole	Location (m)		Ground Surface Elevation (m)	Depth (m)
	Northing	Easting		
Geocres No. 30M5-269				
49	4 791 248	272 816	85.01	6.55
50	4 791 271	272 833	84.95	6.55
Geocres No. 30M5-29				
1	4 791 250	272 822	85.50	31.03
4	4 791 267	272 833	85.31	30.57

The locations of the current and previous boreholes are shown in plan on Drawing 1 and are noted on the Record of Borehole sheets. The locations of the previous boreholes should be considered approximate since the locations were referenced to imperial chainages and offsets rather than metric MTM coordinates.



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4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the current boreholes, together with the results of the in situ and laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report. The Record of Borehole sheets and results of laboratory testing from the previous boreholes are presented in Appendices A and B. The stratigraphic boundaries shown on the Record of Borehole sheets and stratigraphic profiles are inferred from non-continuous sampling and observations of drilling resistance and represent transitions between soil types rather than exact planes of geological change. Subsurface conditions will vary between and beyond the borehole locations. It should be noted that material described as silty clay in the previous boreholes (Geocres 30M5-29) has generally been inferred to be clayey silt based on the results of previous Atterberg limits testing and comparison with the stratigraphy of adjacent boreholes. Furthermore, fill deposits associated with the construction of footings/pile caps extending to the underside of these elements should be expected at all pier locations. In addition, remnants of former temporary construction works may be present adjacent to the piers. The extents of such works are difficult to determine through boreholes alone. However, their presence should be expected since several of the structures are located in areas of known refuse disposal and soft surficial soils. Temporary shoring may have been installed to support excavations such as pile caps and, given the time period in which these structures were erected, these works may have been left in place.

The locations of the boreholes and simplified stratigraphies are shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized below.

The stratigraphy generally consists of the pavement structure or surficial topsoil and/or fills overlying an extensive deposit of clayey silt that overlies shale bedrock.

4.2 Soil Conditions

4.2.1 Asphalt

Asphalt layers 150 and 230 millimetres thick were encountered at the ground surface in boreholes 49 and 50 (30M5-269).

4.2.2 Topsoil and Fill

Topsoil was encountered at the ground surface in boreholes 112 and 113. The topsoil layers were 90 and 120 millimetres thick.

Highly variable fill materials containing layers of silt, sand and gravel, sand, clayey silt and silty sand were found in all of the boreholes. The fill materials included refuse, slag, bricks, cinders and wood fragments. Fill was beneath the topsoil layers boreholes 112 and 113 from elevations 87.6 to 87.6 metres, below the asphalt in boreholes 49 and 50 (30M5-269) from elevations 84.7 to 84.9 metres and at the ground surface in boreholes 1 and 4 (30M5-29) to elevations 77.0 to 84.0 metres. The upper granular fill in boreholes 49 and 50 (30M5-269) typically consisted of granular materials associated with the shoulder paving of Highway 403.



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Standard penetration test N values in the fill ranged from 3 to 68 blows per 0.3 metres but were typically less than 12 blows per 0.3 metres. Water contents of the fill varied from 6 to 20 per cent. One Atterberg limits determination was carried out on a sample of fill from borehole 49. The results indicated that the fill at that particular location was of low plasticity with plastic and liquid limits of 14 and 24 per cent, respectively, and a plasticity index of 10 per cent.

4.2.3 Clayey Silt

Clayey silt was encountered beneath the fill layers in all boreholes except borehole 112 from elevation 76.8 and 84.0 metres. Borehole 113 encountered native soils at elevation 84.0 metres at the new pier location. This appears consistent with contours indicating the original ground surface shown on Department of Highways Ontario (DHO) Drawing No. 59-F-125A dated February 20, 1960, and the existing profiles shown on the profile drawing for Main Street prepared for Contract 62-109 and the Main Street Underpass general arrangement drawing circa 1962.

The upper clayey silt layers above elevation 65 metres had N values of 3 to 19 blows per 0.3 metres. In situ field shear vane tests conducted in softer zones indicated undrained shear strengths from 42 to over 144 kilopascals. The sensitivity of this soil varied from 1.6 to 2.4. Water contents of 15 to 35 per cent were measured in this zone. The upper clayey silt is of low plasticity with average plastic and liquid limits of 17 and 30 per cent, respectively, and an average plasticity index of 13 per cent.

Below elevation 65 metres, the clayey silt was very stiff to hard with N values of 20 to over 100 blows per 0.3 metres. The lower clayey silt is of low plasticity with average plastic and liquid limits of 17 and 28 per cent, respectively, and an average plasticity index of 11 per cent.

4.2.4 Shale Bedrock

Red and grey shale bedrock of the Queenston Formation was encountered in boreholes 1 and 4 (30M5-29) from elevation 57.5 to 57.8 metres.

Samples of rock core were obtained in boreholes 1 and 4 (30M5-29) using an AXT barrel. The reported recovery in borehole 4 (30M5-29) was 100 per cent. The core recovery in borehole 1 was not noted.

4.3 Groundwater Conditions

The groundwater conditions in the current boreholes were monitored during and upon completion of drilling. The observed groundwater conditions are noted on the Record of Borehole sheets, on the profiles and sections and are summarized in the following text. Groundwater levels are expected to fluctuate based on climatic and seasonal variations.

Groundwater was encountered in borehole 113 at elevation 84.4 metres. Borehole 112 was dry during and upon completion of drilling. Boreholes 49 and 50 (30M5-269) encountered groundwater during drilling at elevations 82.9 and 81.6 metres, respectively. Groundwater observations were not recorded in boreholes 1 and 4 (30M5-29).

The groundwater level at Pier 4 of the Main Street Underpass structure, based on the measured groundwater levels, change in colour of the native soils and encountered groundwater levels, has been inferred to be at elevation 84 metres.

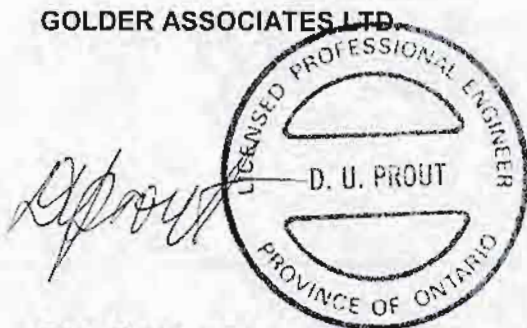


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5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Golder Associates. The field operations were supervised by Mr. Michael Arthur under the direction of Mr. David J. Mitchell. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Mr. Neil Charters and Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
RELOCATION OF PIER 4**

PART B

FOUNDATION DESIGN REPORT

**RELOCATION OF PIER 4
MAIN STREET UNDERPASS (SITE 36-0043)
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00, AGREEMENT NO. 2006-E-0081
MINISTRY OF TRANSPORTATION - CENTRAL REGION**



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6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the proposed relocation of Pier 4 at the Highway 403/Main Street Underpass and associated temporary earthworks based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. 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 and scheduling.

The underpass is a five span concrete bridge which will be rehabilitated and the current Pier 4 will be moved approximately 5 metres east to allow for future widening of Highway 403. The existing Pier 4 is supported on steel H-piles driven to bedrock at approximately elevation 58 metres. The previous report by Golder Associates (Geocres report number 30M5-269, dated March 2009) provides details related to the geotechnical aspects of the bridge rehabilitation.

The ground surface at the current Pier 4 location is approximately elevation 85 metres and the underside of the east abutment pile cap is at approximately 90 metres. The horizontal distance from the proposed new Pier 4 to the east abutment seat is approximately 8 metres. This will result in a temporary cut slope with an angle of approximately 1 horizontal to 1 vertical during construction of the new pile cap and a final cut slope angle of about 2 horizontal to 1 vertical after backfilling the pile cap excavation and reconstruction of the slope.

The subsoils encountered in the boreholes put down during the investigation typically consist of surficial layers of topsoil and fill (up to 8 metres at the current Pier 4 location and approximately 5 metres at the new Pier 4 location) overlying clayey silt approximately 20 metres thick which is underlain by shale bedrock at approximately elevation 58 metres.

The inferred groundwater level is approximately elevation 84 metres or some 1.0 metres below the Highway 403 pavement surface.

6.2 Conversion to Semi-Integral Abutments

It is understood that a conversion of the existing conventional abutments to semi-integral abutments is being contemplated. The abutments are founded on steel H 360 x 109 (14 BP 73) piles likely driven to bedrock. As-built details of the pile layout were not available but, based on the General Arrangement drawing for Contract 62-09, the abutment the pile layout in the abutment area features a mix of both vertical and battered piles. The existing foundations and subsurface conditions are considered to be compatible with a semi-integral abutment design.

6.3 Foundations

Shallow footings are not considered feasible for the new Pier 4 due to the presence or poor quality, highly variable fills 2 to 3 metres in depth underlain by moderately compressible clayey soils beneath the new Pier 4



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location. Excavations for the shallow footings will also extend below the groundwater level. The native clayey soils are likely to result in excessive total settlement beneath relatively heavily loaded shallow footings and will have different load-settlement characteristics from the existing bridge piers and abutments on pile foundations. For these reasons, deep foundations are the preferred foundation option.

The various foundation options considered for this site are compared in Table I. This table includes estimated foundation costs and summaries of the feasibility of each option. The costs given are only estimates provided to give an order of magnitude cost comparison between alternatives rather than absolute figures.

6.3.1 Deep Foundations

Steel H-piles may be driven either to the bedrock at approximately elevation 58 metres or driven as friction piles in the overlying very stiff clayey silt below elevation 65 metres. Both options are considered suitable for support of the structure although, as discussed in Table I, piles driven to bedrock are the preferred technical alternative as they will only be subject to minimal settlements and will provide significantly higher geotechnical axial resistances.

Geotechnical Axial Resistance – Driven Steel H-Piles

For design, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to refusal on the shale bedrock below elevation 58 metres may be taken as 1,200 kilonewtons (kN) per pile. Serviceability Limit States (SLS) loading will not govern as the bedrock can be taken to be unyielding and 25 millimetres of settlement will not occur. The contract drawings should have a pile note stating "Piles to be driven to bedrock".

Should the piles be designed as friction piles founded within the very stiff clayey silt at approximately elevation 65 metres, the factored axial geotechnical resistance at ULS may be taken as 320 kN per pile. The unfactored SLS axial resistance may be taken as 210 kN per pile. These resistances will increase as the pile length increases due to additional skin friction. The rate of increase will be approximately 30 kN per metre for ULS capacity and 20 kN per metre for SLS capacity. Due to the presence of the clayey soils, at least one pile load test should be conducted on the friction pile no sooner than 15 days after installation.

Vertically driven piles should be equipped with Type I driving shoes in accordance with MTO current practices (Ontario Provincial Standard Drawing (OPSD) 3000.100 and SP903S01) and battered piles should be equipped with Type II driving shoes to ensure adequate seating into the bedrock. The steel H-piles should be installed and monitored in accordance with SP903S01.



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Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

$$k_s = \text{coefficient of horizontal subgrade reaction (MPa/m)} = \frac{67 S_u}{d} \quad \text{for cohesive soils}$$

d = pile width or diameter (m)

S_u = undrained shear strength of the soil (MPa)

Soil Type	Elevation (m)		S_u kPa
	From	To	
Fill (cohesive)	Surface	80	25 – 100
Firm clayey silt	80	65	50
Very stiff clayey silt	65	58	150

Group action for lateral loading should 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:

Pile Spacing in Direction of Loading, d = Pile Diameter	Subgrade Reaction Reduction Factor R^6
8d	1.00
6d	0.70
4d	0.40
3d	0.25

A maximum lateral resistance of 160 kN at ULS and 70 kN at SLS is recommended for HP310 x 110 piles.

Constructability

Construction of the foundations for the relocated Pier 4 will be challenging due to headroom and lateral space restrictions. Information provided by Morrison Hershfield indicates that at Highway 403 Station 13+439, the vertical clearances at the edge of asphalt in the outer shoulder of the eastbound lanes are 6.3 metres and 6.0 metres at the north and south faces of the structure, respectively. In order to install the piles, the contractor can either elect to install them through slots cut in the bridge deck using conventional pile driving equipment or install them from the shoulder of Highway 403 using pile driving equipment specially adapted for low headroom conditions.

The former option would require closing at least one lane of Main Street during the duration of the work and provision of the temporary patches. It is likely that this work would only be allowed during a night time closure

⁶ Foundations and Earth Structures - Design Manual 7.2, NAVFAC DM-7.2, Department of the Navy, Naval Facilities Engineering Command (1982)



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and maybe subject to restrictions. This option is not feasible due to noise concerns and the structural adequacy of the deck to accommodate the resulting vibrations and loading.

The latter alternative is the preferred technical option. We have confirmed the existence of one Ontario based contractor with a low head drop pile driver. This machine requires only 5 metres of clearance and has the ability to drive 1.2 metres long pile sections before splicing is required. The estimated production rate is 10 to 12 metres per day. A ULS value of 1200 kilonewtons can be used for piles successfully achieving refusal in the underlying shale bedrock. The main drawback with such equipment is that the driving of battered piles is not feasible.

6.4 Seismic Design

The site is located in Hamilton in southwestern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.05. The corresponding acceleration related seismic zone, Z_a , is 0. The following seismic performance zones are applicable to the proposed structure based on the assigned importance category:

Importance Category	Seismic Performance Zone
Lifeline bridge	2
Emergency Route and other bridges	1

This bridge is not a designated lifeline bridge, and therefore a rigorous seismic analysis for earthquake loads is not required as it is in Seismic Performance Zone (SPZ) 1. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

6.4.1 Seismic Hazard Assessment

The site location has historically been considered to be in an area of low seismicity with peak ground acceleration (PGA) values of less than 0.05 g from an earthquake with a 10 per cent probability of exceedance in 50 years. The soil stratigraphy was screened to see if the composition and state of the soils was susceptible to liquefaction. Liquefaction is not considered to be a hazard due to the lack of granular soils below the groundwater table. Therefore, an evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundation and the effect of seismic forces on embankment stability and the bridge abutment is not considered warranted.

6.5 Excavations and Temporary Cut Slopes

Excavations will be required for the pile cap at the new Pier 4 location. The excavation will encounter surficial fill materials, although it may penetrate into the native clayey silt at the eastern side of the excavation. The groundwater level is expected to be near elevation 84 metres and will fluctuate with seasonal and climatic variations. The bottom metre of the excavation is expected to penetrate the groundwater level, although seepage volumes are likely to be low due to the fine grained nature of the fill and native soils. If necessary, groundwater control may be effected using properly filtered sumps located outside the foundation area. Surface



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water runoff should be directed away from the excavations at all times. The appropriate NSSP should be included in the contract documents.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils. The native clayey materials would be classified as Type 2 soils.

Based on the assumption of a 0.5 metre wide working space around the new pile cap at Pier 4, a 45 degree (1 horizontal to 1 vertical) slope between the base of the excavation and the base of the abutment pile cap is possible. Alternatively, there are a number of feasible retention options for this slope and they are discussed in Table II. The preferred technical solution is to implement a detailed monitoring program of the east abutment pile cap and foreslope during excavation and construction to identify any signs of movement and treat these as necessary. All retention options are likely to be expensive and require major works due to the height of the slope to be retained and the very limited working space beneath the bridge. Design parameters for the temporary shoring are discussed below in the event that implementation of a retention system becomes necessary.

Protection Systems

Protection systems may have to be provided to support the existing fills during construction in accordance with SP105S19. The shoring should be designed to Performance Level 2. The temporary protection system could consist of soil nails and a facing system, driven steel sheet piles or soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds. Longitudinal drains and weep holes, where required, should be installed to provide positive drainage of the retained soil.

The temporary protection systems may be designed using the following parameters:

Soil Type	Unfactored Coefficients of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Clayey Fill	0.36	0.53	2.78	28	20
Clayey Silt	0.30	0.50	3.00	30	20

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

Active earth pressures may be used in the geotechnical design of the structure if the wall is designed to allow outwards rotation of greater than 1 per cent of the wall height. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

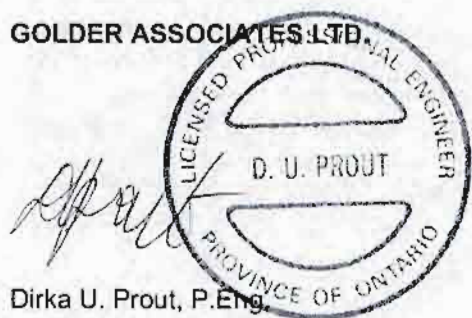


FOUNDATION INVESTIGATION AND DESIGN REPORT RELOCATION OF PIER 4

7.0 MISCELLANEOUS

This report was prepared by Mr. Neil Charters and Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

GOLDER ASSOCIATES LTD.



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NC/DUP/PRB/FJH/sll

n:\active\2008\1132 - geotechnical\1132-000-0108-1132-013-1 mh - addnl fdns - hwy 403 & 6\reports\0811320131-r03 - main st pier 4\0811320131-r03 aug 18 09 - part a&b - fdn rpt - relocation of pier 4 - hwy 403 and 6.docx

TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Relocation of Pier 4
Main Street Underpass, Site 36-0043
Highway 403 Bridge Rehabilitations
GWP 2172-06-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Spread footings supported on native soils	<ul style="list-style-type: none"> Not feasible due to depth of excavation required, limited geotechnical resistance and settlement concerns 	<ul style="list-style-type: none"> Less expensive than piled foundations 	<ul style="list-style-type: none"> Founding on shallow footings can potentially result in differing settlement behaviour from existing deep foundations Native clayey soils immediately underlying the fill have low shear strengths and are moderately compressible. As such, bearing resistance is limited. 	<ul style="list-style-type: none"> Least expensive foundation 	<ul style="list-style-type: none"> Shallow foundations may be adversely affected by settlement of underlying clayey deposits Shoring will likely be required due to depth of excavation High risk of excessive total and differential settlement
Friction steel H-pile foundations driven into native clayey silt soils	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Less settlement than spread footings founded on native soils 	<ul style="list-style-type: none"> Will not remove risk of excessive settlement Limited axial capacity More costly than shallow foundations Load testing of at least one pile required 	<ul style="list-style-type: none"> Slightly less expensive per pile than piling to shale bedrock though more piles required 	<ul style="list-style-type: none"> Low to moderate risk of excessive total and differential settlement May require more piles than if driven to bedrock if axial capacity is limiting factor in design

COMPARISON OF FOUNDATION ALTERNATIVES

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
End bearing steel H-pile foundations driven to refusal onto shale bedrock	<ul style="list-style-type: none"> Feasible for all loading conditions Preferred technical solution 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement Fewer piles required compared to friction piles 	<ul style="list-style-type: none"> Possibility of pile tip damage during driving in rock More costly than shallow footings 	<ul style="list-style-type: none"> Estimated costs of \$435/metre per steel pile 	<ul style="list-style-type: none"> Possible pile tip damage if piles are not adequately protected while driving in bedrock Negligible risk of excessive total and differential settlement

NOTES: 1. Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.

2. Table to be read in conjunction with accompanying report.

Prepared By: NC/DUP
Checked By: PRB

TABLE II

COMPARISON OF TEMPORARY SHORING ALTERNATIVES

Relocation of Pier 4
 Highway 403 - Main Street Underpass, Site 36-0043
 Highway 403 Bridge Rehabilitations
 GWP 2172-06-00

RETENTION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Temporary Cut Slope (1 horizontal to 1 vertical)	<ul style="list-style-type: none"> Suitable for short term construction provided adequate monitoring is in place 	<ul style="list-style-type: none"> Least expensive option 	<ul style="list-style-type: none"> Detailed monitoring regime required May need to implement further retention work should monitoring indicate movement of slope 	<ul style="list-style-type: none"> Least expensive solution 	<ul style="list-style-type: none"> Low to moderate risk of failure Excessive movement of embankment fill may trigger implementation of retention solution
Soil Nails	<ul style="list-style-type: none"> Suitable for short and long term construction 	<ul style="list-style-type: none"> May be less expensive than sheet piles or piled foundations More movement required before nails mobilized 	<ul style="list-style-type: none"> Will not remove risk of excessive movement of slope Possible space conflicts due to limited installation space under the bridge Will need full wall facing 	<ul style="list-style-type: none"> May be less expensive than piled options 	<ul style="list-style-type: none"> Low risk of excessive movement affecting the bridge abutment

COMPARISON OF RETENTION ALTERNATIVES

RETENTION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Sheet Piles	<ul style="list-style-type: none"> Feasible for temporary shoring; may need ground anchors for permanent solution 	<ul style="list-style-type: none"> Less risk of excessive movement of bridge abutment than soil nail option 	<ul style="list-style-type: none"> Limited headroom will require special installation procedures Will probably need ground anchors due to 4 m retained height 	<ul style="list-style-type: none"> More expensive than soil nail and soldier piles and lagging 	<ul style="list-style-type: none"> Intermediate risk of excess movement or deformation between soil nails and soldier piles and lagging
Soldier piles and timber lagging	<ul style="list-style-type: none"> Feasible for temporary and permanent conditions 	<ul style="list-style-type: none"> Will provide permanent solution once road is widened 	<ul style="list-style-type: none"> More costly than some solutions Limited headroom will require special installation procedures Tie backs may be required 		<ul style="list-style-type: none"> Negligible risk of excessive total and differential settlement of wall affecting bridge abutment

- NOTES:
1. Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.
 2. Table to be read in conjunction with accompanying report.

Prepared By: NC/DUP
Checked By: PRB

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.) split spoon sampler for a distance of 300 mm (12 in.)

(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

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
$\log_{10} x$	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_{oc1}	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

(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_1	sensitivity

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
 2 shear strength = (compressive strength)/2
 * density symbol is ρ . Unit weight symbol is γ where
 $\gamma = \rho g$ (i.e. mass density \times acceleration due to gravity)

PROJECT 05-1132-013-1		RECORD OF BOREHOLE No 112		1 OF 1	METRIC
W.P. 2172-06-00		LOCATION N 4791248.7 E 272825.1		ORIGINATED BY MA	
DIST HWY 403		BOREHOLE TYPE HAND AUGER/OPEN HOLE		COMPILED BY DMB	
DATUM GEODETIC		DATE May 13, 2009		CHECKED BY <i>PD</i>	

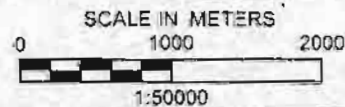
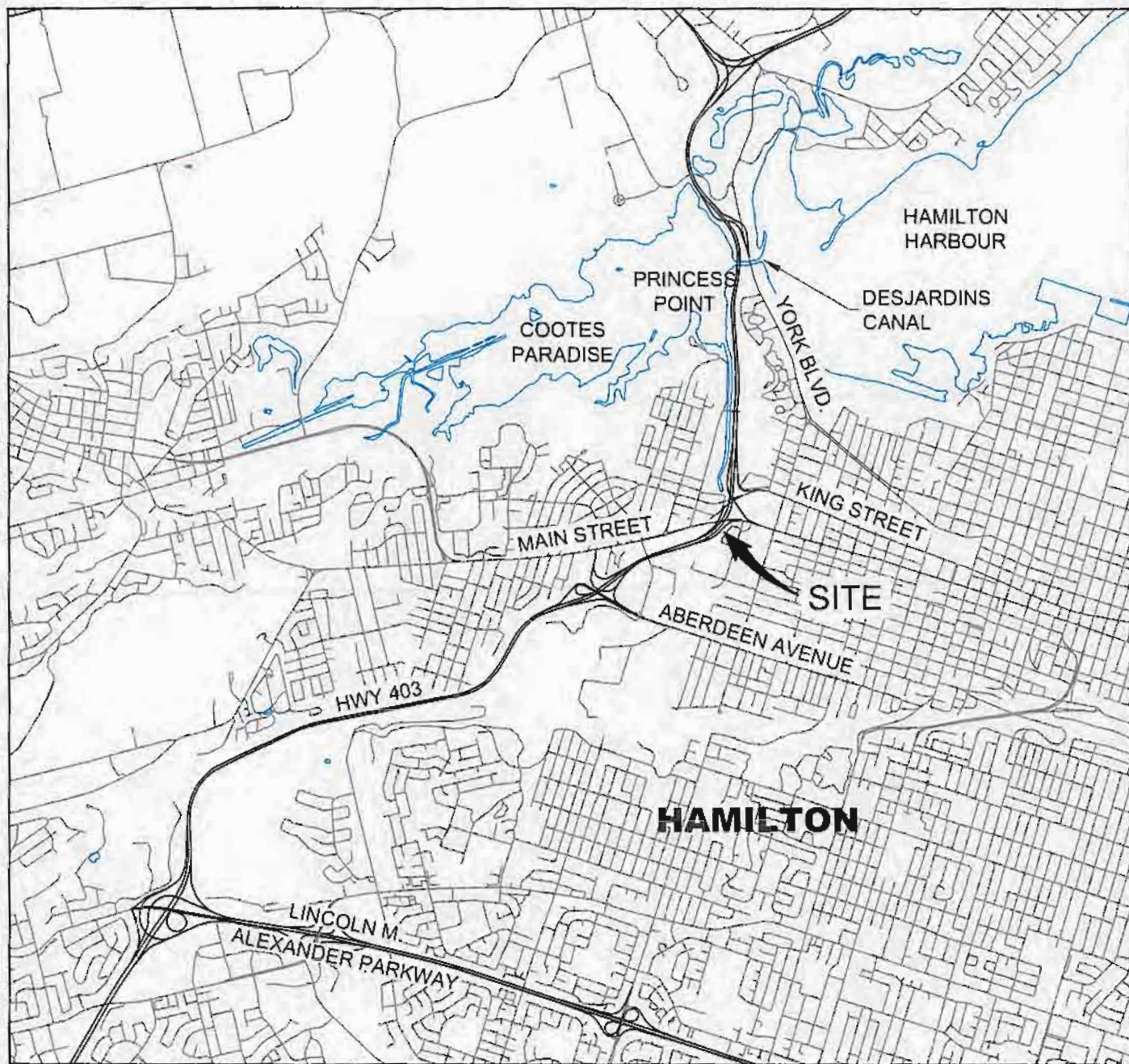
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
87.68 0.00 0.12	GROUND SURFACE TOPSOIL, silty Brown FILL, clayey silt, trace sand, trace to some gravel, slag, brick Firm to hard Brown													
			1	SS	7									
			2	SS	35									
			3	SS	12									
84.63 3.05	END OF BOREHOLE Borehole dry during drilling on May 13, 2009.		4	SS	17									

LDN MTO 01 05-1132-013-1.GPJ LDN MTO GDT 8/18/09

PROJECT 08-1132-013-1		RECORD OF BOREHOLE No 113		1 OF 1	METRIC
W.P. 2172-06-00		LOCATION N 4791270.9 E 272843.0		ORIGINATED BY MA	
DIST HWY 403		BOREHOLE TYPE HAND AUGER/OPEN HOLE		COMPILED BY DMB	
DATUM GEODETIC		DATE May 13, 2009		CHECKED BY <i>RP</i>	

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			20	40					
87.71	GROUND SURFACE													
0.09	TOPSOIL, silty, some sand Brown													
0.30	FILL, silt, some sand, some gravel, slag, brick, trace clay Brown													
	FILL, clayey silt, trace sand, trace gravel, trace topsoil, with sandy pockets, wood and cinders below about elev. 84.7m Soft to firm Brown and grey		1	SS	3									
			2	SS	8									
			3	SS	5									
			4	SS	3									
83.96	CLAYEY SILT, trace to some sand, trace gravel Soft Brown		5	SS	3									
3.75			6	SS	4									
82.68	END OF BOREHOLE													
5.03	Groundwater encountered at about elev. 84.4m during drilling on May 13, 2009.													

LDN_MTO 01 08-1132-013-1.GPJ LDN_MTO.GDT 8/18/09



PROJECT
PIER 4 RELOCATION, MAIN STREET UNDERPASS
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

TITLE

KEY PLAN



PROJECT No.	08-1132-013-1	FILE No.	08-1132-013-1-F09501
CADD	LVR	July 15, 10	SCALE As SHOWN
CHECK	ALP	Aug 14, 10	REV. 0

FIGURE 1

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

CONT No.
WP No. 2172-06-00

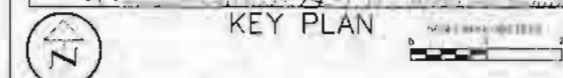


SHEET

PIER 4 RELOCATION
MAIN STREET UNDERPASS
HIGHWAY 403 BRIDGE REHABILITATIONS
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
LONDON ONTARIO CANADA



KEY PLAN

LEGEND

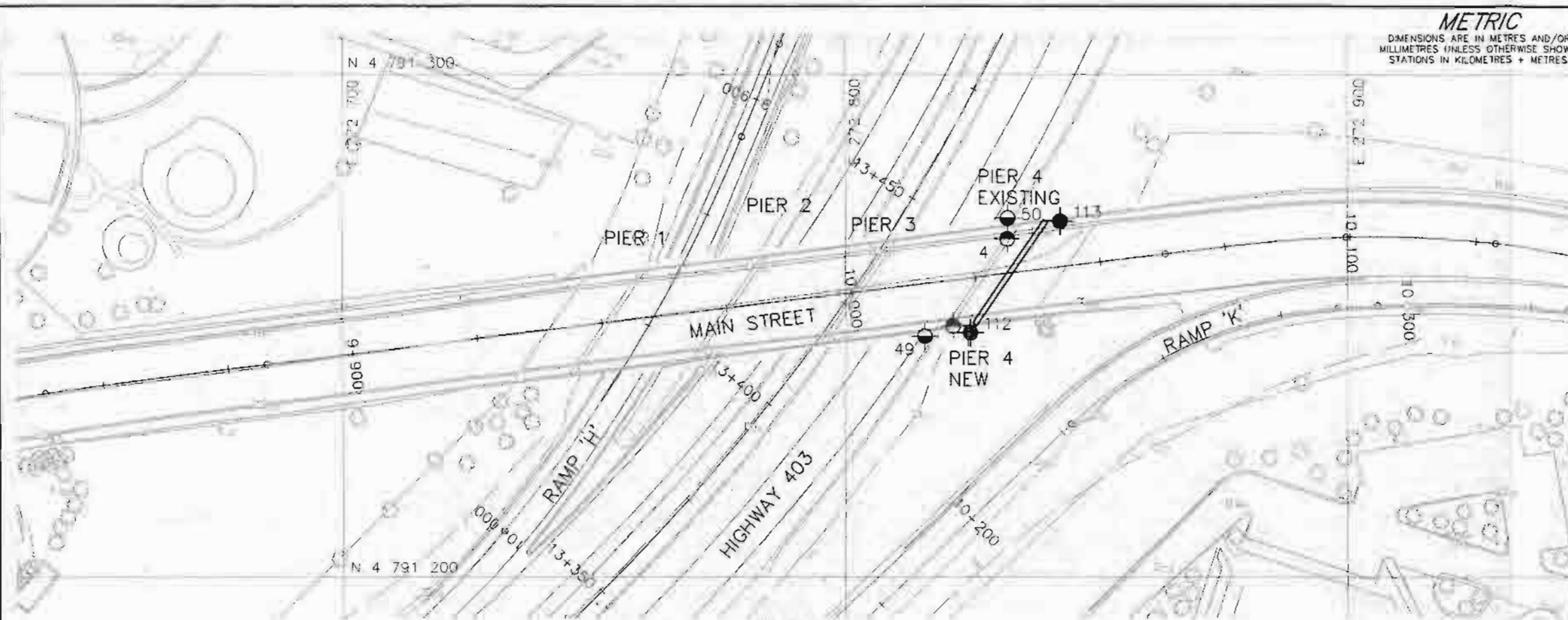
- Borehole - Current Investigation
- Borehole (By Others) (Geocres #30M5-29)
- Borehole Geocres #30M5-269
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 i/blow)
- DRY Borehole dry during drilling
- Wt encountered during drilling
- Measured WL
- rubble Material may contain 1 or more of the following: slag, cinders, plastic, wood, brick, glass, rubber, porcelain, refuse
- Geocres #30M5-29 report did not indicate any groundwater conditions during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
112	87.68	4 791 248.7	272 825.1
113	87.71	4 791 270.9	272 843.0
By Others (Geocres #30M5-29)			
1	85.50	4 791 250.0	272 821.6
4	85.31	4 791 267.3	272 832.5
Geocres #30M5-269			
49	85.01	4 791 248.0	272 815.9
50	84.95	4 791 271.4	272 832.6

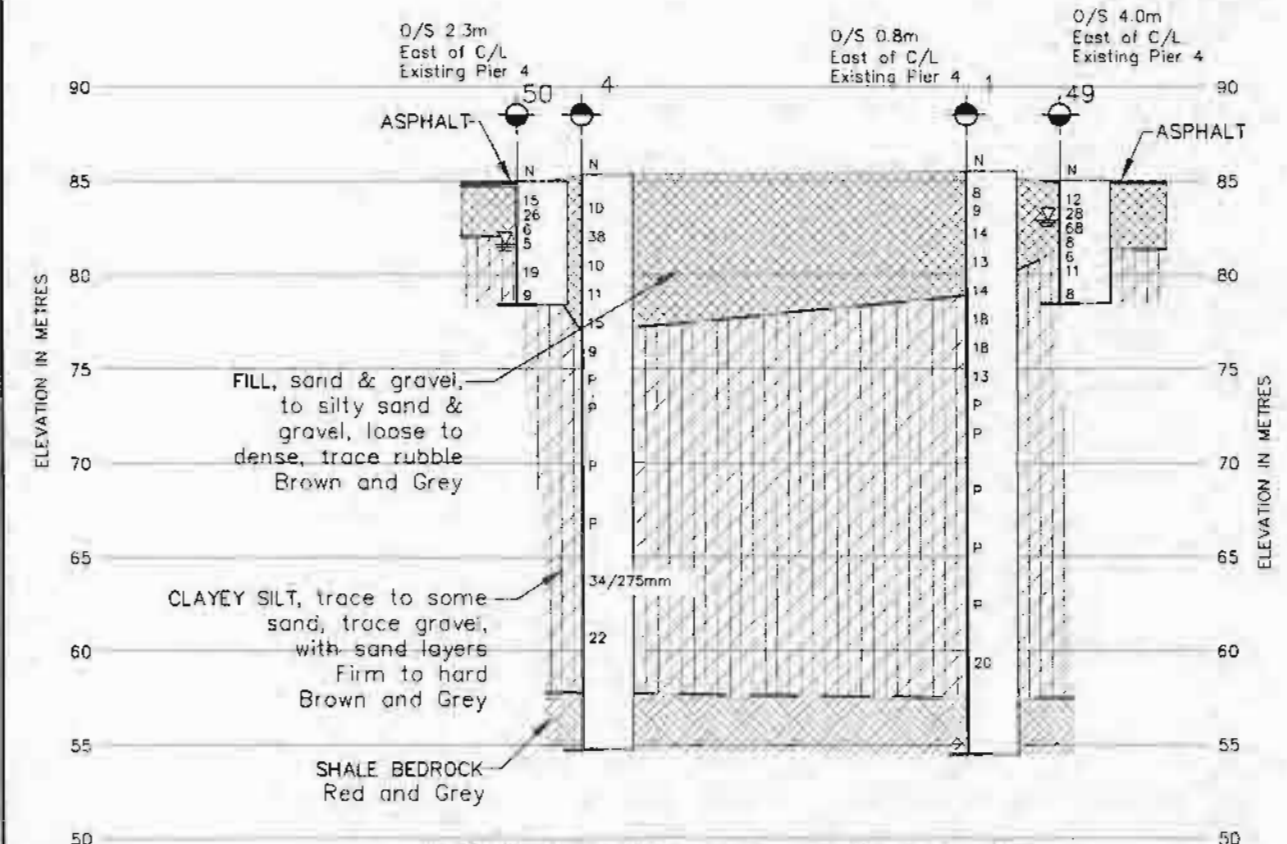
NOTES

REFERENCE

NO.	DATE	BY	REVISION
1			
Geocres No. 30M5-275			
HWY	403	PROJECT NO	08-1132-013-1
SUBM'D. DUP	CHKD	DATE	July 15/09
DRAWN: LMK	CHKD	APPD.	SITE 36-0043
			DWG. 1

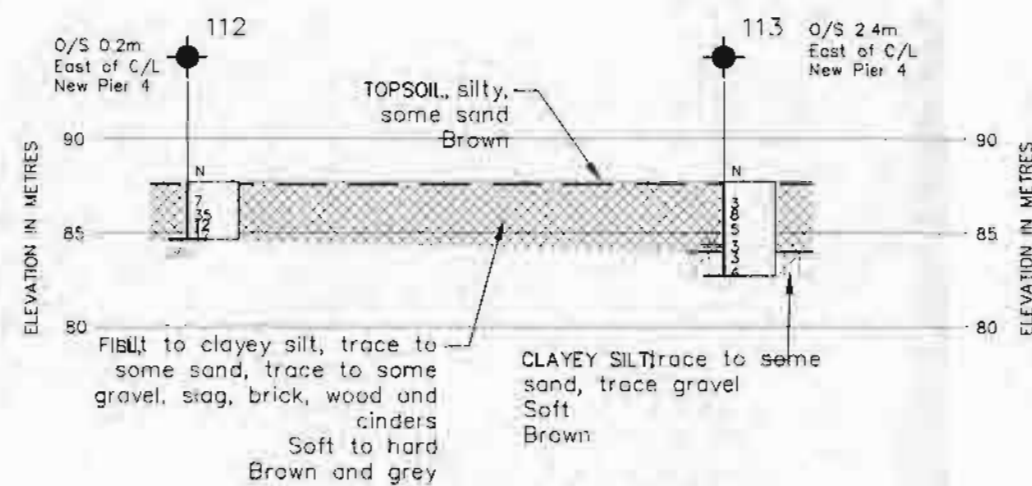


PLAN
SCALE
0 10 m



SECTION ALONG C/L EXISTING PIER 4

SCALE
0 4 m



SECTION ALONG C/L NEW PIER 4



**FOUNDATION INVESTIGATION AND DESIGN REPORT
RELOCATION OF PIER 4**

APPENDIX A

**Records of Previous Boreholes
(Geocres No. 30M5-29)**

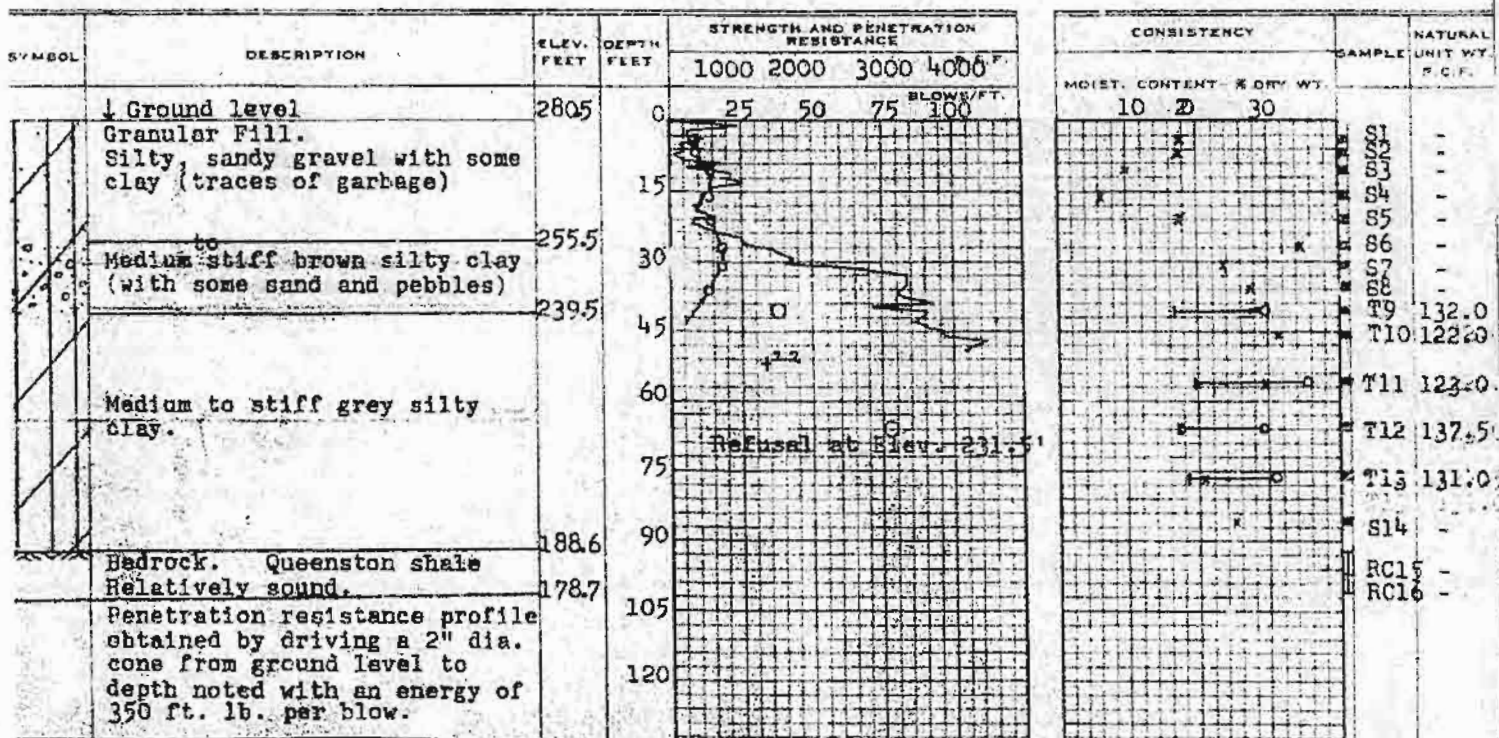
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 180-6 BORE HOLE NO. 1
 JOB F59-116 STATION 13+09 (33' Lt.)
 DATUM 280.5' COMPILED BY R.K.
 BORING DATE Jan. 7/59 CHECKED BY N.K.

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 2" SHELBY
 CASING

LEGEND

1/2 UNCONFINED COMPRESSION (QU) 0
 VANE TEST (C) AND SENSITIVITY (S) +
 NATURAL MOISTURE AND LIQUIDITY INDEX X
 LIQUID LIMIT 0
 PLASTIC LIMIT +



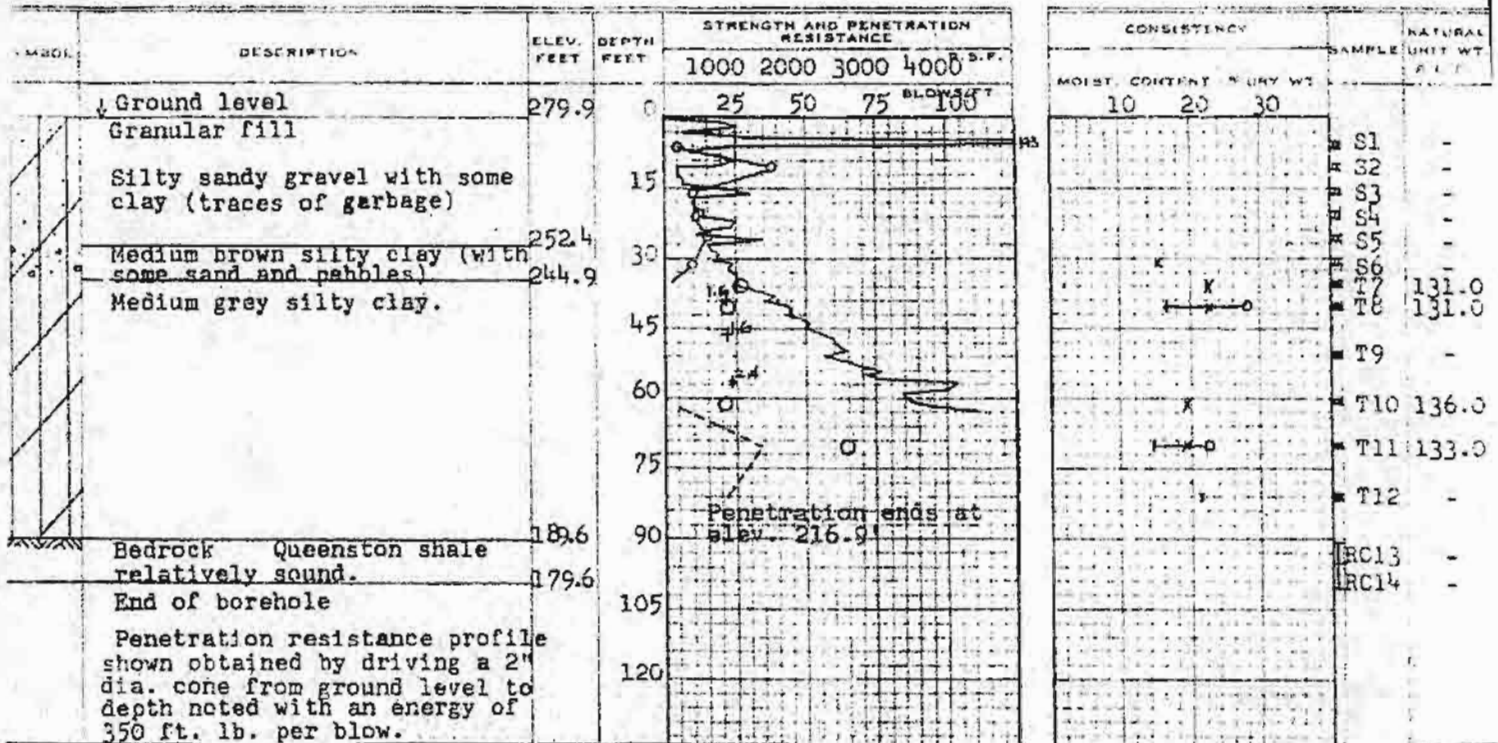
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

I.P. 180-60 BORE HOLE NO. 4
 OB. F59-116 STATION 12+77 (19' Lt.)
 ELEV. 279.9' COMPILED BY B.K.
 DATING DATE Dec. 22/59 CHECKED BY V.K.

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 2" SHELBY
 CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) — O
 VANE TEST (C) AND SENSITIVITY (S) — +
 NATURAL MOISTURE AND LIQUIDITY INDEX — X
 LIQUID LIMIT —
 PLASTIC LIMIT —



0005512

SUMMARY OF FIELD & LABORATORY TESTS

JOB F59-116

W.P. 180-60

Page 1

WOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH P.S.F.	UNIT WEIGHT P.C.F.	REMARKS
1	S1	3'-4.5'	Silty sandy gravel with some clay (Traces of garbage)	8	17.5	-	-	-	-	
	S2	6'-7.5'	"	9	17.2	-	-	-	-	
	S3	10'-11.5'	"	14	10.0	-	-	-	-	
	S4	15'-16.5'	"	13	6.2	-	-	-	-	
	S5	20'-21.5'	"	14	17.4	-	-	-	-	
	S6	25'-26.5'	Medium to stiff brown silty clay (with some sand and pebbles)	18	34.6	-	-	-	-	
	S7	30'-31.5'	"	18	23.6	-	-	-	-	
	S8	35'-36.5'	"	13	27.4	-	-	-	-	
	T9	40'-41.5'	"	P	28.0	16.4	29.6	1510	132.0	
	T10	45'-46.5'	Medium to stiff grey silty clay.	P	31.3	-	-	1090	122.0	
	Vane	51.5'	"	-	-	-	-	1329	-	Sens: 2.2
	T11	55'-56.5'	"	P	29.4	19.9	35.5	-	123.0	
	Vane	61.5'	"	-	-	-	-	>2000	-	Sens: -
	T12	65'-66.5'	"	-	17.2	17.4	29.3	3110	137.5	
	T13	75'-76.5'	"	P	20.6	18.2	30.6	3770	131.0	
	S14	85'-86.5'	"	20	25.0	-	-	-	-	

0005512

SUMMARY OF FIELD & LABORATORY TESTS

JOB F59-116

W.P. 180-60

Page 2

HOLE NO.	SAMP. No.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH P.S.I.	UNIT WEIGHT P.C.F.	REMARKS
1	RC15	91.8-96.8'	Queenston shale - relatively sound. Alternate red and grey layers.	-	-	-	-	-	-	
	RC16	96.8-101.8'	"	-	-	-	-	-	-	
2	S1	5'-6.5'	Soft brown silty clay with some sand and decayed vegetation.	5	29.6	20.2	27.7	-	-	
	S2	10'-11.5'	"	5	30.0	-	-	-	-	
	S3	15'-16.5'	"	11	30.4	-	-	-	-	
	T4	20'-21.5'	Medium grey silty clay.	P	34.1	20.7	31.7	653	126.0	Sens: 1.4
	Vane	26.5'		-	-	-	-	880	-	
	T5	30'-31.5'	"	P	19.8	-	-	850	132.0	
	S6	40'-41.5'	"	51	16.7	-	-	-	-	
	T7	45'-46.5'	"	39	19.2	18.4	27.3	1740	136.0	
	S8	5'-51.5'	"	16	20.8	-	-	-	-	
	T9	60'-61.5'	"	23	20.4	-	-	645	135.0	
	S10	70'-71.5'	"	82-10"	12.3	-	-	-	-	
	RC11	71.5-76'	Queenston shale - weathered and reddish in colour.	-	-	-	-	-	-	8% recovery.
	RC12	76'-81'	"	-	-	-	-	-	-	40% recovery

0005512

SUMMARY OF FIELD & LABORATORY TESTS

JOB P59-116

W.P. 180-6

Page 4

POLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETR RESIST. BLOWS/FI	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT P.C.F.	REMARKS
3	RC14	95'-99'	Queenston shale-relatively sound - alternate red and grey layers.	-	-	-	-	-	-	100% recovery
4	S1	5'-6.5'	Silty sandy gravel with some clay (traces of garbage)	10	-	-	-	-	-	
	S2	10'-11.5'	"	38	-	-	-	-	-	
	S3	15'-16.5'	"	10	-	-	-	-	-	Lost
	S4	20'-21.5'	"	11	-	-	-	-	-	
	S9	25'-25.6'	"	15	-	-	-	-	-	Lost
	S6	3'-31.5'	Medium brown silty clay with some sand and pebbles.	9	15.4	-	-	-	-	
	T7	35'-36.5'	"	P	22.5	-	-	1020	131.0	
	Vane 20'		Medium grey silty clay	-	-	-	-	920	-	Sens: 1.6
	T8	40'-41.5'	Medium grey silty clay	P	22.4	16.5	28.2	893	131.0	
	Vane 46'		"	-	-	-	-	880	-	Sens: 1.6
	T10	50'-51.5'	"	P	-	-	-	-	-	
	Vane 50.5'		"	-	-	-	-	900	-	Sens: 2.4
	T10	60'-61.5'	"	P	19.3	-	-	805	136.0	
	T11	70'-71.5'	"	34-11	19.2	14.6	24.7	2590	133.0	
	T12	80'-81.5'	"	22	21.4	-	-	-	-	

0005512

SUMMARY OF FIELD & LABORATORY TESTS

JOB F59-110

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W.P. 180-60

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETR. RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT PCF	REMARKS
4	RC13	90.3'-95.3'	Queenston shale - relatively sound-alternate red and grey layers.	-	-	-	-	-	-	100% recovery
	RC14	114'-119'	"	-	-	-	-	-	-	100% recovery
5	S1	10'-11.5'	Silty sandy gravel with some clay (traces of garbage)	15	9.9	-	-	-	-	
	S2	15'-16.5'	"	16	9.8	-	-	-	-	
	S3	20'-21.5'	"	12	14.1	-	-	-	-	
	S4	25'-26.5'	"	49	20.9	-	-	-	-	
	S5	30'-31.5'	Soft brown silty clay with some sand and decayed vegetation.	10	32.1	-	-	-	-	
	T6	35'-36.5'	"	P	21.2	-	-	750	128.0	
	T7	40'-41.5'	Medium to stiff grey silty clay.	P	25.7	-	-	373	125.00	
	T8	50'-51.5'	"	P	27.5	18.9	33.9	1585	127.0	
	Vane	55'	"	-	-	-	-	>2000	-	Sens: > 1.1
	T9	60'-61.5'	"	48	16.3	-	-	-	-	
	T10	70'-71.5'	"	49	18.7	-	-	3760	136.0	
	T11	80'-81.5'	"	18	22.4	20.0	26.5	1715	130.0	
	RC12	92'-97'	Queenston shale-weathered reddish colour and interbedded with limestone layers.	-	-	-	-	-	-	40% recovery.

0005512



**FOUNDATION INVESTIGATION AND DESIGN REPORT
RELOCATION OF PIER 4**

APPENDIX B

**Records of Previous Boreholes
(Geocres No. 30M5-269)**

PROJECT 08-1132-013-0 RECORD OF BOREHOLE No 49 1 OF 1 METRIC
W.P. 2172-06-00 LOCATION N 4791248.0; E 272815.9 ORIGINATED BY DJM
DIST HWY 403 BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS COMPILED BY LMK
DATUM GEODETIC DATE July 20, 2008 CHECKED BY *[Signature]*

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			20 40 60 80 100	SHEAR STRENGTH kPa						
85.01	ROAD SURFACE								○ UNCONFINED + FIELD VANE						GR SA SI CL
0.00	ASPHALT								● QUICK TRIAXIAL × LAB VANE						
0.15	FILL, sand and gravel, crushed, some silt Compact Brown														
83.94			1	SS	12		84								
1.07	FILL, clayey silt, some sand, some gravel Firm to hard Brown and grey		2	SS	28		83								10 42 31 17
			3	SS	68		82								
			4	SS	8		81								
81.35															
3.66	CLAYEY SILT, trace to some sand, trace gravel, trace topsoil Firm to stiff Grey		5	SS	6		80								
			6	SS	11		79								
79.68															
5.33	CLAYEY SILT, trace sand, trace gravel Stiff Grey														
78.46			7	SS	8										
6.55	END OF BOREHOLE														
	Groundwater encountered at about elev. 82.9m during drilling on July 20, 2008.														

LDN_MTO 01 08-1132-013-0.GPJ LDN_MTO GDT 8/18/08

RECORD OF BOREHOLE No 50

1 OF 1

METRIC

PROJECT 08-1132-013-0

W.P. 2172-06-00

LOCATION

N 4791271.4 E 272832.6

ORIGINATED BY DJM

DIST HWY 403

BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS

COMPILED BY LMK

DATUM GEODETIC

DATE

July 21, 2008

CHECKED BY

[Signature]

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
84.95	ROAD SURFACE														
0.00	ASPHALT														
0.23	FILL, sand and gravel, crushed, some silt Brown														
84.04															
0.91	FILL, sand, some silt, some gravel Compact Brown and grey		1	SS	15		84								
82.82			2	SS	26		83								15 62 13 10
2.13	FILL, clayey silt, trace sand, gravel Firm Grey		3	SS	6										
82.05							82								
2.90	CLAYEY SILT, with sand layers Firm Grey		4	SS	5										
80.99							81			144.0					
3.96	CLAYEY SILT, trace sand, trace gravel Stiff to very stiff Grey		5	SS	19		80								8 9 49 34
78.40			6	SS	9		79								
6.55	END OF BOREHOLE														
	Groundwater encountered at about elev. 81.6m during drilling on July 21, 2008.														



LDN_MTO_01_08_1132-013-0 GP7 LDN_MTO.GDI 8/18/09

+ 3 x 3

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE