FOUNDATION INVESTIGATION AND DESIGN REPORT
COMPOSITE RETAINING AND NOISE BARRIER WALL
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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TABLE OF CONTENTS

TABLE OF CONTENTS CONTINUED

PART A – FOUNDATION INVESTIGATION REPORT

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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 foundation investigations as part of the detail design work for GWP 2172-06-00.

This report was prepared for the design of the composite retaining/noise barrier wall that will be constructed along the proposed extension of the Aberdeen Avenue S-W Ramp speed change lane. The proposed wall will be constructed on the north side of the Highway 403 westbound lanes along the Aberdeen Avenue S-W Ramp.

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' proposal P71-3148 dated August 21, 2008. 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.

2.0 SITE DESCRIPTION

The 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 proposed composite retaining/noise barrier wall will be constructed on the north side of the Highway 403 westbound lanes along the Aberdeen Avenue S-W Ramp. The site is situated in Hamilton, Ontario. The site location is shown in the Key Plan, Figure 1.

Highway 403 in the Aberdeen Avenue Interchange area is a divided highway with two lanes and a speed change lane in each direction. The Highway 403/CPR Subway (Site 36-0048) is located at the western limit of the proposed barrier wall. A residential subdivision and Stroud Road Park are north and west of the site, respectively. The Aberdeen Avenue Underpass is some 400 metres to the east and a CPR railyard and the Chedoke Civic Golf Course are situated to the south.

Highway 403 in this area was constructed in an area with irregular topography with elevations varying from 88 to 91 metres. Land use immediately adjacent to the site is primarily residential north of the site and industrial and recreational south of Highway 403.

2.1 Site Geology

The Aberdeen Avenue Interchange is situated in the physiographic region of southern Ontario known as the Iroquois Plain[1]. 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 are primarily composed of lacustrine and outwash sands.[2] The surface of the bedrock is reported to be between elevations 30 and 91 metres.[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 Formation is a marine deposit of relatively uniform composition. The Queenston shale is irregularly interlayered with occasional beds or pockets of olive green

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[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., 1958. Bedrock Geology Toronto-Windsor Area. 1969. Geological Survey of Canada Map 1263 A. Scale 1:250,000.

calcareous siltstone. A 1960 study conducted by the Department of Highways found the top 3.0 to 4.0 metres is weathered.[5] The shale is highly fissile, susceptible to weathering under certain conditions and breaks easily parallel to the bedding planes.

[5] Geocres Report No. (30M5-95) by Department of Highways, Ontario entitled "Engineering Study, Properties of Queenston Shale, Proposed Chedoke Expressway, Hamilton Area, Ontario" dated August 19, 1960.

3.0 INVESTIGATION PROCEDURES

The field work for this project was carried out concurrent with drilling for the foundation investigation for the temporary supports for the bridge rehabilitations. Three boreholes, numbered 56 to 58, were drilled to depths of 3.9 to 5.0 metres on July 27, 2008.

The boreholes were advanced using a truck mounted CME 45 power auger supplied and operated by a specialist drilling contractor. Samples of the overburden were obtained at 0.75 to 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures.

Groundwater conditions in the boreholes were observed during drilling and these observations are provided on the corresponding Record of Borehole sheets. 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 an experienced member 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. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits testing were carried out on selected samples. The results of the field and laboratory testing are given on the Record of Borehole sheets and the figures in Appendix A.

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		
56	4 790 527.7	271 266.2	89.49	3.90
57	4 790 500.4	271 220.7	90.01	5.03
58	4 790 467.3	271 176.4	90.86	3.90

Information from the current boreholes was supplemented with boreholes from previous geotechnical investigations conducted by others for Highway 403. This report incorporates data from the following two previous reports completed by others:

- Record of Borehole 705 from Geocres Report No. 30M5-31, entitled "Highway #403, Contract #62-109, Channel Excavation and Earth Borrow, Detailed Soil Investigation with Continuous Sampling" dated February 4, 1963.

- Record of Borehole 6 from Geocres Report No. 30M5-39, entitled "Proposed Subway at Chedoke Expressway and T.H. & B. Railway Crossing in Hamilton, Twp. Of Ancaster, Dist. #4" dated January 28, 1960.

The Record of Borehole sheets for boreholes 6 and 705 and associated Summary of Field and Laboratory Test sheets are presented in Appendix B. The table below summarizes the locations, ground surface elevations and depths of the previous boreholes:

BOREHOLE	LOCATION (m)		GROUND SURFACE ELEVATION	DEPTH
	Northing	Easting	(m)	(m)
6	4 790 469.3	271 184.8	89.92	3.05
705	4 790 569.0	271 304.4	88.70	3.51

The locations of the current and previous boreholes are shown in plan on Drawing 1 and noted on the Record of Borehole sheets. Due to the age of the previous investigation records, the borehole locations should be considered to be somewhat approximate.

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 and in Appendix A. The Record of Borehole sheets and results of laboratory testing from the previous boreholes are presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets 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.

The locations of the boreholes are shown on the attached Drawing 1 along with a stratigraphic profile along retaining/noise barrier wall. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.1.1 Pavement Structure

Asphalt layers corresponding to the paved outer shoulder of Highway 403 westbound were encountered at the ground surface in boreholes 56, 57 and 58. The asphalt layers were 270 to 340 millimetres thick.

4.1.2 Fill

The asphalt in boreholes 56, 57 and 58 was underlain by granular fill layers 0.6 to 0.8 metres thick. The fill surface was found from elevations 89.2 to 90.6 metres. The fill is compact to dense with N values of 11 and 36 blows per 0.3 metres. Water contents of 4 to 6 per cent were measured in samples of the sand and gravel fill. The results of grain size analysis testing of two samples of granular fill are shown on Figure A-1 in Appendix A.

4.1.3 Sandy Silt

The fill in boreholes 56 and 58 was underlain by sandy silt from elevations 88.4 and 90.0 metres, respectively. The sandy silt layers were 0.8 to 1.0 metres thick. The sandy silt was loose to compact with N values of 8 and 21 blows per 0.3 metres. Water contents of 9 and 10 per cent were measured in the sandy silt. The results of a grain size analysis conducted on a single sample of sandy silt are presented on Figure A-2.

4.1.4 Silt

Silt was encountered in borehole 57 beneath the fill at elevation 88.9 metres. The silt was compact with N values of 10 and 18 blows per 0.3 metres. The silt had a water content of 11 per cent. The results of the grain size distribution testing for a sample of silt are presented on Figure A-3.

4.1.5 Clayey Silt

The shale bedrock in boreholes 56 and 57 is overlain by clayey silt layers 0.7 and 0.4 metres thick, respectively. The clayey silt was found from elevation 87.4 metres in both boreholes. The clayey silt is soft to stiff based on N values of 3 and 10 blows per 0.3 metres. A water content of 15 per cent was measured in a sample of clayey silt in borehole 56. The clayey silt is of low plasticity based on plastic and liquid limits of 16 and 25 per cent, respectively, and a plasticity index of 8 per cent. The results of the Atterberg limits testing are shown on Figure A-5

Materials described as low plasticity clay or silty clay were encountered at the ground surface in boreholes 705 and 6. For the purposes of this report, these materials will be classified as clayey silt. The clayey silt layer in borehole 705 was 1.5 metres thick and had an N value of 66 blows per 0.3 metres near the surface of the shale bedrock. The clayey silt layer in borehole 6 was 1.2 metres thick.

The results of a grain size analysis carried out on a sample of clayey silt are presented on Figure A-4.

4.1.6 Bedrock

Shale bedrock of the Queenston Formation was encountered beneath the clayey silt in boreholes 56, 57, 6 and 705 from elevations 86.7 to 88.7 metres; and below the sandy silt in borehole 58 from elevation 89.2 metres.

It was possible to conduct standard penetration testing in the shale to depths of 1.0 to 2.1 metres below the bedrock surface in boreholes 56 to 58. Standard penetration test N values of 86 to over 100 blows per 0.3 metres were obtained in this material. The samples of bedrock obtained from the standard penetration testing easily disintegrated suggesting that the shale is not cemented and is weathered. In borehole 6, a 1.5 metre long sample of shale was retrieved using an AXT core barrel. A recovery of 100 per cent was reported. The original geotechnical report completed for the existing CPR bridge noted that the upper 0.6 metres of the bedrock was weathered.

4.2 Groundwater Conditions

All boreholes were dry during and upon completion of drilling. Grey soils were not encountered in any borehole. It is anticipated that the groundwater level is below the bedrock surface and below elevation 85.0 metres, the lowest elevation investigated.

5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the supervision 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 Ms. Dirka U. Prout, P. Eng. under the direction of the Project Manager, 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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PART B – FOUNDATION DESIGN REPORT

COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
FROM HIGHWAY 6 WESTERLY TO ABERDEEN AVENUE
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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 composite retaining and noise barrier wall to be constructed between approximately Stations 11+596 and 11+790 on the north side of Highway 403 immediately east of the Highway 403/Canadian Pacific Railway Subway. The wall will be constructed along the proposed extension of the Aberdeen Avenue S-W ramp speed change lane. The recommendations are 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 design of the retaining wall was underway at the time of preparation of this report. Therefore, the following discussion is of a general nature. Once the details of the retaining wall design are complete, geotechnical comments more specific to the selected design will be incorporated into our finalized report.

6.2 Wall Foundations

The subsurface conditions encountered in the retaining wall area typically consist of asphalt and fill overlying shallow overburden consisting of sandy silt, silt and clayey silt. The overburden thickness is 1.2 to 3.1 metres. The surface of the underlying shale bedrock of the Queenston Formation was encountered between elevations 86.7 and 89.2 metres.

The relatively shallow shale bedrock is suitable for conventional spread or strip footings to support the proposed wall. Suitable wall designs are cast-in-place concrete cantilever walls and gravity retaining walls. Consideration was also given to founding in the compact silt and sandy silt and/or clayey silt between elevations 88.9 and 90.0 metres. It was noted, however, that the silty soils are not continuous over the wall area and the sandy silt was found to be loose in borehole 56 and the underlying clayey silt was soft. For those reasons, founding in the shale bedrock is preferred.

Alternatively, the selected composite retaining and noise barrier wall design may be of a proprietary nature, incorporating pre-cast panels and requiring use of drilled caissons which extend into the bedrock. Soil parameters required for design of caisson foundations are given in Section 6.2.2 and Table I. Such foundations are preferable due to the variability in depth and characteristics of the overburden along the wall alignment and improved resistance against lateral loads. At some locations, the depth to bedrock is in the order of 3 metres and approaching the practical limit of conventional spread/strip footing construction. For these reasons, a foundation solution using augered caissons is the preferred technical solution. A comparison of foundation alternatives is presented on Table II.

6.2.1 Strip/Spread Footings

Geotechnical Resistance

Strip or spread footings for gravity and cantilever walls founded on the surface of the shale bedrock below elevations 86.7 to 89.2 metres can be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 750 kilopascals and a geotechnical resistance of 500 kilopascals at Serviceability Limit States (SLS). The SLS value allows for 25 millimetres of settlement.

6.2.2 Caissons

The wall foundations should be designed and constructed in accordance with MTO's Special Provision SP599F01. If augered caissons are used for support of the composite retaining and noise barrier wall, a diameter of 0.6 to 0.9 metres should be used. The augered caissons should be socketed into the shale bedrock such that the socketed length is three times the caisson diameter.

Geotechnical design parameters for design of the caisson foundations based on the soil conditions encountered along the proposed noise barrier wall alignments on a station by station basis are provided in Table I. The stratigraphy presented in Table I has been simplified for the purposes of the composite retaining and noise barrier wall foundation design.

Where both an undrained shear strength, c_u, and an effective friction angle, ϕ', have been given for a specific stratum, the caisson design should be checked for both the drained and the undrained condition, and the larger of the two calculated caisson depths shall govern. Due to the relatively shallow overburden composed of generally compact granular materials or soft to stiff cohesive materials, augered caissons socketed into rock should be assumed to derive their load carrying capacity from socket resistance only. The ultimate shaft resistance, R_s may be computed as follows:

$$R_s = \pi b_s L_s q_s$$

Where

b_s = socket diameter (m);
L_s = length of socket (m);
q_s = average unit shear resistance along the socket.

An average unit shear resistance along the socket, q_s, for unweathered shale of 0.65 megapascals can be used for design. Assuming a socket length of 3 times the socket diameter of 0.9 metres, the factored geotechnical axial resistance at ULS is 3.3 megapascals. The SLS value is not applicable as the shale bedrock at the base of the caisson is considered to be an unyielding material.

6.2.3 Resistance to Lateral Forces

Strip/Spread Footings

The lateral pressures acting on the retaining portion of the wall will depend on the backfill soils, the type and method of placement of the backfill materials behind the wall and the subsequent lateral movement of the structure.

The resistance to lateral forces/sliding resistance between the compacted granular fill (assumed to be Granular A) and the subgrade soils should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC). Also, the retaining wall shall be checked for overturning. Assuming that the founding rock is not loosened/disturbed during excavation and footing construction, the following angle of friction and corresponding unfactored coefficient of interface friction, tan δ, may be used for the interaction between the concrete and the founding rock:

Footings on shale bedrock	angle of friction	28°
	tan δ	0.53

In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

Caissons

The passive resistance within the upper 1.2 metres below ground surface should be neglected to account for frost action. In addition, for foundation design, full passive resistance will be mobilized only where the ground surface in front of and behind the caissons is level. Where sloping ground is present adjacent to the noise barrier wall, the K_p values used in the calculation of the passive resistance should be adjusted to account for the presence of the sloping ground. The adjusted K_p value is to be applied to that portion of the caisson that is above the elevation of the ground surface in the area of sloping ground; below this elevation, the full K_p value may be applied.

6.2.4 Construction Considerations

The Queenston shale is known to be prone to deterioration upon exposure to air and/or water. Whether strip/spread footings or caissons are constructed, the footing areas should be protected and the concrete for the footings poured as soon as possible after excavation. The cleaned excavation base should be inspected by a qualified geotechnical person prior to placement of the concrete.

Strip/Spread Footings

The footings should be provided with a minimum of 1.2 metres of earth cover or equivalent thermal insulation for frost protection purposes.

Caissons

Caving of the granular fills and silty soils which are present in the overburden layer may occur. In order to minimize caving and protect the integrity of the caisson excavations, it is recommended that a temporary liner be used to minimize disturbance during excavation and construction. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the Contractor of this condition since it may affect installation of the composite retaining and noise barrier wall.

6.3 Lateral Earth Pressures

The lateral pressures acting on the retaining portion of the wall will depend on the wall type, the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. The following recommendations are made concerning the design of the retaining wall in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A but with less than 5 per cent passing the 75 micron sieve should be used as backfill behind the wall. The granular fill should be placed in accordance with Ontario Provincial Standard Drawing (OPSD) 3121.150 and compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP105 S10. 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 subdrains should be in accordance with OPSD 3190.100.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC, Figure 6.9.3. Compaction equipment should be used in accordance with SP105 S10.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC, Clause C6.9.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC, Clause C6.9.1).

- For either Case a or Case b, the pressures are based on compacted granular fill and the following parameters (unfactored) may be assumed:

	GRANULAR A	GRANULAR B (TYPE II)
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.30
At rest, K_o	0.43	0.46
Passive, K_p	3.69	3.39

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design parameters assume level backfill. If the final design slopes differ, these parameters should be adjusted as in CHDBC C6.9.1 (e).

6.4 Excavations and Temporary Cut Slopes

Excavations for the wall footings will extend through the existing surficial fill materials, through the native sandy silt, silt and clayey silt overburden soils into the underlying shale bedrock. The upper surface of the shale is expected to be weathered. Based on the description of the mechanical effort required to pulverize samples of weathered Queenston shale, reported in Geocres Report No. 30M5-95, it is anticipated that excavation of the weathered shale may be similar to excavating in a hard cohesive till.

Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical. Further comments specific to maintaining the stability of augered excavations for caissons are presented in Section 6.2.4.

Excavations are not expected to penetrate the groundwater level which has been confirmed to be below elevation 85.0 metres. Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away from the excavations at all times. The appropriate Non Standard Special Provision (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 silty soils and clayey silt soils would be considered Type 2 or 3 soils, depending on consistency or density, as applicable. The weathered shale is considered to be a Type 1 material.

Temporary Shoring

Temporary shoring may be required if there is insufficient space for open cuts and space is restricted. The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2.

7.0 MISCELLANEOUS

This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Project Manager, 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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TABLE I

GEOTECHNICAL DESIGN PARAMETERS

Composite Retaining and Noise Barrier Wall
Highway 403 Bridge Rehabilitations
GWP 2172-06-00

STATION	STRATUM	ELEVATION INTERVAL	UNDRAINED SHEAR STRENGTH c_u (kPa)	ANGLE OF SHEARING RESISTANCE φ' (°)	COEFFICIENT OF PASSIVE EARTH PRESSURE K_p	UNIT WEIGHT γ (kNm^{-3})
11+596 to 11+600	Granular fill	Ground surface to elevation 90m	-	30	3.0	20
	Compact to very dense sandy silt	Elevation 90m to elevation 89m	-	30	3.0	20
	Shale bedrock	Below elevation 89m		28	2.8	26
11+600 to 11+630	Soft clayey silt	Above elevation 89m	25	25	2.5	21
	Shale bedrock	Below elevation 89m		28	2.8	26
11+630 to 11+730	Compact to dense granular fill	Ground surface to elevation 89m	-	30	3.0	20
	Loose to compact sandy silt/silt to soft to stiff clayey silt	Elevation 89m to elevation 87m	-	25	2.5	20
	Shale bedrock	Below elevation 87m	-	28	2.8	26
11+730 to 11+790	Hard clayey silt	Above elevation 87m	200	28	3.0	21
	Shale bedrock	Below elevation 87m	-	28	2.8	26

NOTES:
1. Where both c_u and φ' have been given for a specific stratum, the foundation design should be checked for both the undrained and the drained conditions, and the larger of the two calculated foundation depths shall govern.
2. Passive earth pressure coefficient (K_p) values are provided for level ground. Where sloping ground is present adjacent to the composite retaining/noise barrier wall, adjusted K_p values must be used in the foundation design.
3. Table to be read in conjunction with accompanying report.

Prepared By: DUP
Checked By: PRB

TABLE II

COMPARISON OF FOUNDATION ALTERNATIVES

Composite Retaining and Noise Barrier Wall
Highway 403 Bridge Rehabilitations
GWP 2172-06-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Spread footings supported on shale	• May not be compatible with some proprietary retaining/noise wall designs. • Construction likely to be difficult due to variability in depth to bedrock.	• May be cheaper solution in areas with very shallow bedrock (within 1 metre of surface).	• While spread footings may be suitable for sections of the wall which act primarily as a retaining wall, a design incorporating spread footings may not offer enough resistance to wind load in sections acting primarily as noise barriers. • Complexity of construction increases if the depth to competent bedrock is greater • Risk of exposing large area of bedrock which is prone to weathering.	• Approximately $78, 000 assuming 1 metre wide footings.	• Construction becomes difficult and costs can escalate quickly if extensive sections with depth to bedrock surface greater than 1.5 metres. • Potential incompatibility with proprietary retaining/noise wall design.
Augered caissons socketed into shale	• Likely to be compatible with several proprietary retaining/noise wall designs. • Easily adaptable to varying bedrock depth.	• Minimizes excavation. • Rapid construction. • Provides best resistance to wind loads. • Preferred technical alternative.	• Possibility of poor bearing surface produced due to collapse of sidewalls if liner not used or if base not cleaned out adequately, especially for small diameter caissons which can only be inspected visually.	• Approximately $110, 000 assuming 3.6 metre spacing and average caisson length of 3.7 metres.	• Excessive settlement if caisson constructed without a liner or not adequately cleaned prior to pouring concrete.

NOTES:
1. Foundation 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: 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

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.)

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, 60º 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 cm^2 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	c_u, s_u 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[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_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 = σ'_p/σ'_{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 δ
c'	effective cohesion
c_u,s_u	undrained shear strength (ϕ = 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

* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 56

METRIC

PROJECT 08-1132-013-0
G.W.P. 2172-06-00
DIST ______ HWY 403
DATUM GEODETIC

LOCATION N 4790527.7 ;E 271266.2
BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS
DATE July 27, 2008

ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY ______

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.49	PAVEMENT SURFACE				
0.00	ASPHALT				
89.15 0.34	FILL, sand and gravel, crushed, some silt Compact Brown	1	SS	11	55 33 (12)
88.36 1.13	SANDY SILT, some gravel, trace clay Loose Brown	2	SS	8	
87.36 2.13	CLAYEY SILT, some sand, trace topsoil Soft Brown	3	SS	3	0 23 54 23
86.70 2.79	SHALE BEDROCK Grey	4	SS	100/ 25mm	
85.59 3.90	END OF BOREHOLE Borehole dry during drilling on July 27, 2008.	5	SS	100/ 75mm	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (89, 88, 87, 86); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 57

METRIC

PROJECT 08-1132-013-0			
G.W.P. 2172-06-00	LOCATION N 4790500.4 ;E 271220.7	ORIGINATED BY MA	
DIST	HWY 403	BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS	COMPILED BY BRS
DATUM GEODETIC	DATE July 27, 2008	CHECKED BY	

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.01	PAVEMENT SURFACE						
0.00	ASPHALT						
0.30	FILL, sand and gravel, some silt Dense Brown	1	SS	36	89	~6	53 34 (13)
88.88 / 1.13	SILT, trace to some gravel, some clay, some sand, trace topsoil Compact Brown	2	SS	18	88	~11	2 23 52 23
87.36 / 2.65	CLAYEY SILT, trace sand, trace gravel with silt pockets Stiff Brown	3	SS	10			
86.93 / 3.08	SHALE BEDROCK Brown and grey	4	SS	100	87		
		5	SS	113/ 280mm	86		
		6	SS	86	85		
84.98 / 5.03	END OF BOREHOLE Borehole dry during drilling on July 27, 2008.						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 58

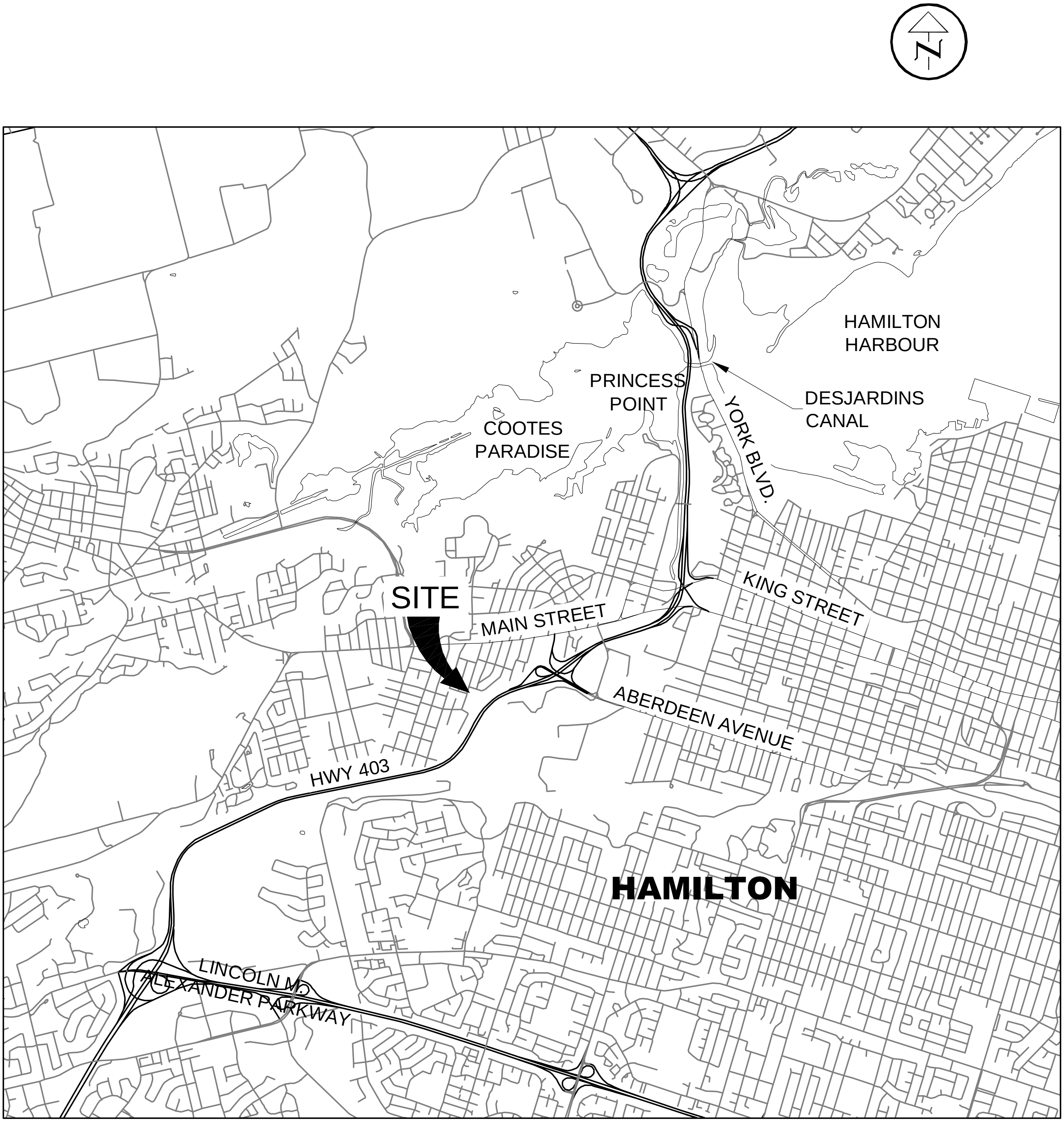
METRIC

PROJECT 08-1132-013-0
G.W.P. 2172-06-00
LOCATION N 4790467.3 ;E 271176.4
ORIGINATED BY MA
DIST HWY 403
BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS
COMPILED BY BRS
DATUM GEODETIC
DATE July 27, 2008
CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.86	PAVEMENT SURFACE						
0.00	ASPHALT						
0.27	FILL, sand and gravel, trace silt Brown						
89.98 / 0.88	SANDY SILT, some clay, trace gravel Compact to very dense Brown	1	SS	21		90	2 44 39 15
		2	SS	100/ 250mm			
89.15 / 1.71	SHALE BEDROCK Brown					89	
		3	SS	100/ 75mm			
		4	SS	100/ 75mm		88	
86.96 / 3.90	END OF BOREHOLE Borehole dry during drilling on July 27, 2008.	5	SS	100/ 75mm		87	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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



SCALE IN METERS
0 1000 2000
1:50000

PROJECT
COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

TITLE
KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.		08-1132-013-0	FILE No.	0811320130-F01001
			SCALE	AS SHOWN REV. 0
CADD	LMK	Sept. 29/08		
CHECK			FIGURE 1	

DRAWING FILE: 0811320130-F01001.DWG Plot Date: Feb 11, 2009 - 11:00am

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

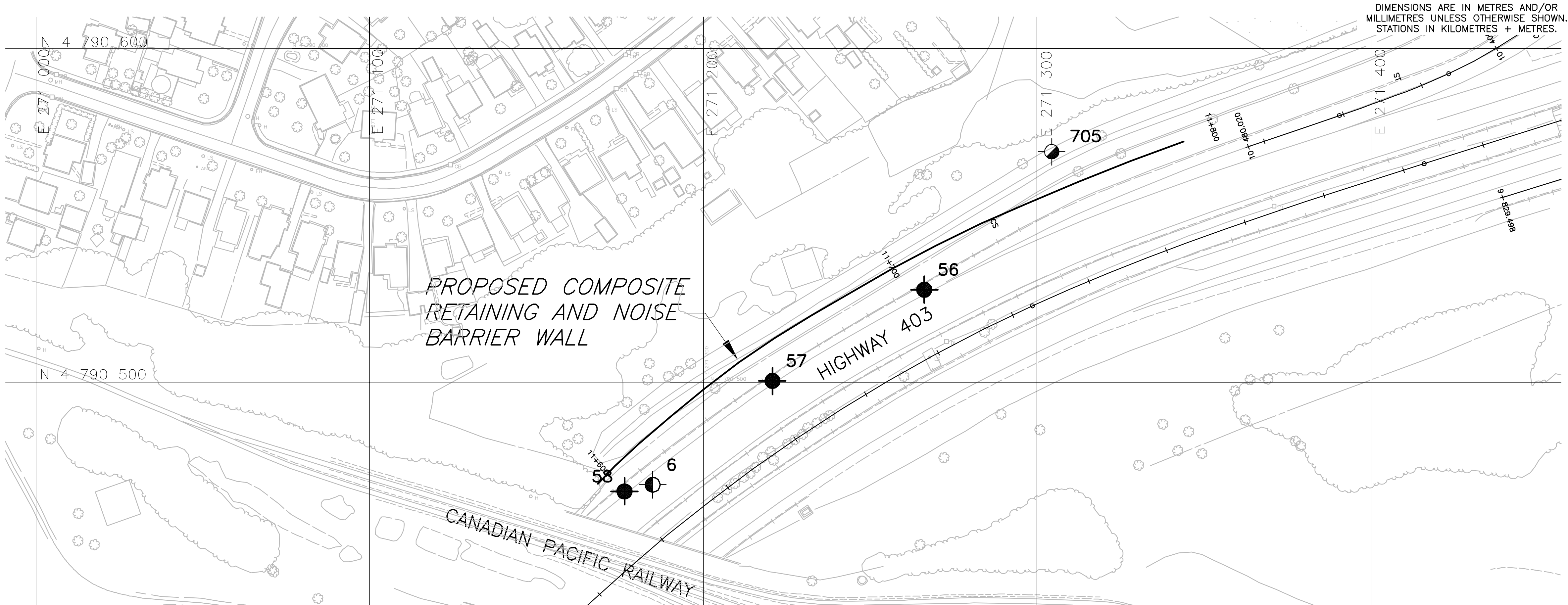
CONT No.
WP No. 2172-06-00

SHEET

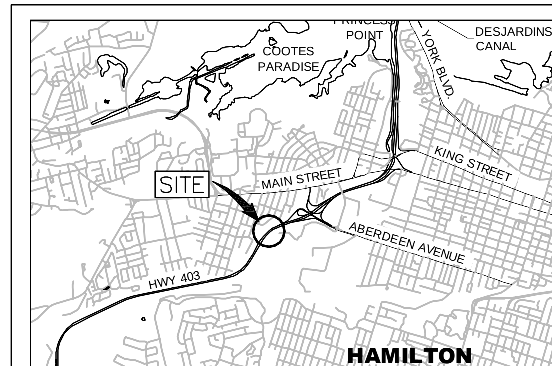
COMPOSITE RETAINING AND NOISE BARRIER WALL

HIGHWAY 403 BRIDGE REHABILITATIONS
BOREHOLE LOCATIONS AND PROFILE

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



PLAN
SCALE
15 0 15 m

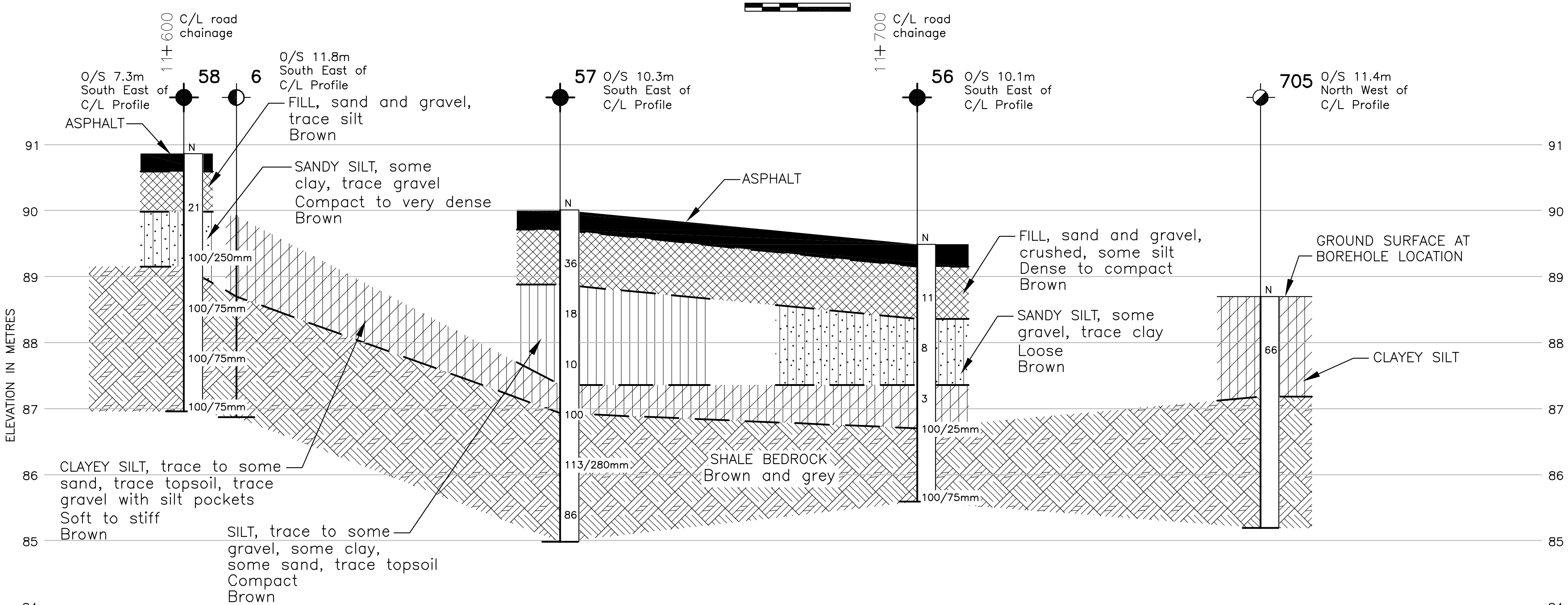


KEY PLAN
SCALE IN KILOMETRES
0 1 2

LEGEND

- Borehole – Current Investigation
- Borehole (By Others) (Geocres #30M5-31)
- Borehole (By Others) (Geocres #30M5-39)
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- DRY — Borehole / Test Pit dry during drilling / excavation
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10) NORTHING	EASTING
6	89.92	4 790 469.3	271 184.8
56	89.49	4 790 527.7	271 266.2
57	90.01	4 790 500.4	271 220.7
58	90.86	4 790 467.3	271 176.4
705	88.70	4 790 569.0	271 304.4



PROFILE ALONG PROPOSED COMPOSITE RETAINING AND NOISE BARRIER WALL

VERTICAL SCALE 1 0 1 m
HORIZONTAL SCALE 10 0 10 m

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield.

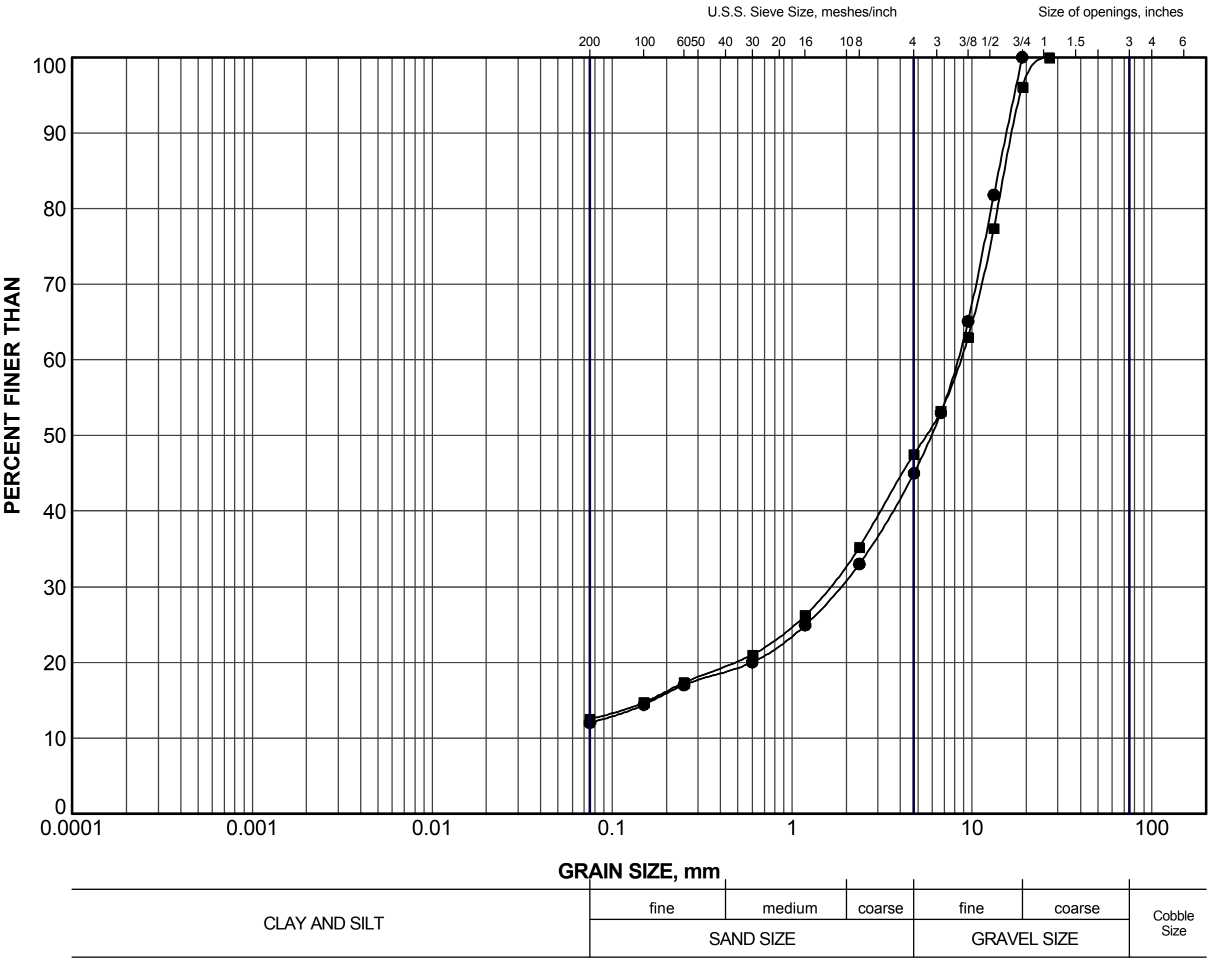
NO.	DATE	BY	REVISION

Geocres No. 30M5-266

HWY. 403		PROJECT NO. 08-1132-013-0	DIST.
SUBM'D. DUP	CHKD.	DATE: SEP 19/08	SITE:
DRAWN: LMK	CHKD.	APPD.	DWG. 1

APPENDIX A

LABORATORY TEST DATA



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	56	1	88.5
■	57	1	89.0

PROJECT COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

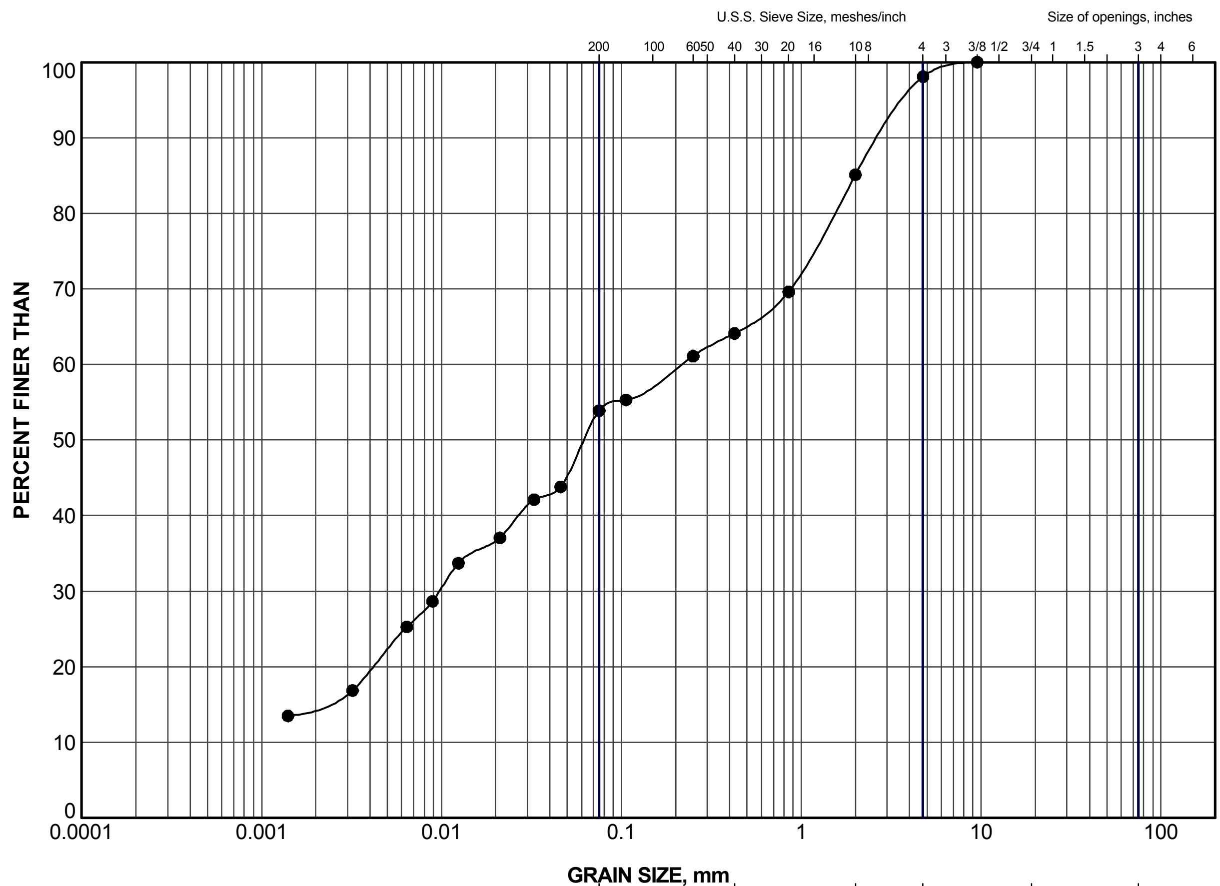
TITLE

GRAIN SIZE DISTRIBUTION
FILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-013-0		FILE No.	0811320130-R010A1
			SCALE N/A	REV.
DRAWN	LMK	Sep 12/08	**FIGURE A-1**	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	58	1	89.9

PROJECT

COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

TITLE

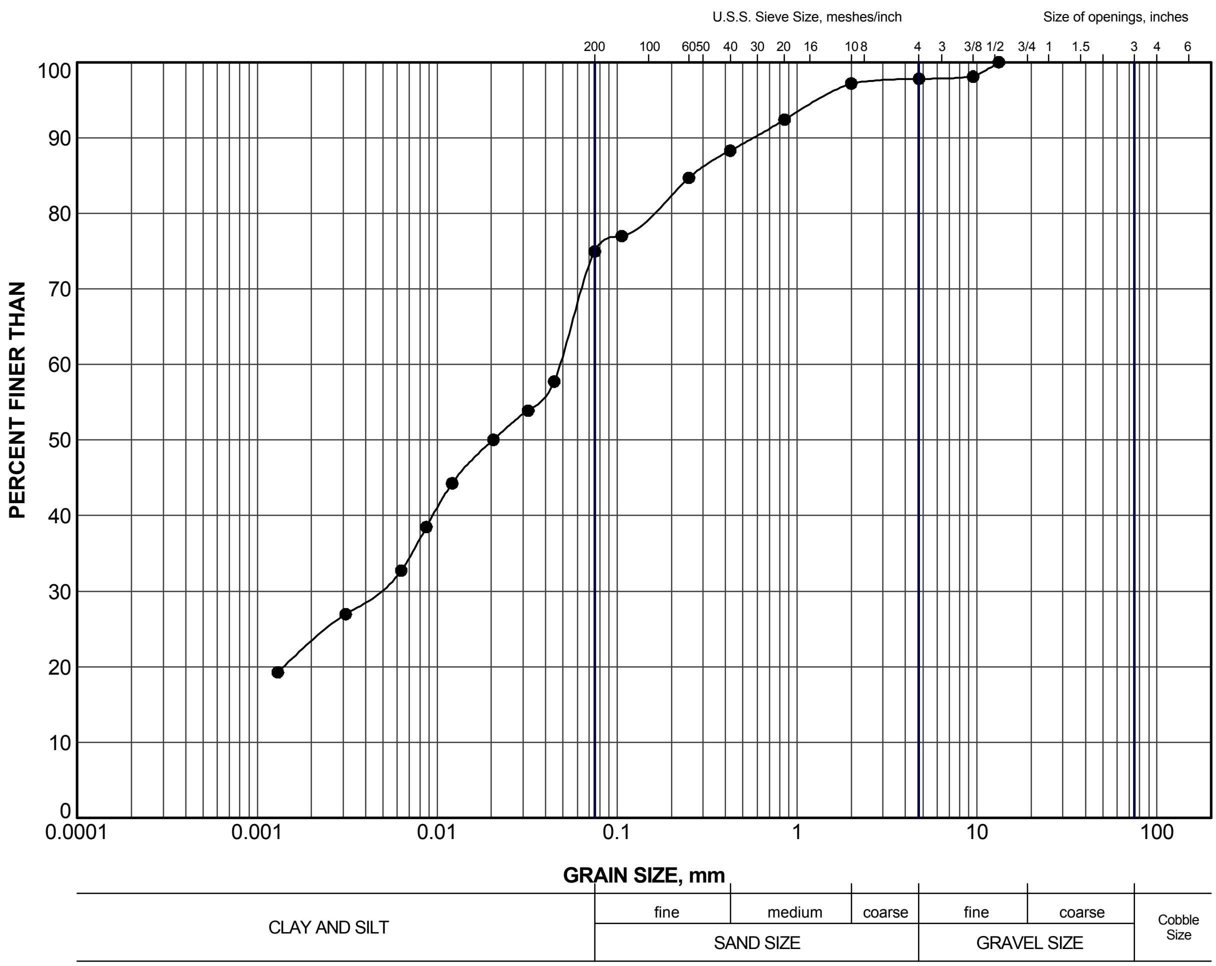
GRAIN SIZE DISTRIBUTION

SANDY SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-013-0	FILE No.	0811320130-R010A2
		SCALE N/A	REV.
DRAWN LMK	Sep 12/08	**FIGURE A-2**	
CHECK			

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	57	2	88.3

PROJECT COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

TITLE

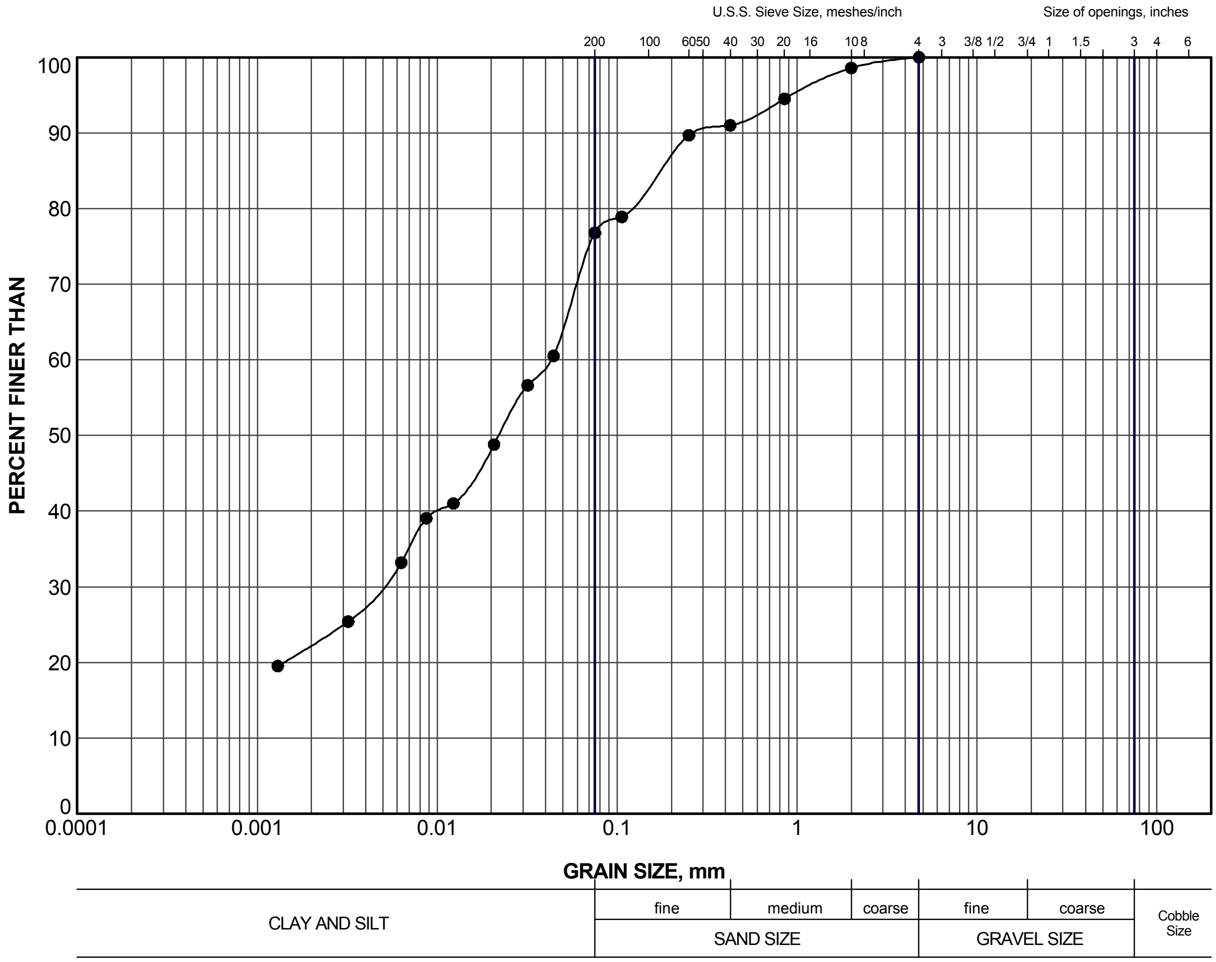
GRAIN SIZE DISTRIBUTION

SILT

PROJECT No.	08-1132-013-0		FILE No.	0811320130-R010A3
			SCALE N/A	REV.
DRAWN	LMK	Sep 12/08	FIGURE A-3	
CHECK				

Golder Associates
LONDON, ONTARIO

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	56	3	87.0

PROJECT COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

TITLE

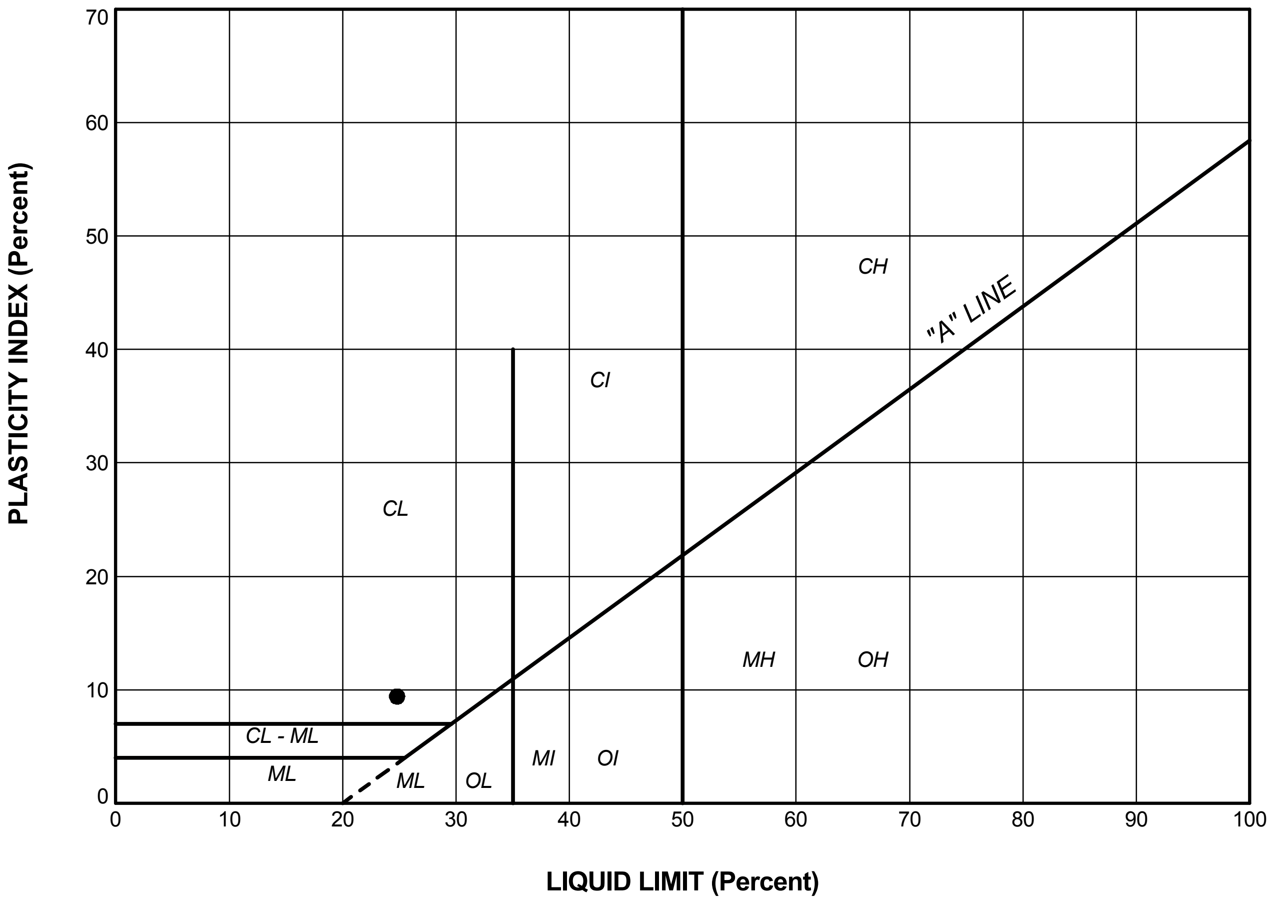
GRAIN SIZE DISTRIBUTION

CLAYEY SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-013-0		FILE No.	0811320130-R010A4
			SCALE N/A	REV.
DRAWN	LMK	Sep 12/08	FIGURE A-4	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	56	3	24.8	15.4	9.4

PROJECT
COMPOSITE RETAINING AND NOISE BARRIER WALL
HIGHWAY 403 BRIDGE REHABILITATIONS
GWP 2172-06-00

TITLE

PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-013-0	FILE No.	0811320130-R010A5
		SCALE	N/A REV.
DRAWN	LMK Sep 12/08		
CHECK		**FIGURE A-5**	

MTO_PI GLDR_LDN.GDT

APPENDIX B

RESULTS OF PREVIOUS INVESTIGATIONS BY OTHERS

APPENDIX B1

PREVIOUS BOREHOLE, LABORATORY AND FIELD DATA (GEOCRES NO. 30M5-31)

FORM OB-ML-126 (REV. 1959)--59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 140-57-1, 231-58-2, 231-58-3

JOB 60-F-14

DATUM 291.0'

BORING DATE May 31/60

BORE HOLE NO. 705

STATION 497+20 (170' Rt. C.E.B.L.)

COMPILED BY B.K.

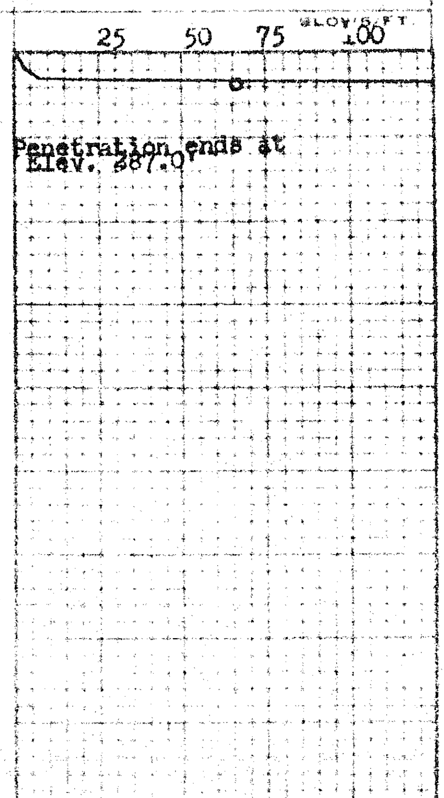
CHECKED BY H.D.

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

LEGEND

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS/FT.	CONSISTENCY / MOIST. CONTENT - % DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	291.0	0				
	Clay of low plasticity-hard-red brown	286.0				S1	-
	Bedrock-red shale	279.5	10			RC2	-
			20				



Penetration resistance profile shown; obtained by driving a 2" dia cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-14
W.P. 140-57-1
231-58-2
231-58-3

HOLE 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
703	S1	3'-4.5'	Silt with organic matter, sand and fine gravel, med dense, brown	25	12.7	-	-	-	-	
	S2	6'-7'	Sandy silt with fragments of shale-v. dense brown	78	10.2	-	-	-	-	
	RC 3	7'-10'	Weathered shale	-	-	-	-	-	-	
	RC 4	10'-12'	Sound shale	-	-	-	-	-	-	
	RC 5	12'-16'	" "	-	-	-	-	-	-	
704	S1	3'-4.5'	Clay of low plasticity with fragments of shale-hard-brownish red	99	10.7	-	-	-	-	
	RC 2	5.5'-7.5'	Bedrock-shale, red	-	-	-	-	-	-	
	RC 3	7.5'-12.5'	" " "	-	-	-	-	-	-	
705	S1	3'-4.5'	Clay of low plasticity with fragments of shale-hard-red	66	-	-	-	-	-	
	RC 2	6'-11.5'	Bedrock-shale,red	-	-	-	-	-	-	

APPENDIX B2

PREVIOUS BOREHOLE, LABORATORY AND FIELD DATA (GEOCRES NO. 30M5-39)

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 144-60
BORE HOLE NO. 6

JOB F 59-109
STATION 502+11 (54' RT)

DATUM 295.0'
COMPILED BY B.K.

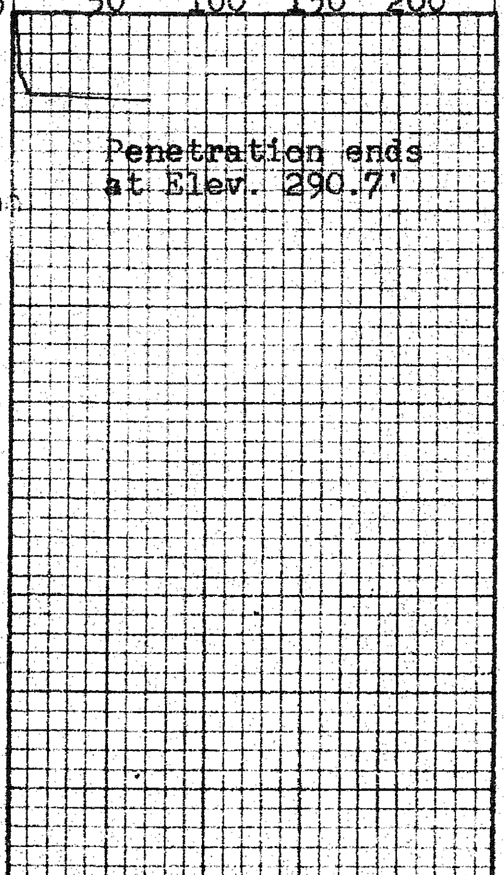
BORING DATE Nov. 12/59
CHECKED BY V.K. & A.L.

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) +S
NATURAL MOISTURE AND LIQUIDITY INDEX LI X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS/FT. 50 100 150 200	CONSISTENCY MOIST. CONTENT - % DRY WT.	SAMPLE	NATURAL UNIT WT P.C.F.
	groundlevel	295.0	0				
	Soft brown silty clay with sand ([illegible])						
	Bedrock Red & grey shale (Queenston formation)	291.0		Penetration ends at Elev. 290.7'		RC1	-
	End of borehole	285.0	10				
			20				
			30				
			40				



Penetration resistance profile shown; obtained by driving a 2" dia cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.

FORM OB—ML—222
FEB 59
1M-58-5569

SUMMARY OF FIELD & LABORATORY TESTS

JOB F 59-109
W.P. 144-60

HOLE 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
4	RC3	11.5'-15'	Bedrock-Decomposed Red Shale & Hard Clay	-	-	-	-	-	-	50 % Recovery
	RC4	15'-20'	Sound bedrock-red & grey Shale	-	-	-	-	-	-	100 % Recovery
	RC5	20'-25'	"	-	-	-	-	-	-	76 % Recovery
	RC6	25'-30'	"	-	-	-	-	-	-	100 % Recovery
	RC7	30'-35'	"	-	-	-	-	-	-	100 % Recovery
5	RC1	5'-10'	Bedrock-red & grey Shale	-	-	-	-	-	-	100 % Recovery
6	RC1	5'-10'	Bedrock-red & grey Shale	-	-	-	-	-	-	100 % Recovery
7	S1	5'-6.5'	Stiff brown silty clay with sand	42	13.7	-	-	-	-	
	RC2	9.2'-14.2'	Sound bedrock-red & grey Shale	-	-	-	-	-	-	100 % Recovery
	RC3	14.2'-19.2'	"	-	-	-	-	-	-	95 % Recovery
	RC4	19.2'-24.2'	"	-	-	-	-	-	-	100 % Recovery
	RC5	24.2'-29.2'	"	-	-	-	-	-	-	100 % Recovery
8	H1	-	Brown silty clay	-	-	-	-	-	-	
	RC2	4'-9'	Bedrock-red & grey shale	-	-	-	-	-	-	70 % Recovery
	RC3	9'-14'	Sound bedrock-red & grey Shale	-	-	-	-	-	-	100 % Recovery

S denotes split spoon sample
RC " rock core
H " hand sample
T " shelby tube sample