



August 2009

FOUNDATION INVESTIGATION AND DESIGN REPORT

High Fills for Temporary Detour at Highway 403
Aberdeen Avenue Interchange
Highway 403 Rapid Bridge Replacement
Highway 6 Westerly to Aberdeen Avenue
Hamilton, Ontario
GWP 2172-06-00, Agreement No. 2006-E-0081
Ministry of Transportation, Ontario - Central Region

Submitted to:

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REPORT



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FOUNDATION INVESTIGATION AND DESIGN REPORT HIGH FILLS FOR TEMPORARY DETOUR AT HWY 403 ABERDEEN AVE

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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORD OF BOREHOLE SHEETS

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGH FILLS FOR TEMPORARY DETOUR AT HWY 403 ABERDEEN AVE**

PART A

FOUNDATION INVESTIGATION REPORT

**HIGH FILLS FOR TEMPORARY DETOUR AT
HIGHWAY 403 ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 RAPID BRIDGE REPLACEMENT
GWP 2172-06-00, AGREEMENT NO. 2006-E-0081
MINISTRY OF TRANSPORTATION - CENTRAL REGION**



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 overall project is the rehabilitation of eight underpasses. This report presents the results of a supplementary geotechnical investigation conducted for the high fills associated with a temporary detour to be constructed for this rapid bridge replacement project. The detour is required to carry westbound Highway 403 traffic during the highway closure required for replacement of the Aberdeen Avenue underpass bridge deck. An additional investigation completed for a nearby construction area was reported separately.

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.



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 as illustrated on the Key Plan, Figure 1. This report addresses the foundations engineering aspects of a temporary detour to be constructed within the Aberdeen Avenue interchange area.

Highway 403 at the Aberdeen Avenue interchange is a divided highway with three lanes in each direction and speed change lanes at the interchange. Highway 403 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. In order to accommodate Highway 403, Chedoke Creek was realigned with some segments channelized or rerouted through underground culverts. A 914 millimetre diameter trunk supply sewer is located north of the Chedoke Creek channel as shown on Drawing 1. Ground surface elevations within the project area vary between elevation 86 metres near Ramp 'D' to elevation 107 metres on the slopes overlooking Highway 403.

A temporary detour will be constructed between Ramps 'E' and Ramp 'B' to facilitate movement of traffic during the replacement of the Highway 403/Aberdeen Avenue overpass deck. Photograph 1 in Appendix C shows the eastern portion of the temporary detour area. At the location of the temporary detour, Chedoke Creek currently flows in an open channel just east and west of the proposed embankments and crosses beneath Ramp 'D' through an approximately 88 metre long concrete box culvert. A residential subdivision and Stroud Road Park are north and west of the site, respectively. The Aberdeen Avenue underpass is some 30 metres to the southwest and a Canadian Pacific Rail yard and the Chedoke Civic Golf Course are situated to the south.

2.1 Site Geology

The project area is situated in the physiographic region of southern Ontario known as the Iroquois Plain¹. At the head of Lake Ontario, the Iroquois sand plain exists as a narrow plain between Lake Ontario and the Niagara escarpment. 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 90 metres in the Aberdeen Interchange 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 easily breaks 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. (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 additional field work for this project was carried out concurrently with fieldwork for the adjacent staging area for the Aberdeen Avenue rapid bridge replacement (reported separately). Six boreholes, numbered 101 to 106, were drilled to depths of 3.0 to 24.3 metres from April 22 to 28, 2009. Three cone penetration tests were carried out in the vicinity of the high fill to depths of 12.4 metres to 14.2 metres. The cone penetration tests (CPT) were conducted adjacent to boreholes 101 to 103 in order to develop a continuous stratigraphic profile within the primarily cohesive deposits. The results of the cone penetration tests are presented on the Record of Cone Penetration Test sheets provided following the text of this report.

Boreholes 101 to 103 were advanced using an all terrain vehicle mounted D50 turbo power auger supplied and operated by a specialist drilling contractor. Boreholes 104 to 106 were drilled using a portable tripod mounted unit using washboring techniques after the borehole casing was installed to approximately 1.5 metres. 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. In addition, a dynamic cone penetration test was carried out at borehole 103 to assess the apparent density of the underlying sand and gravel materials.

Groundwater conditions in the boreholes were observed during drilling. The installation details and 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.

Piezocene penetration testing (CPT) was carried out adjacent to boreholes 101, 102 and 103. CPT is a state-of-the-art, in-situ technique for geotechnical site characterization studies. The CPT unit consists of a special rod equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve and porewater pressure. It is advanced at a constant rate of 20 millimetres per second into the ground using a drill rig. A reliable, continuous stratigraphic profile, together with important engineering properties, such as undrained shear strength, can be interpreted from the CPT results. Further, because the CPT data are collected on a continuous basis, the detection of thin, soft/loose and/or pervious layers is possible. This is important as these zones could control the behaviour and performance of the soil mass. These layers may go undetected by conventional drilling and sampling operations.

The piezocene penetration testing was carried out on April 28 and 29, 2009 by a senior member of our engineering staff with extensive experience using the equipment. The results of the CPT testing are provided on the Record of Cone Penetration Test sheets following the report text. The CPT test data presented in the sheets show the field collected data consisting of normalized tip resistance, local sleeve friction and porewater pressure at the test locations.

The field work was supervised on a full-time basis by experienced members of our 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 the benchmark elevations are referenced to geodetic datum.



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

Table 1: Locations of Borehole Explorations, 2009

Borehole	Location (m)		Ground Surface Elevation (m)	Depth (m)
	Northing	Easting		
101	4 790 786	271 747	86.67	19.72
102	4 790 807	271 739	88.00	24.29
103	4 790 786	271 799	86.75	22.56
104	4 790 787	271 680	87.63	19.66
105	4 790 778	271 675	87.17	14.17
106	4 790 762	271 688	87.72	12.95

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 and grain size distribution analyses, were carried out on selected samples. The results of the field and laboratory testing are given on the Record of Borehole sheets and on the figures in Appendix A.

Information from the recently completed boreholes and CPT explorations was supplemented with boreholes from a previous geotechnical investigation conducted by others for the original design of Highway 403. Subsurface data are present on the sections and profiles shown on the drawings included in this report. Previous borehole and testing data was obtained from the following report:

- Records of Boreholes 61 to 63 from Geocres Report No. 30M5-31, entitled "Hwy. 403, Contract No. 62-109, Channel Excavation and Earth Borrow, Detailed Soils Investigation with Continuous Sampling", dated February 4, 1963.

The Record of Borehole sheets for the previous boreholes 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.

Table 2: Locations of Previous Boreholes

	Location (m)		Ground Surface Elevation	Depth
Borehole	Northing	Easting	(m)	(m)
Geocres No. 30M5-31:				
61	4 790 786	271 700	82.91	16.61
62	4 790 784	271 730	82.30	13.59
63	4 790 767	271 812	82.30	13.11

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 and therefore, have been based on scaled distances on available site plans. It should be noted that past construction activities may have modified the conditions reported in the previous boreholes.



4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes completed during the 2009 program, 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 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 clay or silty clay in the previous boreholes 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.

Remnants of former temporary construction works may be present adjacent to the Chedoke Creek channel where it crosses Ramp 'D'. The extent of such works is difficult to determine through boreholes alone. However, their presence should be expected since the structures are located in areas of known soft surficial soils. Temporary shoring may have been installed to support excavations 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 Drawings 1 and 2. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized below.

In general, the stratigraphy generally consists of surficial topsoil and fills overlying an extensive deposit of clayey silt that overlies sand and gravel which is then underlain by shale bedrock. The clayey silt is interlayered with silty clay and silt.

4.1.1 Topsoil and Fill

Topsoil was encountered at the ground surface in boreholes 101, 102 and 103 in layers 90 to 150 millimetres thick. Classification of materials identified in this report as topsoil was based solely on visual and textural evidence. Testing of organic content or for other constituents or nutrients or the topsoil's general suitability as a growth supporting medium was not carried out. Therefore, the use of materials classified as topsoil in this report cannot be relied upon for supporting growth of landscaped vegetation (e.g. selected grasses).

Fill materials were encountered below the topsoil in boreholes 101, 102 and 103 from elevation 86.6 to 87.9 metres and from the ground surface in boreholes 104, 105 and 106. The fill layers ranged from 2.1 to 7.4 metres in total thickness. The fill consisted primarily of clayey silt with variable proportions of sand, gravel, topsoil and other organic material.

The fill had standard Penetration Test (SPT) N values of 3 to 37 blows per 0.3 metres with typical N values between 5 and 18 blows per 0.3 metres. Measured water contents in the fill materials ranged from 11 to 30 per cent. The results of five grain size analyses carried out on samples of clayey silt fill are presented on Figure A-1 in Appendix A. A plot of shear strength profile based on CPT and field vane in situ testing is shown on Figure A-6.



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4.1.2 Sand

A 1.7 metre thick layer of sand was encountered at the surface of borehole 62 (30M5-31).

4.1.3 Clayey Silt

Clayey silt was encountered beneath the fill in boreholes 101 to 103, 105 and 106 from elevations 80.4 to 84.6 metres, beneath silt in borehole 101 from elevation 72.1 metres and below the silty clay in borehole 104 from elevation 75.4 metres.

The clayey silt had N values between 3 and 43 blows per 0.3 metres. In situ vane shear tests were conducted in the softer clayey silt layers and these tests indicated undrained shear strengths ranging from 30 to over 140 kilopascals (kPa). The sensitivity ranged from 1.3 to 4.0. Measured water contents in the clayey silt samples were 17 to 39 per cent. The clayey silt was of low plasticity with plastic limits of 15 to 18 per cent and liquid limits of 22 to 34 per cent. The average plastic and liquid limits were 17 and 28 per cent, respectively, with an average plasticity index of 11 per cent. The results of grain size testing conducted on fourteen samples of clayey silt are presented on Figures A-2 and A-3. The results of the Atterberg limits testing are shown on Figure A-5.

Soils described as clay of low plasticity in the previous boreholes have been classified as clayey silt for the purposes of this report based on the reported Atterberg limits results. Clayey silt was encountered at the ground surface of previous boreholes 61 and 63 (30M5-31) and below the sand in borehole 62 (30M5-31) from elevation 80.9 metres. At the time that boreholes 61 and 63 (30M5-31) were advanced, Chedoke Creek was not channelized, therefore the uppermost 1.1 to 1.9 metres of clayey silt likely represents geologically recent fluvial (floodplain) sediments.

The clayey silt in the previous boreholes had N values up to 50 blows per 0.3 metres. In situ field vane shear tests conducted in the softer clayey silt zones yielded shear strengths of 25 to 84 kilopascals indicating a firm to stiff consistency. A plot of the shear strength data from boreholes 61 through 63 (30M5-31) together with that from the recent CPT testing is presented in Figure A-7. Measured water contents in the clayey silt range from 18 to 28 per cent. The clayey silt is of low plasticity with average plastic and liquid limits of 16 and 27 per cent, respectively, and an average plasticity index of 11 per cent.

4.1.4 Silt

The clayey silt in borehole 101 was interlayered with a 1.2 metre thick layer of silt at about elevation 70.7 metres. The silt was compact with an N value of 15 blows per 0.3 metres and a water content of 21 per cent.

4.1.5 Silty Clay

The clayey silt layers in boreholes 101 and 103 were underlain by silty clay from elevation 68.2 and 69.5 metres. Silty clay was also encountered beneath the fill in borehole 104 from elevation 80.9 metres. The silty clay layers had N values of between 3 and 28 blows per 0.3 metres but generally less than 10 blows per 0.3 metres. The silty clay was firm to stiff with undrained shear strengths of 30 to 66 kilopascals based on in situ shear vane tests and the sensitivity ranged from 1.3 to 2.4. Water contents of 22 to 34 per cent were measured in the silty clay. The silty clay is of intermediate plasticity based on a plastic and liquid limits of 19 and 36 per cent, respectively, and a plasticity index of 17 per cent. The results of the Atterberg limits determination are shown on Figure A-5. The results of a grain size analysis conducted on a single sample of silty clay are shown on Figure A-4.



4.1.6 Sand and Gravel

The silty clay in borehole 101, the clayey silt in borehole 102 and the silty clay in borehole 103 were underlain by sand and gravel with cobbles from elevation 66.2 metres to 68.2 metres. The sand and gravel had N values of 24 to greater than 100 blows per 0.3 metres. Measured water contents in the sand and gravel ranged from 7 to 10 per cent.

4.1.7 Shale Bedrock

Shale bedrock of the Queenston Formation was encountered below elevations 67.8 and 68.2 metres in boreholes 101 and 104, respectively. Shale bedrock was encountered in previous boreholes 61 and 62 (30M5-31) below 13.6 to 15.1 metres of overburden and below approximately elevations 67.8 to 68.7 metres. In borehole 61, a 1.5 metre length of AXT core was obtained. The bedrock surface in borehole 62 (30M5-31) was inferred based on refusal to further auger drilling at elevation 68.7 metres.

4.2 Groundwater Conditions

The groundwater conditions in all of the 2009 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 and table. Groundwater levels are expected to fluctuate based on climatic and seasonal variations.

Groundwater was encountered in borehole 102, 103 and 105 between elevations 83.6 and 85.8 metres. Borehole 101 was dry during and upon completion of drilling. Due to the use of washboring techniques, the groundwater level could not be determined at boreholes 104 and 106.

Groundwater was encountered in previous boreholes 61 to 63 (30M5-31), inclusive, at elevations 81.1 to 82.3 metres. Flowing artesian groundwater was encountered at the bedrock interface near elevation 67.8 metres in boreholes 61 and 62 (30M5-31). The artesian head in borehole 61 (30M5-31) was reportedly 6.1 metres above ground level, or at elevation 89.0 metres.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
101	86.67	-	-
102	88.00	4.4	83.6
103	86.75	1.8	84.9
		20.9	65.9
104	87.63	-	-
105	87.17	1.4	85.8
106	87.72	-	-
61 (30M5-31)	82.91	0.6	82.3*
62 (30M5-31)	82.30	0.0	82.3*
63 (30M5-31)	82.30	1.2	81.1

NOTE: *Flowing artesian water encountered in boreholes 61 and 62 at elevation 67.8 m, estimated head 6 metres above ground surface.



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The water level in Chedoke Creek was measured at elevation 80.86 metres at the Ramp 'D' culvert outlet and at elevation 81.24 metres at the culvert inlet on April 29, 2009.

Natural groundwater levels in the area of the proposed temporary detour (within the Chedoke Creek valley), based on the measured groundwater levels, change in colour of the native soils and encountered water levels, has been inferred to be between elevation 81 and 82 metres in the native deposits. However, groundwater may also be encountered at higher elevations between about 84 to 86 metres within the overlying fill, likely as a result of infiltration and the variable permeability of the fill materials.

Groundwater levels are expected to be subject to seasonal and climatic variation and may be influenced by the water levels within the Chedoke Creek.




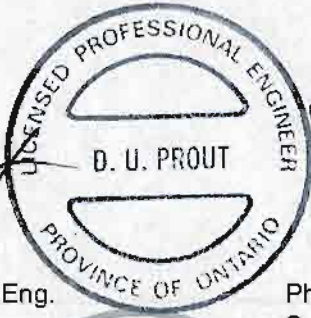
5.0 MISCELLANEOUS

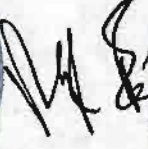
This investigation was carried out using equipment supplied and operated by Walker Drilling Inc., who are Ontario Ministry of Environment licensed well contractors. The field operations were supervised by Mr. Matt Rhody and Mr. Tom Zalucki under the direction of Mr. David J. Mitchell. The laboratory 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.

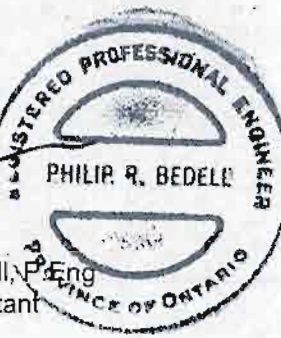
This report was prepared by Mr. Neil Charters under the direction of the Project Engineer, Ms. Dirka Prout, P.Eng. and the Team leader, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.


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**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGH FILLS FOR TEMPORARY DETOUR AT HWY 403 ABERDEEN AVE**

PART B

FOUNDATION DESIGN REPORT

**HIGH FILLS FOR TEMPORARY DETOUR AT HIGHWAY 403
ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 RAPID BRIDGE REPLACEMENT
GWP 2172-06-00, AGREEMENT NO. 2006-E-0081
MINISTRY OF TRANSPORTATION - CENTRAL REGION**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the temporary high embankment fills that have been proposed to divert westbound traffic on Highway 403 during the Rapid Bridge Replacement project at the Aberdeen Avenue interchange. The recommendations are based on our interpretation of the factual information obtained during the explorations described in this report. 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.

6.1.1 Subsurface Conditions

The borehole data indicate that the subsurface conditions within the design area typically consist of approximately 6 metres of fill used to infill the original Chedoke Creek valley when this segment of Highway 403 was constructed in the early 1960s. The Ramp 'D' embankment rises a further 6 metres above the valley fill at the point where the temporary westbound detour crosses Ramp 'D'. The fill is underlain by native clayey silt interlayered with silt and clayey silt till to elevation 66 to 68 metres. The clayey silt is underlain by sand and gravel deposits near Station 0+070. Weathered shale bedrock was encountered near elevation 68 metres. Bedrock may be deeper east of Station 0+070. The groundwater level is expected to be near the Chedoke Creek elevation of 81 to 82 metres; however groundwater may also be present in the fill between elevations 84 and 86 metres and a previous investigation encountered flowing artesian pressures at the bedrock interface (Geocres 30M5-31). An artesian head elevation of 89 metres was reported.

6.2 Embankment Design

6.2.1 Proposed Embankment Configuration

Design drawings provided by Morrison Hershfield indicate that two sections of temporary embankments for the temporary westbound detour and temporary connection will be constructed on either side of the existing Ramp 'D' embankment (Main Street West to Highway 403 eastbound on-ramp). The entire structure will be approximately 200 metres long and up to 7 metres high. The main temporary westbound detour will convey westbound Highway 403 traffic. There will also be an approximately 50 metre temporary connection between Ramp 'D' and the main detour for westbound traffic. The temporary westbound detour will cross the centreline of Ramp 'D' at approximately Station 0+105 detour chainage. Once the rehabilitation work on the Aberdeen Avenue underpass structure is complete, it is understood that the paved surfaces of the temporary westbound detour and temporary connection will be decommissioned. The MTO has indicated that the entire detour embankment will be removed.

The proposed detour fills will be placed above the existing Ramp 'D' Chedoke Creek culvert which conveys flows from the west to the east under Ramp 'D' at approximately Stations 0+100 and 0+115 along the temporary westbound detour and approximately Stations 0+020 and 0+030 along the temporary connection. With standard 2 horizontal to 1 vertical side slopes, the southern toe of the detour embankment between Stations 0+040 and



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0+075 and the northern toe of the temporary connection embankment between 0+050 and 0+080 will encroach upon the existing open channel sections of Chedoke Creek. Roadway protection will be required to maintain the stability of the embankment in these areas since space does not permit embankments with slopes of 2 horizontal to 1 vertical or flatter in this area without some form of retaining scheme. The detour fills will also be constructed above an existing 914 millimetre diameter sewer which parallels the Chedoke Creek culvert about 16 metres to the north.

6.3 Existing Structures/Features

Ramp 'D' Chedoke Creek Culvert

A survey of the Chedoke Creek culvert was carried out on May 21, 2009 by a senior staff member from our office. The Chedoke Creek culvert is a concrete box culvert with dimensions of about 4.9 metres by 3.1 metres by 88.4 metres. As-built drawings of the culvert are not available; however, the dimensions obtained from contract drawings for DHO Contract No. 63-116 were consistent with the field measurements. Between the outlet and approximately 60 metres west of the outlet, a mixture of cobbles, boulders, silt and gravel has been deposited on the concrete invert. Therefore, the culvert profile was determined based on surveying the interior obvert profile.

The elevation of the culvert obvert was determined at 5 metre intervals. A difference in elevation of 165 millimetres between the inlet and outlet was measured. This corresponded to an approximate grade along the culvert invert of 0.19 per cent. Analysis of the survey data indicated that non-uniform settlement has occurred along the longitudinal axis of the culvert. A graph showing the settlement profile along the culvert invert relative to an average grade of 0.19 per cent (between the culvert ends) is shown in Figure E-1. Cracks showing signs of water seepage were noted in the culvert walls and roof at several locations as shown on photographs 2 to 4 in Appendix C.

Chedoke Creek Open Channel

The temporary detour embankments will be constructed adjacent to the existing Chedoke Creek Open Channel. The open channel is a concrete lined trapezoidal section with an invert typically 2.4 metres wide with side slopes inclined at approximately 1.4 horizontal to 1 vertical. The total top of channel width is about 11 metres. As can be seen in photographs 5 to 7, the channel invert has cracks and missing sections of concrete where infiltration, exfiltration or scour can occur and vegetation has begun to colonize the channel. A sinkhole approximately 2 metres long and 0.6 metres wide was discovered immediately northeast of the outlet of the Ramp 'D' Chedoke Creek culvert, suggesting erosion and loss of ground into the broken culvert sections.

Trunk Sanitary Sewer

A 914 millimetre diameter trunk sanitary sewer is located approximately 16 metres north of the Chedoke Creek culvert and open channel within the vicinity of the proposed embankments. The trunk sanitary sewer invert elevation ranges from approximately 84.2 metres near Ramp 'E' to 84.5 metres adjacent to the inlet of the Ramp 'D' Chedoke Creek Culvert. It is understood that the sewer was constructed in about 1962. Where the sewer pipe was located in fills, it was laid on a concrete cradle which was constructed on a compacted Granular B or sand cushion.



6.4 Embankment Settlement

6.4.1 Analyses

Settlements resulting from new embankment construction were estimated using Settle 3D, a commercially available settlement analysis program. The settlement analyses utilized information from the existing boreholes, results of the piezocone testing, results of eight oedometer tests conducted on samples obtained at the adjacent Aberdeen Avenue site and reported in Geocres Reports No. 30M5-269 and 30M5-36, and the design information provided to date. Calibration of the model was accomplished through a back analysis of the settlement of the Ramp 'D' Chedoke Creek culvert to obtain parameters which generated a settlement profile that was similar to the results of the level survey of the culvert. Once satisfactory parameters had been obtained, the new temporary detour was modelled as 100 metre long embankments having the dimensions of the proposed embankments.

The pre-consolidation profile at the site was established by comparing the results of previous oedometer tests to preconsolidation pressures inferred from the following correlation developed by Mesri (1975)⁶:

$$\sigma'_p = s_u/0.22 \text{ where}$$

s_u – average mobilized undrained shear strength (kilopascals)

σ'_p – preconsolidation pressure (kilopascals)

The undrained shear strengths of the existing cohesive soils were estimated based on the results of in-situ field vane shear strength tests reported in Geocres Report No. 30M5-36 and shear strengths inferred from the recent piezocone testing. Figures D-1 to D-4 present summaries of the results of field and laboratory testing conducted in the clayey silt deposit or on samples of clayey silt. These summaries were used to interpret the engineering parameters used for this analysis. The table summarizes the parameters used in the settlement analyses:

Table 3: Summary of Geotechnical Engineering Parameters

Soil Unit	Recompression Index, C_r	Compression Index, C_c	Initial Void Ratio, e_0	Consolidation Coefficient, C_v
Clayey Silt, ground surface to 67 m (lightly overconsolidated)	0.008	0.08	0.53	7.5 m ² /yr

The results of the settlement analyses are presented in Appendix E.

Initial Highway 403 Construction

Calibration of the model was completed by simulating the initial Highway 403 construction by assuming an overall area grade raise of approximately 6 metres representing the general filling of the Chedoke Creek valley followed by the local 12 metre grade raise for the Ramp 'D' embankment. The area grade raise was modelled with a 200 metre square fill area and Ramp 'D' was modelled by a 6 metre high embankment with a crest width of approximately 14 metres and side slopes at 2 horizontal to 1 vertical. Estimated total settlements along the centreline of the culvert ranged from approximately 70 millimetres at each end of the culvert to approximately

⁶ Mesri, G. 1975. Discussion on: a New Design Procedure for Stability of Soft Clays. American Society of Civil Engineers Journal of the Geotechnical Engineering Division, V. 101, GT4, pp. 409 – 412.



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155 millimetres in the centre of the culvert. It is anticipated that all settlement of the culvert related to the construction of Highway 403 has concluded, given the approximately 40 years since construction.

Table 4: Summary of Estimated Settlement, Highway 403 Construction

Distance from west of outlet (m)	Description	Estimated Total Settlement (mm)	Relative Settlement (mm)	
			Based on Settle 3D modelling	Based on field measurements
0	Culvert Outlet	70	0	0
25	East toe Ramp 'D' embankment	75	5	22
40	Centreline Ramp 'D' embankment	155	85	93
60	West toe Ramp 'D' embankment	75	5	38
88	Culvert Inlet	70	0	0

Proposed Detour Embankment

The proposed detour fill will involve placing two embankments up to 7 metres high, separated by the Ramp 'D' embankment. This has been modelled by two embankments, one 5 metres high and one 7 metres high, having the approximate dimensions of the temporary detour embankments. It has been assumed that the embankments will have side slope angles of approximately 2 horizontal to 1 vertical. The following table shows the estimated total long-term settlements below the temporary detour embankment.

Table 5: Summary of Estimated Settlement, Detour Embankment

Detour Station	Estimated Settlement (mm)
0+030	15
0+060	60
0+070	65
0+095	55
East toe Ramp 'D' embankment	
0+110	5
East crest Ramp 'D' embankment	
0+115	6
West crest Ramp 'D' embankment	
0+130	70
0+150	90
0+160	50
0+170	7



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The maximum total settlements on the east and west sections of the temporary detour embankment were estimated to be 90 and 65 millimetres respectively. The influence of construction of the temporary detour embankments on the existing Ramp 'D' pavement is anticipated to be minimal.

Along the trunk sanitary sewer alignment, settlement will be a concern between approximately Stations 0+030 and 0+085 of the temporary westbound detour where the total settlement is expected to range between 12 and 55 millimetres. Settlement along the Ramp 'D' Chedoke Creek culvert will range from approximately 15 millimetres near the east toe of the Ramp 'D' embankment to 50 millimetres at the outlet. Settlement will be greater where the culvert passes below the temporary connection and will range from 5 millimetres near the inlet to 75 millimetres near the toe of the Ramp 'D'. Settlement profiles along both the Chedoke Creek culvert and the 914 millimetre diameter sewer pipe are shown on Figures E-1 and E-2 and E-3.

At the time this report was prepared, the duration that the detour embankment would remain in place was unknown. The anticipated settlement of the underlying Chedoke Creek culvert and sewer will be affected by the duration of time that the embankments are left in place. The estimated total settlements summarized above are considered reasonable for long-term conditions. If evaluations of the existing culvert and sewer determine that the additional long-term settlements caused by the detour embankments would be unacceptable, other than replacing and/or protecting the culvert and sewer, two primary and practical options for mitigating damaging settlements include:

- Removing the detour embankments once they are no longer required (a period of two years or less); or
- Use lightweight fill in lieu of conventional earth fill for the embankments.

Of these two options, the choice may largely depend on which of these is more economical. Figures E-1, E-2 and E-3 illustrate profiles of total and differential settlement along the culvert and sewer for use in decisions related to construction options. The structural impacts of the additional embankment loading on the sewer are being addressed by Morrison Hershfield.

6.5 Embankment Stability

The embankment cross-sections were analyzed using the limit equilibrium software SLOPE/W (Geoslope, 2008). The dimensions of the models were based on the general arrangement drawings supplied by Morrison Hershfield and dated June 4, 2009. For analysis of long-term stability, fully drained soil conditions (effective stress analysis) were assumed for the analyses. Short-term stability was also checked assuming undrained conditions. Geotechnical engineering parameters for the slope stability analyses are summarized in the table below.

Table 6: Summary of Geotechnical Engineering Parameters for Stability Analyses

Material	Effective Stress Internal Angle of Friction (degrees)	Effective Stress Cohesion Intercept (kPa)	Total Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)
Proposed Detour Embankment				
Granular Fill	35	0	20	-
Rock Fill	38	0	18	-
Existing Clayey Silt Fill	25	0	20	75
Native Clayey Silt				
- above elevation 75 metres	28	0	20	40
- below elevation 75 metres	28	0	20	100



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The results of the stability analyses are presented in Appendix F.

Preliminary planning for the detour embankment by Morrison Hershfield included embankment side slopes of 2 horizontal to 1 vertical. In some areas, these side slopes are achievable. From Station 0 to Station 0+025, the detour will be located at grade on an existing embankment. Between approximately Stations 0+075 and 0+100, and 0+120 to 0+145, the embankment will be placed on relatively flat ground and, built as outlined above, will have a factor of safety of at least 1.3 against instability for side slopes of 2 horizontal to 1 vertical. From about Station 0+105 to 0+115 the embankment crosses the existing Ramp 'D' embankment and will be less than 1 metre in height. In places, however, the side slopes come close to or extend beyond the top of the Chedoke Creek open channel. Two critical sections have been identified, one on each side of the Ramp 'D' embankment:

- Between approximately Stations 0+030 and 0+075 the southern toe of the embankment is close to, or within, the creek channel.
- From Station 0+150 to 0+180, the embankment will be built on a side slope and the toe will be immediately adjacent to the Chedoke Creek channel if a 2 horizontal to 1 vertical slope angle is used. The concrete lining in the channel is in poor condition and likely would not be capable of withstanding additional lateral loads from the presence of an embankment nearby.

It is considered that stability can be maintained and the effects of the embankment on the open channel in these areas can be minimized by using temporary vertical retained soil system (RSS) walls with a conventional slope, over-steepened and/or geo-reinforced embankments. Cohesive fill materials should not be used for the embankments in these areas. Stability analyses conducted for these critical sections indicate a global factor of safety greater than 1.3. The results of the stability analyses are summarized on Figures F-1 to F-4.

The toe of slope should be kept a minimum of 5 metres from the top edge of the Chedoke Creek channel to mitigate against damage to the liner of the open channel. The maximum inclination of an unreinforced oversteepened slope will depend on the type of material used to construct the embankment. The maximum sideslope inclinations for these embankments are:

1.5 horizontal to 1 vertical – Granular B Type II

1.25 horizontal to 1 vertical – Granular B Type II with geo-grid reinforcement

In areas where side slopes immediately adjacent to the existing open channel are required to be steeper than the above in order to maintain a 5 metre setback, it will be necessary to either reinforce the new fill using additional uniaxial geogrids embedded within the new fill or construct a vertical RSS wall as a protection system with a conventional unreinforced slope above the wall. Since the proposed embankment will be temporary and rapid construction and removal at a reasonable cost is required, use of a vertical RSS wall protection system in combination with a conventional slope is the preferred technical option.

RSS Protection System with Conventional Slope

The maximum wall heights required for RSS walls constructed for temporary protection systems in the two critical sections are 2.6 metres and 4.2 metres near Stations 0+160 and 0+070, respectively. This assumes that Granular B Type II material is used both for the wall backfill and construction of conventional 1.5 horizontal to 1 vertical embankment slopes above the wall sections. The maximum wall height includes 0.2 metres buried at the toe. The reinforcing elements (strips or geogrid) should be approximately 11 metres long to ensure global stability. Schematics showing this option at Stations 0+070 and 0+160 are shown in Figures F-1 and F2, respectively.



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The temporary retaining wall system could consist of a proprietary system such as a Terratrel Wall from the Reinforced Earth Company Ltd. or a SierraScape wall from Nilex. In the Terratrel system, the facing panels consist of two parts. One part is the wire mesh facing element and the other is a tie strip that connects the facing to the reinforcing strips. A double layer of geotextile is placed along the inside of the panels to retain fill for temporary applications. With the SierraScape system, the facing made of wire mesh. Uniaxial geogrids are used as primary soil reinforcement and biaxial geogrids provide secondary reinforcement and maintain facing alignment. The geogrid and the facing elements are connected with rods. Geotextiles are used to retain the fill at the face.

Although more expensive than imported earth fill, use of Granular B Type II for the RSS backfill and overlying embankment is preferred due to the wide availability of this material, the uniformity of material and improved frictional characteristics. Some cost savings may accrue if earth fill such as a clayey type material were used; however, this would likely result in a reduction in the inclination of the slope above the temporary wall to 2 horizontal to 1 vertical. Problems may also arise in obtaining sufficient earth fill in the quantities required with delivery to meet the project schedule. For these reasons, the use of granular fill is preferred.

Reinforced Soil Slope

The geogrids should consist of a primary reinforcement layer at 1.2 metre vertical intervals and a secondary reinforcement layer between each of the primary reinforcement layers (see Figures F-1 and F-2) at a maximum spacing of 0.6 metres from the geogrid layer above or below. For preliminary design considerations, the primary grids should have a 50 kilonewton tensile capacity and be installed for the full width of the new embankment. The secondary reinforcement grids should extend back from the face a minimum of 2 metres for the first 5 metres of embankment height and 1.2 metre thereafter to the peak embankment height. The new fill should be benched into the slope in accordance with OPSD 208.010 and only placed on flat surfaces. Benches must be long enough to accommodate the geogrid layers. The RSS designer should specify the minimum length of geogrid required. Adequate surface drainage must be provided in the areas of the proposed temporary detour embankments. The use of subsurface drains within the reinforced embankment sections is recommended.

Typically, the faces of reinforced soil embankments are supported by wrapping the geogrids around the face and embedding the geogrid beneath the next subsequent lift of fill. In addition, a geotextile fabric may also be wrapped around the face in a similar manner to minimize losses of embankment fill through the geogrids and slope. Protection of the slope face must be provided to avoid loss of fill materials through the geogrid or geotextile facing materials and the character of the protection will depend on the desired life-span of the reinforced embankment. All geotextiles should be selected for compatibility with selected fill materials. For short-term use, the wrapped geotextile and geogrid face may be acceptable. However, some geotextiles and geogrids degrade under prolonged exposure to ultraviolet light. Further, continued exposure to freezing, thawing, wetting and drying cycles as well as heavy precipitation events may result in loss of fine soils directly through the geotextiles, imperfections in the wrapping and joints between the sheets of these geosynthetic materials, or through damage by animals. In some cases, where more permanent use of reinforced embankments is to be considered, the slope face may be provided with additional protection such as shot-crete, precast concrete panels or other materials. If the slopes are to remain as permanent structures, additional consideration to the nature of the face protection should be given during design so as to avoid adverse aesthetic or surficial stability performance issues.



Suitability of Alternative Retaining Structures

In general, the use of retaining structures, such as "toe walls", to reduce the impingement of the slope on the neighbouring channel is not recommended in this area. The height of the embankment and consequent soil loads combined with the character of the fill and native soils below the ground surface, and anticipated settlements in these critical areas render the use of conventional cantilever retaining structures impractical. Further, the use of a temporary cantilever shoring system, such as soldier piles and lagging, may not be suitable due to limited passive resistance in front of the wall (associated with the creek channel and soil conditions) combined with loads that would likely result in relatively high bending moments and displacements. Should a reinforced embankment be considered undesirable for long-term conditions, it should be feasible to construct relatively low-height (less than 4 metre) retaining walls for supporting embankment fills in these areas using a two-stage reinforced soil system wall. The first stage of wall construction consists of building the reinforced soil mass (granular fill and geo-grid, reinforcing strips, etc.) as the embankment is constructed except that no permanent facing is constructed (compared to a conventional single-stage RSS wall). Instead of a permanent facing, the RSS granular fill is supported with a temporary face consisting of a geo-grid and geotextile wrapped around the face of each fill lift. Once it has been determined that post-construction settlement will be within tolerable limits, a permanent facing is constructed in front of the temporary face and fixed to the RSS reinforcing elements to complete the wall. The ARES (Tensar) and the Terratrel (Reinforced Earth Company) are two example wall systems that can be constructed as two stage walls. With a two-stage reinforced soil system for the wall, it is thus possible to construct the reinforced soil with a temporary facing that is settlement tolerant, followed by a permanent facing once the rate and post-construction magnitude of settlement have sufficiently diminished.

6.6 Subgrade Preparation and Embankment Construction

All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankment construction. The exposed subgrade should be proofrolled prior to fill placement under the direction of qualified geotechnical personnel. In addition, all surficial topsoil and deleterious materials should be removed from the existing embankment slopes. It is not considered necessary to remove the existing embankment fills except where benching is required to join the new and existing embankments or where slope stabilization measures are required. Grading and embankment construction should be conducted in accordance with MTO Special Provision 206S03.

The new embankment fills should consist of an approved granular borrow such as Granular B Type II, rockfill or lightweight fill. Boreholes drilled through the embankment fills indicated that the existing embankment consists mainly of clayey silt borrow, with some silt layers. While it is anticipated that the new abutment construction will utilise granular fill material, inspections by qualified geotechnical personnel during construction may be required to ensure that the placement of any clayey embankment fill does not result in adverse effects on the drainage of the existing fill. Should this occur, elevated porewater pressures could develop and adversely affect the stability of the new or existing embankment side slopes. In areas where it is necessary to maintain the existing drainage of the clayey fill materials, Granular B Type I material should be placed or drains installed. If it is proposed to use cohesive fill material (clayey silt), positive drainage should be provided beneath the new embankment and between the existing and new embankments to prevent the development of excess porewater pressures. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments in accordance with Ontario Provincial Standard Drawing (OPSD) 208.010 and compacted to a minimum of 95 per cent of the material's maximum laboratory dry density as determined by the standard "Proctor" compaction test (ASTM D698).



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Lightweight fills could be utilized to minimize settlements while maintaining adequate embankment stability. Lightweight fill may consist of materials such as environmentally and physically stable blast-furnace slag, expanded shale products or expanded polystyrene foam. Each of these materials offers advantages and disadvantages in comparison with each other and natural materials. Slag fill may be the least costly but should be examined for its environmental suitability. Some slag fills may also have undesirable crushing characteristics that cause greater settlement within the embankment structure itself and adversely affect performance as a pavement subgrade. Slag fill also has the smallest ratio of fill height to weight of the lightweight fill materials. Expanded shale products (shale passed through a high temperature kiln) have a better height to weight ratio compared to slag but may not be as available in the Hamilton area. Crushing characteristics of expanded shale products should also be examined prior to selecting a particular product to evaluate their performance as a pavement subgrade material. Expanded polystyrene foam (EPS) exhibits the highest fill height to weight ratio but may be the most costly of the lightweight fill materials. Crushing characteristics (strength) of EPS must also be evaluated prior to choosing a particular product. All of these materials should be covered by a layer or layers of natural fill to separate them from landscaping or pavement structures. In addition, EPS must be protected from potential hydrocarbon spills as certain petroleum products or solvents will degrade or destroy EPS.

6.7 Excavations and Temporary Cut Slopes

The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19. The support systems may be designed using the following parameters:

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction	Unit Weight
	Active, K_a	At Rest, K_0	Passive, K_p	(degrees)	(kN/m ³)
Clayey Silt or Silt Fill	0.36	0.53	2.8	28	20
Clayey Silt/Silty Clay	0.33	0.50	3.0	28	21

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.

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 as would any cohesionless materials below the groundwater level. The native clayey materials, properly dewatered cohesionless materials, and tills would be classified as Type 2 soils.




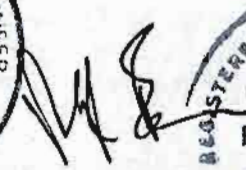
FOUNDATION INVESTIGATION AND DESIGN REPORT HIGH FILLS FOR TEMPORARY DETOUR AT HWY 403 ABERDEEN AVE


7.0 CLOSURE

This report was prepared by Mr. Neil Charters, Ms. Dirka U. Prout, P.Eng. and Dr. Storer J. Boone, 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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TABLE I

COMPARISON OF FILL MATERIAL ALTERNATIVES

High Fills for Detour during Rapid Bridge Replacement
 Highway 403 - Aberdeen Avenue Interchange
 Highway 403 Bridge Rehabilitations
 GWP 2172-06-00

FILL MATERIAL OPTION	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Conventional Fill	<ul style="list-style-type: none"> • Cheapest option • Ease and familiarity with construction methods 	<ul style="list-style-type: none"> • Maximum weight and therefore highest settlements of underlying material • May have to remove embankment following works if settlement is deemed excessive for sewer or culvert • Geogrid reinforcement necessary in some sections 	<ul style="list-style-type: none"> • Least expensive solution 	<ul style="list-style-type: none"> • Up to approximately 65 mm settlement of the 914 mm sewer pipe • May require shoring if deeper excavations impact roadway • Moderate risk of excessive total and differential settlement
Slag Fill	<ul style="list-style-type: none"> • Less expensive than polystyrene option • About 25% less settlement than conventional fill option 	<ul style="list-style-type: none"> • Geogrid reinforcement necessary in some sections • Material subject to availability from steel mills • May have to remove embankment following works if settlement is deemed excessive for sewer or culvert 	<ul style="list-style-type: none"> • Less expensive than polystyrene option 	<ul style="list-style-type: none"> • Low to moderate risk of excessive total and differential settlement

COMPARISON OF FILL ALTERNATIVES

FILL MATERIAL OPTION	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Polystyrene Fill	<ul style="list-style-type: none"> Polystyrene blocks may be re-used following dismantling the embankment Negligible settlement of sewer, culvert and riding surface Rapid construction is possible No need for geogrid reinforcement 	<ul style="list-style-type: none"> More costly than conventional fill materials May need specialist installation contractor 	<ul style="list-style-type: none"> Most expensive solution 	<ul style="list-style-type: none"> Negligible risk of excessive total or differential settlement

NOTES: 1. Table to be read in conjunction with accompanying report.

Prepared By: NC
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.)

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.

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test 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		(a) Index Properties (continued)	
π	3.1416	w	water content
$\ln x$	natural logarithm of x	w_L	liquid limit
$\log_{10} x$	x or $\log x$, logarithm of x to base 10	w_p	plastic limit
g	acceleration due to gravity	I_p	plasticity index $= (w_L - w_p)$
t	time	w_s	shrinkage limit
F	factor of safety	I_L	liquidity index $= (w - w_p)/I_p$
V	volume	I_c	consistency index $= (w_L - w)/I_p$
W	weight	e_{max}	void ratio in loosest state
II. STRESS AND STRAIN		e_{min}	void ratio in densest state
γ	shear strain	I_D	density index $= (e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)
Δ	change in, e.g. in stress: $\Delta \sigma$	(b) Hydraulic Properties	
ϵ	linear strain	h	hydraulic head or potential
ϵ_v	volumetric strain	q	rate of flow
η	coefficient of viscosity	v	velocity of flow
ν	poisson's ratio	i	hydraulic gradient
σ	total stress	k	hydraulic conductivity (coefficient of permeability)
σ'	effective stress ($\sigma' = \sigma - u$)	j	seepage force per unit volume
σ'_{vo}	initial effective overburden stress	(c) Consolidation (one-dimensional)	
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)	C_c	compression index (normally consolidated range)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$	C_r	recompression index (over-consolidated range)
τ	shear stress	C_s	swelling index
u	porewater pressure	C_a	coefficient of secondary consolidation
E	modulus of deformation	m_v	coefficient of volume change
G	shear modulus of deformation	c_v	coefficient of consolidation
K	bulk modulus of compressibility	T_v	time factor (vertical direction)
III. SOIL PROPERTIES		U	degree of consolidation
(a) Index Properties		σ'_p	pre-consolidation pressure
$\rho(\gamma)$	bulk density (bulk unit weight*)	OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$
$\rho_d(\gamma_d)$	dry density (dry unit weight)	(d) Shear Strength	
$\rho_w(\gamma_w)$	density (unit weight) of water	τ_p, τ_r	peak and residual shear strength
$\rho_s(\gamma_s)$	density (unit weight) of solid particles	ϕ'	effective angle of internal friction
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)	δ	angle of interface friction
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)	μ	coefficient of friction $= \tan \delta$
e	void ratio	c'	effective cohesion
n	porosity	c_{u, s_u}	undrained shear strength ($\phi = 0$ analysis)
S	degree of saturation	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 $= (\text{compressive strength})/2$
 * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density \times acceleration due to gravity)

PROJECT 08-1132-013.1

RECORD OF BOREHOLE No 101

1 OF 2

METRIC

W.P. 2172-06-00

LOCATION N 4790785.8 E 271737.0

ORIGINATED BY MR

DIST HWY 403

BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS/TRICONE

COMPILED BY DMB

DATUM GEODETIC

DATE April 22, 2009 - April 23, 2009

CHECKED BY

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					
86.67	GROUND SURFACE													
0.09	TOPSOIL, clayey Dark brown		1	SS	13		86							
	FILL, clayey silt, trace to some sand and gravel, topsoil, rock fragments Firm to very stiff Grey to brown		2	SS	11		85							
			3	SS	6		84							
			4	SS	9		83							
82.86							82							
3.81	FILL, silt, trace to some sand, clay, topsoil, trace rootlets, slag Loose to compact Grey to brown		5	SS	16		81							2 21 60 17
			6	SS	7		80							
			7	SS	14		79							1 2 41 56
80.73							78							
5.94	CLAYEY SILT, trace sand, trace gravel Firm Grey		8	SS	4		77							1 6 54 39
			9	SS	3		76							
			10	SS	5		75							
			11	SS	11		74							
			12	SS	17		73							0 4 69 27
			13	SS	14		72							
72.12														
14.55														

LDN MTO.01 08-1132-013.1 GPJ LDN MTO.GDT 7/21/09

Continued Next Page

+ 3 x 3

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 08-1132-013-1

RECORD OF BOREHOLE No 101

2 OF 2

METRIC

W.P. 2172-06-00

LOCATION N 4790785.8 : E 271737.0

ORIGINATED BY MR

DIST HWY 403

BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS/TRICONE

COMPILED BY DMB

DATUM GEODETIC

DATE April 22, 2009 - April 23, 2009

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					
70.67	SILT, some clay, trace sand Compact Grey		14	SS	15		71							
16.00	CLAYEY SILT, trace sand, gravel Firm Grey		15	SS	6		70							6 7 63 24
69.45	SILTY CLAY Firm to stiff Grey						69	2.3 + 2.4						
68.23	SILTY SAND AND GRAVEL Compact Brown to grey		16	SS	28		68							
18.44	SHALE BEDROCK (weathered)		17	WS	-									
67.77														
18.90														
66.95	END OF BOREHOLE		18	SS	45		67							
19.72														

PROJECT 08-1132-013.1

RECORD OF BOREHOLE No 102

1 OF 2

METRIC

W.P. 2172-06-00

LOCATION N 4790807.0 E 271739.3

ORIGINATED BY MR

DIST HWY 403

BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS/TRICONE

COMPILED BY DMB

DATUM GEODETIC

DATE April 23, 2009 - April 24, 2009

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
88.00	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.09	TOPSOIL, clayey Dark brown FILL, clayey silt, some sand, trace gravel, with topsoil pockets, rootlets Stiff Brown to grey		1	SS	11		87	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
85.87			2	SS	13		86							
2.13	FILL, silt, some topsoil, trace clay, sand		3	SS	6		85							
85.41	Loose						84							
2.59	Grey FILL, clayey silt, trace to some sand, topsoil Stiff to very stiff Brown		4	SS	9		83							
83.58			5	SS	3		82							
4.42	FILL, clayey silt Soft to firm Brown		6	SS	4		81							
			7	SS	7		80							
			8	SS	11		79							
80.53			9	SS	3		78							
7.47	CLAYEY SILT, trace sand, trace gravel Firm to very stiff Grey		10	SS	5		77							
			11	SS	7		76							
			12	SS	11		75							
			13	SS	15		74							

Continued Next Page

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

PROJECT 08-1132-013-1

RECORD OF BOREHOLE No 104

1 OF 2

METRIC

W.P. 2172-06-00

LOCATION N 4790786.6, E 271680.3

ORIGINATED BY T2

DIST HWY 403

BOREHOLE TYPE WASH BORING/BW CASING

COMPILED BY DMB

DATUM GEODETIC

DATE April 22, 2009 - April 27, 2009

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
87.63 0.00	GROUND SURFACE FILL, clayey silt, trace gravel, trace sand Very stiff to soft Brown to grey													
			1	SS	16									
			2	SS	25									
			3	SS	18									
			4	SS	11									0 3 57 30
			5	SS	21									
			6	SS	3									
80.92 6.71	SILTY CLAY, trace gravel, trace sand Firm Grey		7	SS	5									
			8	SS	4									
			9	SS	3									
			10	SS	3									0 3 36 61
			11	SS	7									
75.44 12.19	CLAYEY SILT, trace gravel, trace sand Firm to hard Grey		12	SS	31									
			13	SS	42									
			14	SS	35									
			15	SS	27									0 3 65 32

LDN_MTO_01 08-1132-013-1 GPJ LDN_MTO_GDT 7509

Continued Next Page

+ 3 x 3

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 08-1132-013-1

RECORD OF BOREHOLE No 105

1 OF 1

METRIC

W.P. 2172-06-00

LOCATION N 4790777.8; E 271675.4

ORIGINATED BY TZ/MR

DIST HWY 403

BOREHOLE TYPE WASH BORING/BW CASING

COMPILED BY DMB

DATUM GEODETIC

DATE April 28, 2009

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					
87.17 0.00	GROUND SURFACE FILL, clayey silt, trace sand, trace gravel, containing organics Very stiff to hard Brown to grey					V							
			1	SS	29								
			2	SS	37								
			3	SS	16								
			4	SS	22								
			5	SS	17								
			6	SS	19								
61.68 5.49	CLAYEY SILT, trace sand, gravel Firm to very stiff Grey		7	SS	4								
			8	SS	4								
			9	SS	6								
			10	SS	19								
			11	SS	24								
			12	SS	28								
73.00 14.17	END OF BOREHOLE Groundwater encountered at about elev. 85.7m during drilling on April 28, 2009.												

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

LDN_MTO_01_08-1132-013-1.GPJ LDN_MTO.GDT 7/9/09

PROJECT 08-1132-013-1 RECORD OF BOREHOLE No **106** 1 OF 1 **METRIC**
W.P. 2172-06-00 LOCATION N 4790762 2 E 271688 1 ORIGINATED BY TZ/MR
DIST HWY 403 BOREHOLE TYPE WASH BORING/BW CASING COMPILED BY DMB
DATUM GEODETIC DATE April 29, 2009 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			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE	20
87.72 0.00	GROUND SURFACE FILL, clayey silt, trace gravel, trace sand, containing organics Stiff to very stiff Brown to grey																	
			1	SS	18		87											
			2	SS	15		86								1 4 51 24			
			3	SS	24		85											
			4	SS	10		84											
			5	SS	10		83											
			6	SS	24		82											
			7	SS	11		81											
			8	SS	12		80											
80.40 7.32	CLAYEY SILT, trace gravel, trace sand Firm to hard Grey		9	SS	7		79											
			10	SS	15		78								9 6 49 36			
			11	SS	43		77											
			12	SS	28		76											
			13	SS	41		75											
74.77 12.95	END OF BOREHOLE		14	SS	17													

LDN_MTO_01_08-1132-013-1.GPJ LDN_MTO.GDT 6/19/09

PROJECT: 08-1132-013-1

RECORD OF CONE PENETRATION TEST CPT-101

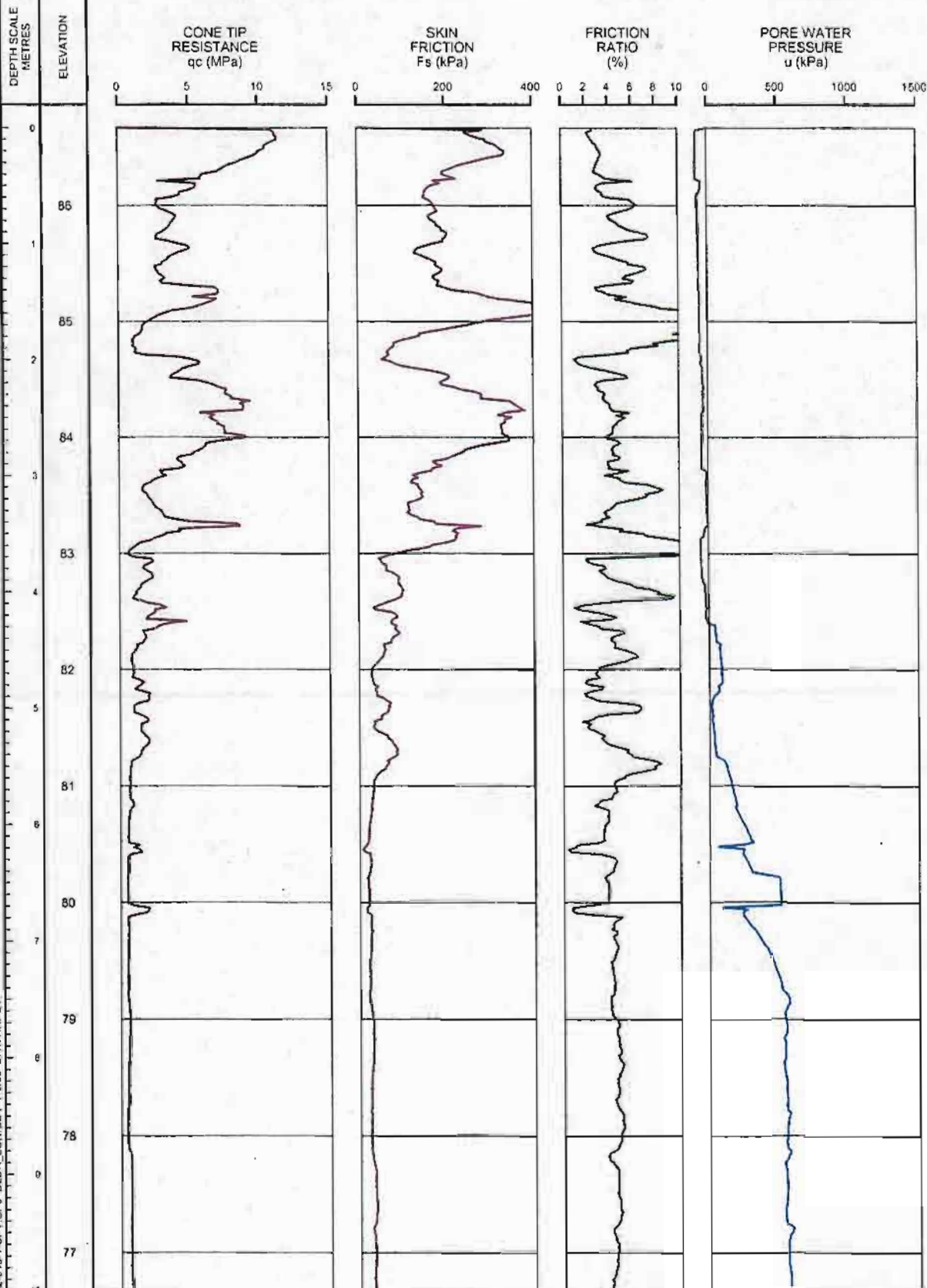
SHEET 1 OF 2

LOCATION: N 4790792 5 E 271747.1

TEST DATE: April 29, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 86.67m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



--- CONTINUED NEXT PAGE ---

DEPTH SCALE

1:50



OPERATOR: MWK

CHECKED: *[Signature]*

PROJECT: 08-1132-013-1

RECORD OF CONE PENETRATION TEST CPT-101

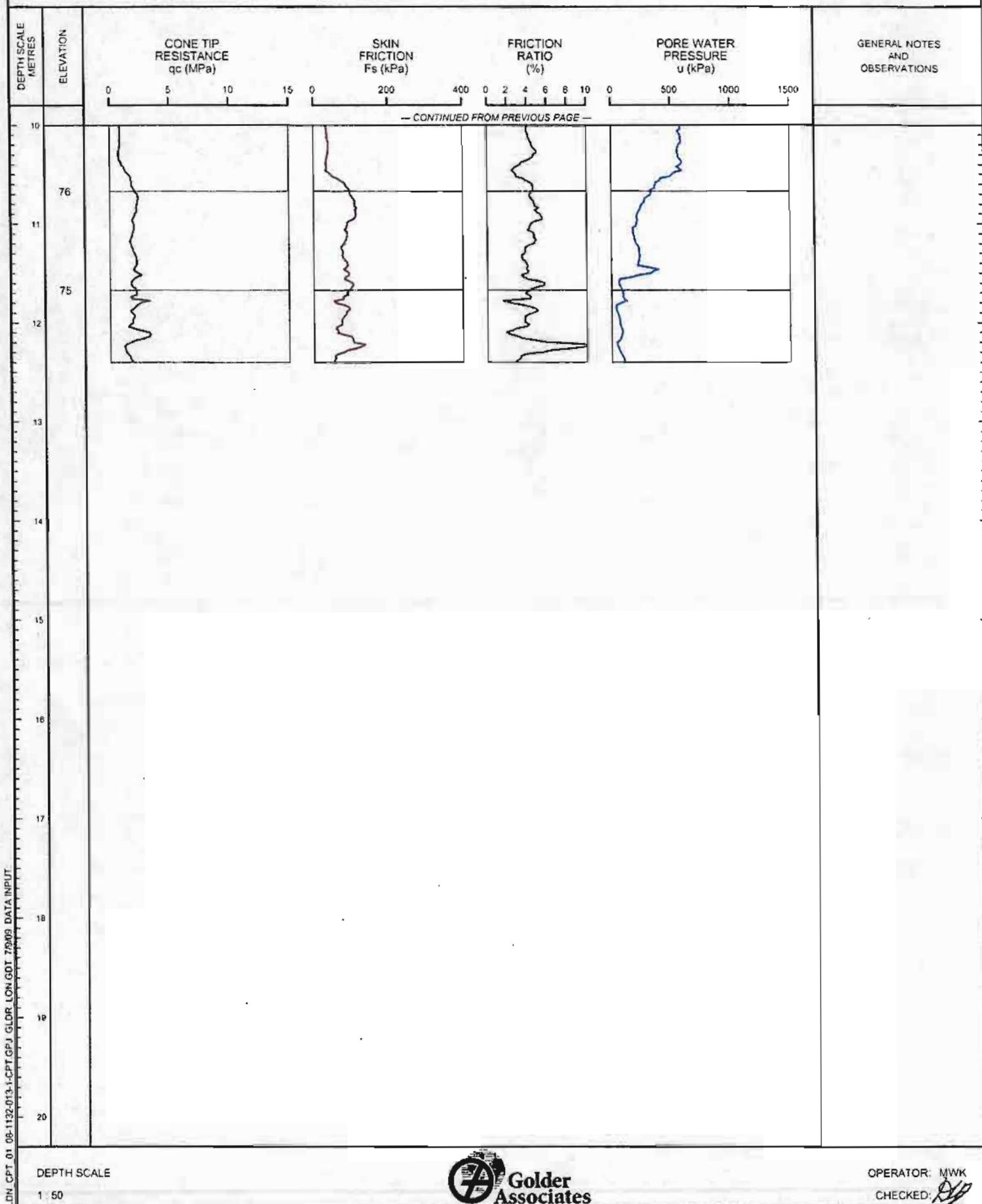
SHEET 2 OF 2

LOCATION: N 4790792.5, E 271747.1

TEST DATE: April 29, 2009

DATUM: GEODETIC

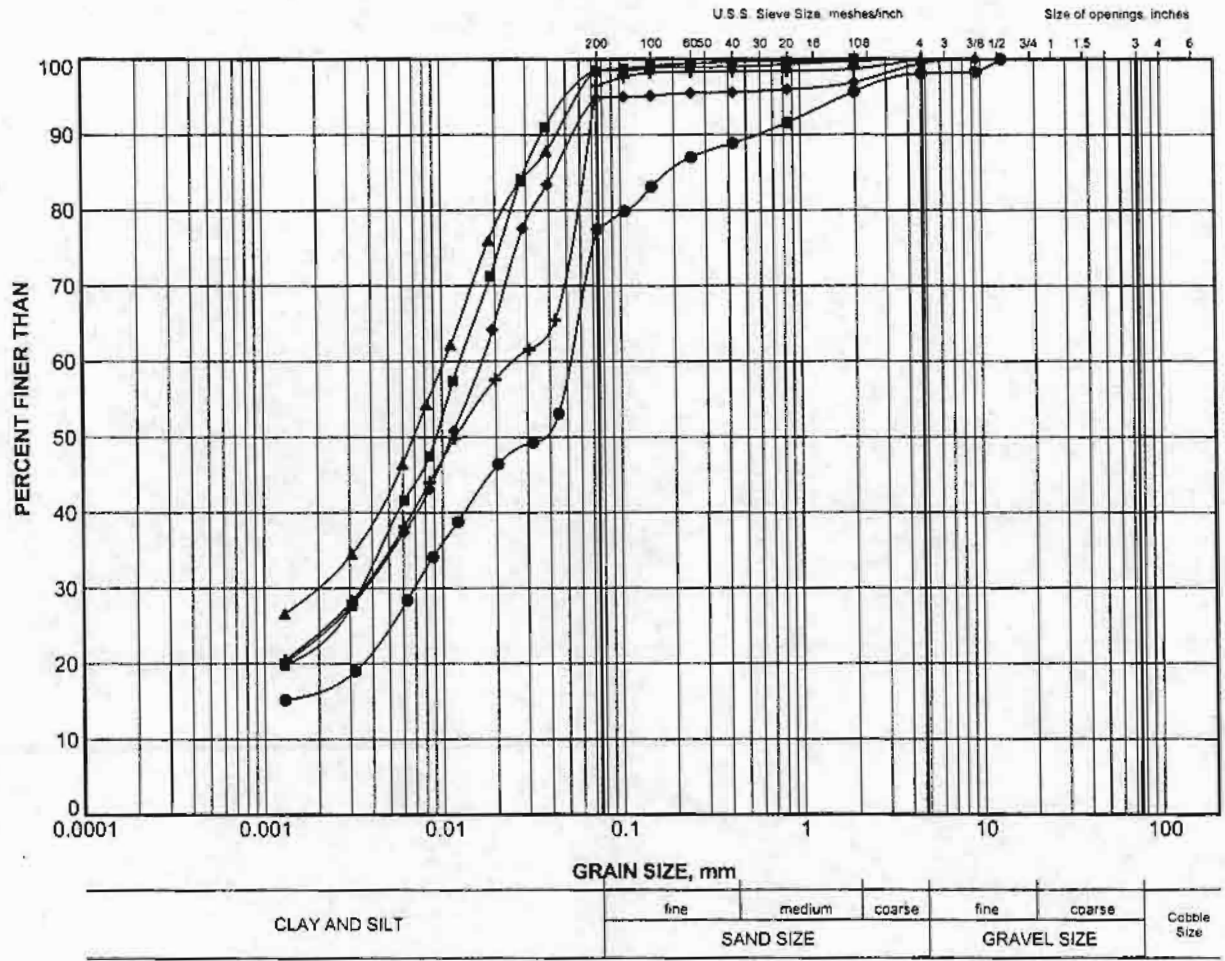
GROUND SURFACE ELEVATION: 76.67m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012





APPENDIX A

Laboratory Test Data (Current Investigation)



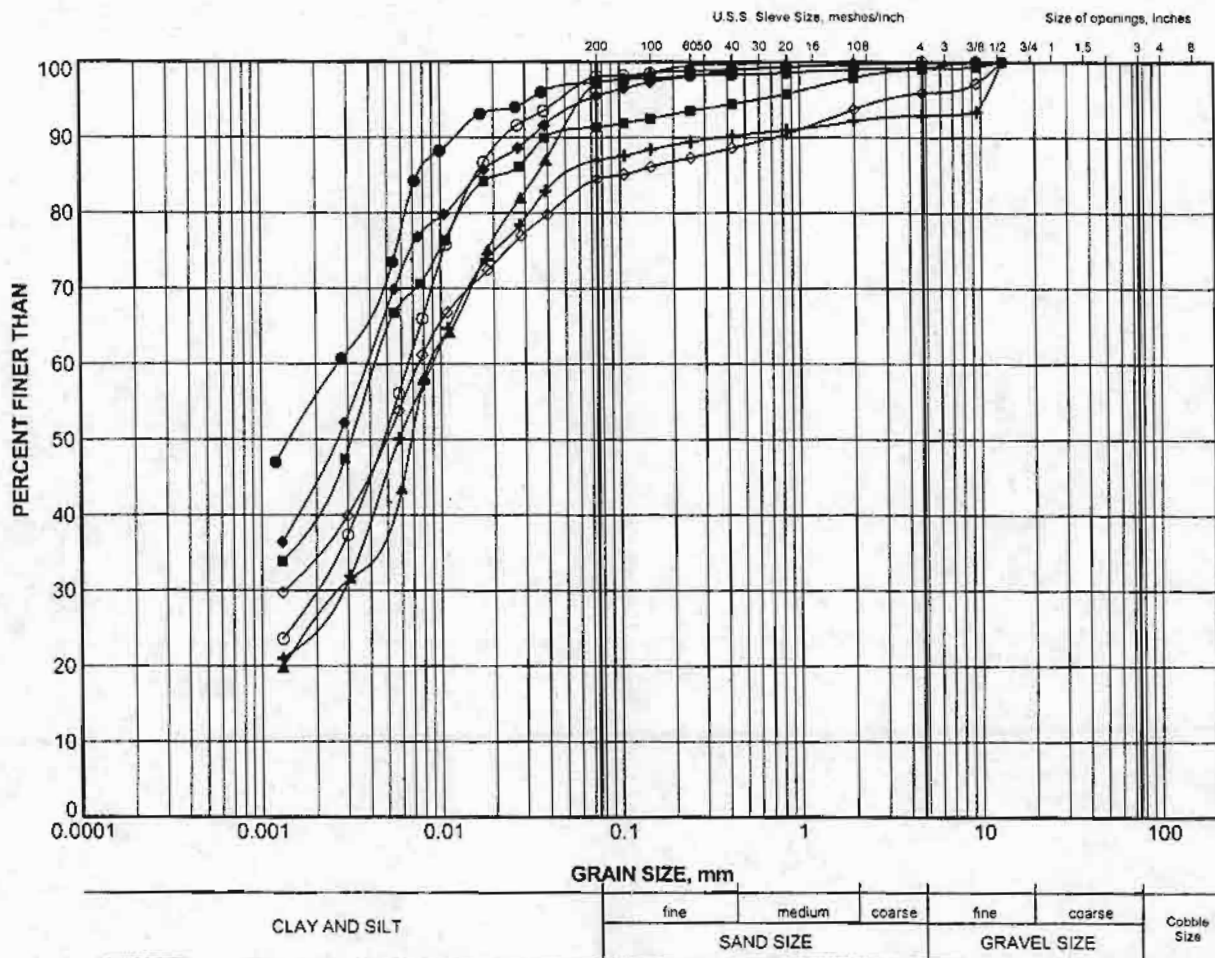
LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	7	81.1
■	102	6	82.4
▲	104	4	84.3
+	105	4	83.8
◆	106	2	86.2

PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE
GRAIN SIZE DISTRIBUTION
FILL

PROJECT No. 08-1132-013-1		FILE No. C811320131-R010A1	
DRAWN LMK		SCALE N/A	
CHECK [Signature]		REV.	
Golder Associates LONDON, ONTARIO		FIGURE A-1	

LDN.MTO.NEW.GLDR.LDN.GDT



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	9	78.8
■	101	10	77.5
▲	101	13	72.9
+	101	15	69.8
◆	102	9	80.2
◇	102	11	77.3
○	102	15	71.2

PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE

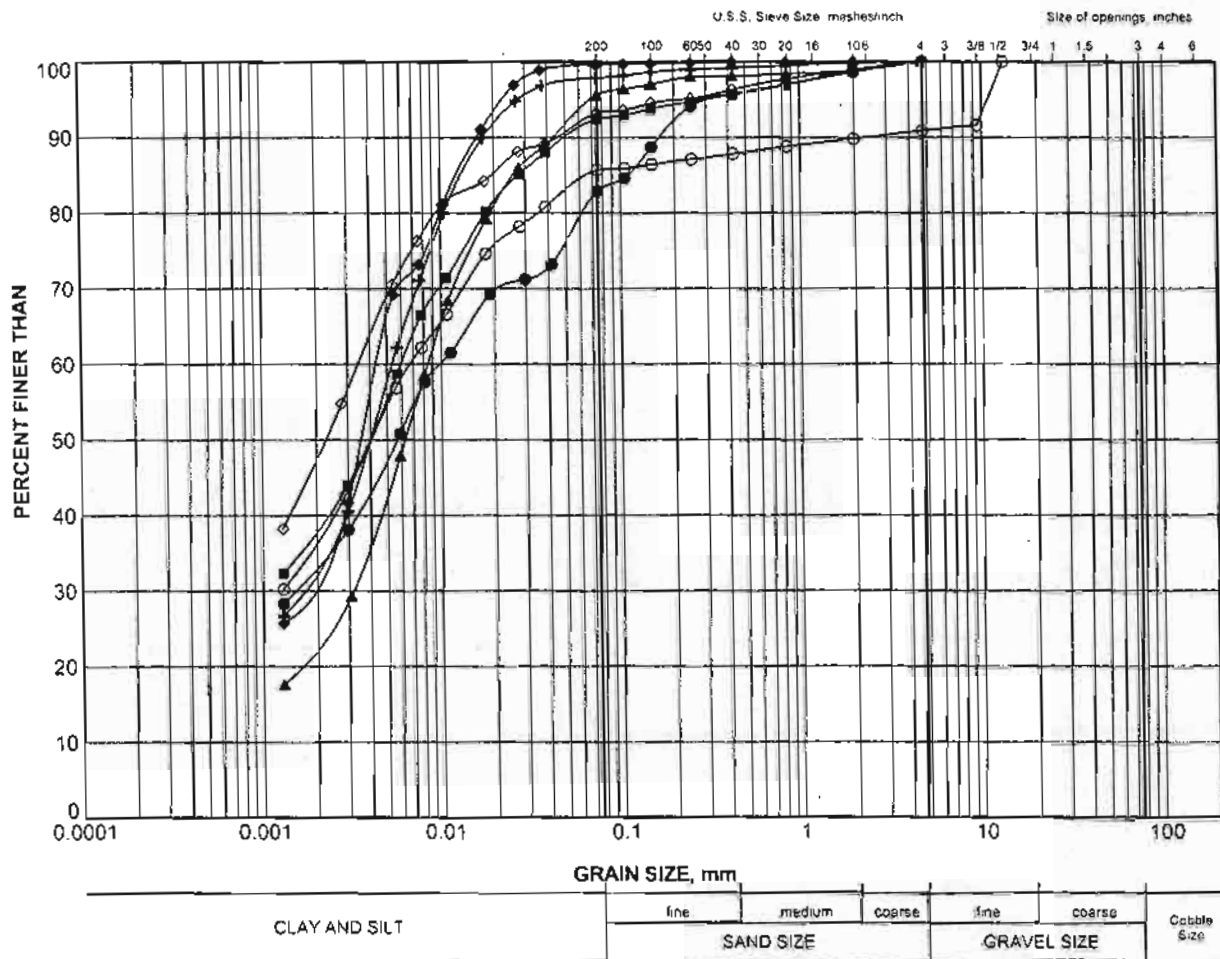
GRAIN SIZE DISTRIBUTION CLAYEY SILT



**Golder
Associates**
LONDON, ONTARIO

PROJECT No.	09-1132-013-1	FILE No.	0811320131-R010A2
DRAWN	LMK	DATE	JUL 21/09
CHECK	[Signature]	DATE	JUL 21/09
SCALE	N/A	REV	

FIGURE A-2



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	103	3	84.2
■	103	6	80.4
▲	103	12	71.6
+	104	15	72.7
◆	104	18	69.0
◇	105	8	79.2
○	106	10	78.3

PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

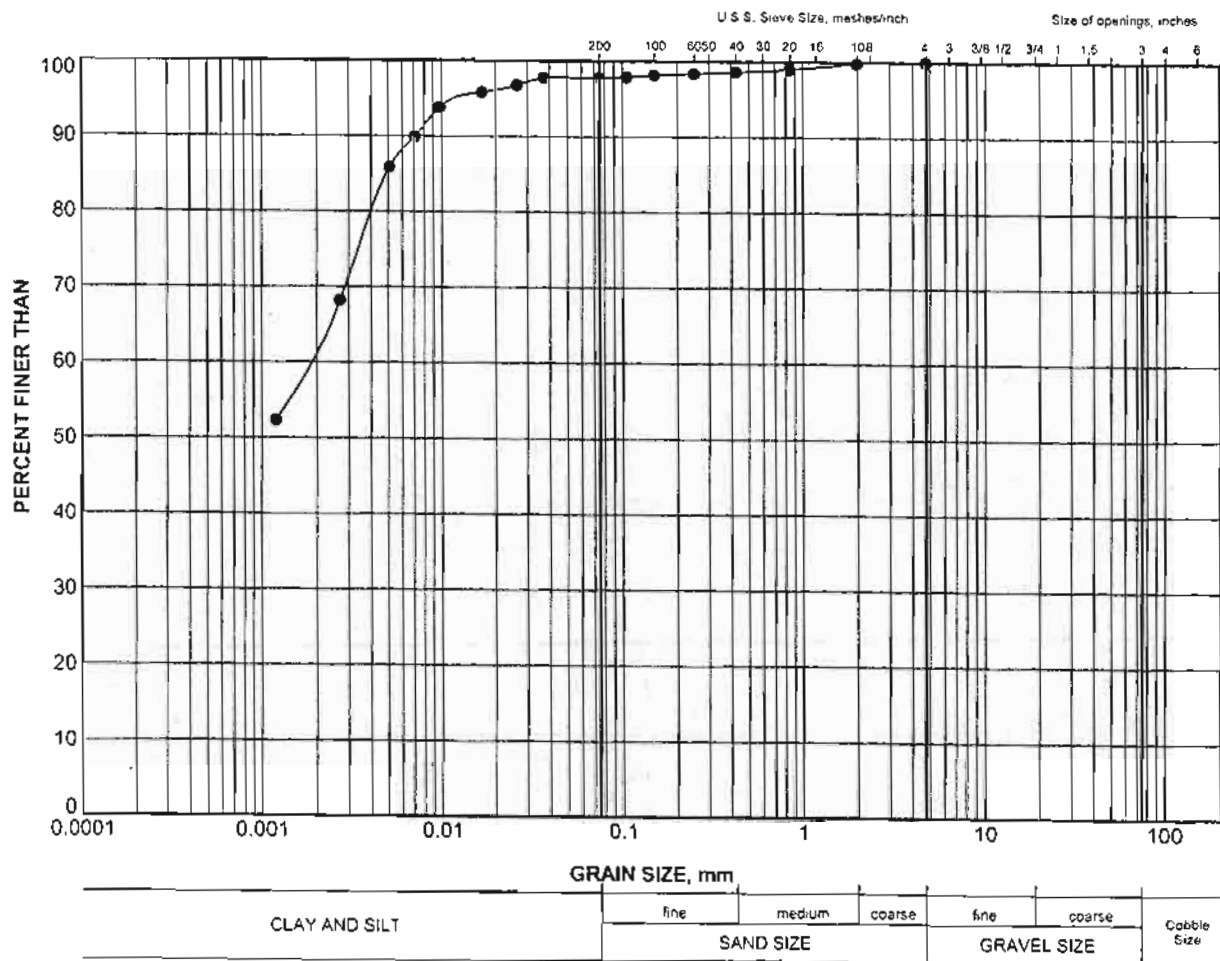
TITLE

GRAIN SIZE DISTRIBUTION CLAYEY SILT



PROJECT No	08-1132-013-1	FILE No	0811320131-R010A3
DRAWN	LMK	SCALE	N/A
CHECK	JUL 21/09	REV	

FIGURE A-3



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
•	104	10	78.2

PROJECT
 HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
 HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
 HIGHWAY 403 BRIDGE REHABILITATION

TITLE
GRAIN SIZE DISTRIBUTION
SILTY CLAY


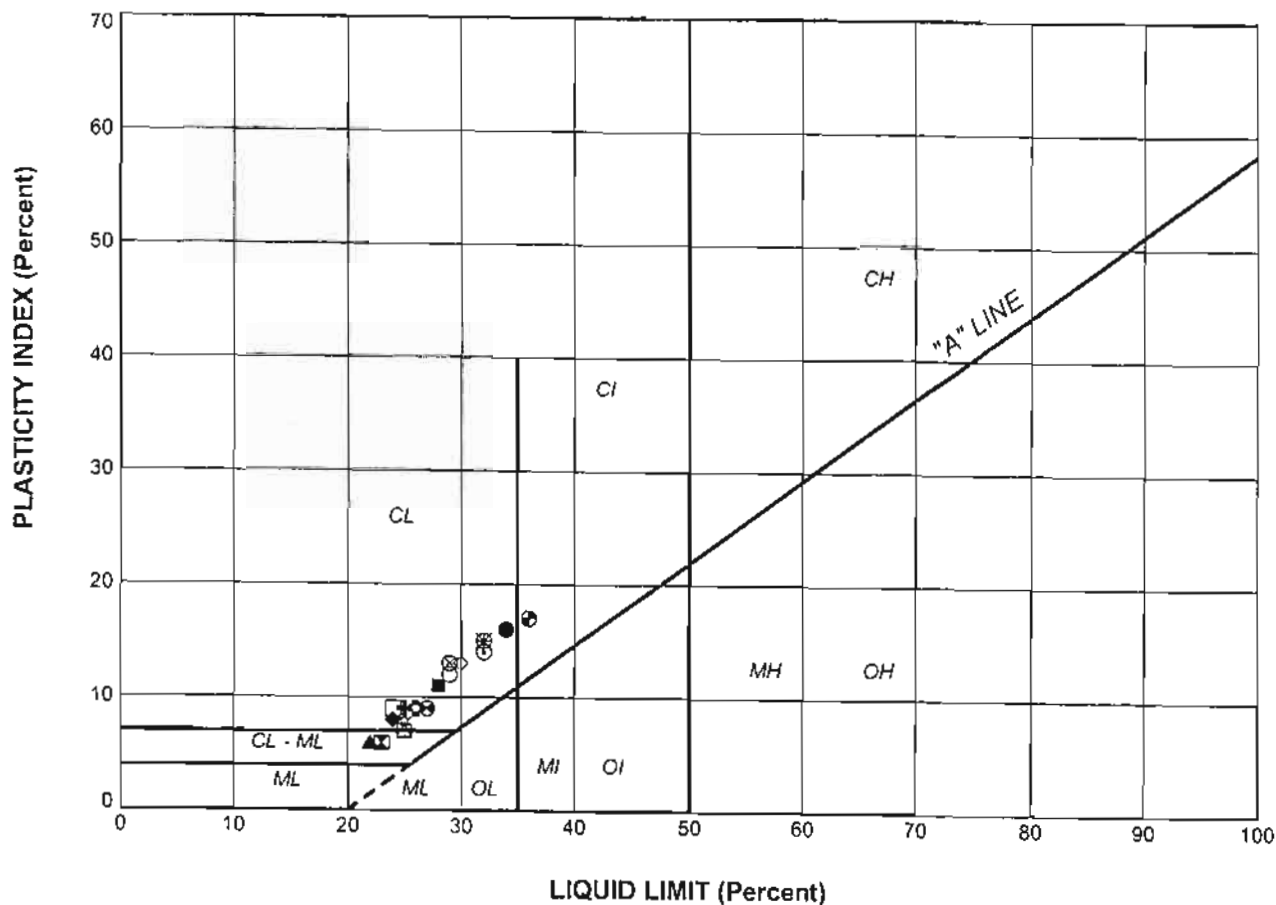
 Golder Associates LONDON, ONTARIO	PROJECT No.	08-1132-0013-1	FILE No.	0811320131-R010A4	
	DRAWN	LMK	JUL 01/08	SCALE	N/A
	CHECK	<i>[Signature]</i>	JUL 21/04	REV	

FIGURE A-4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT					
●	101	9	34.0	18.0	16.0
■	101	10	28.0	17.0	11.0
▲	101	13	22.0	16.0	6.0
+	101	15	25.0	16.0	9.0
○	102	9	30.0	17.0	13.0
○	102	11	29.0	17.0	12.0
△	102	15	25.0	18.0	7.0
⊗	103	3	29.0	16.0	13.0
⊗	103	6	32.0	17.0	15.0
□	103	12	24.0	15.0	9.0
*	104	15	25.0	17.0	8.0
⊗	104	18	25.0	18.0	7.0
⊗	105	8	32.0	18.0	14.0
x	106	10	32.0	17.0	15.0
SILTY CLAY					
⊗	104	10	36.0	19.0	17.0
FILL					
◆	102	6	24.0	16.0	8.0
⊗	104	4	27.0	18.0	9.0
⊗	105	4	23.0	17.0	6.0
⊗	106	2	26.0	17.0	9.0

PROJECT
 HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
 HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
 HIGHWAY 403 BRIDGE REHABILITATION

TITLE

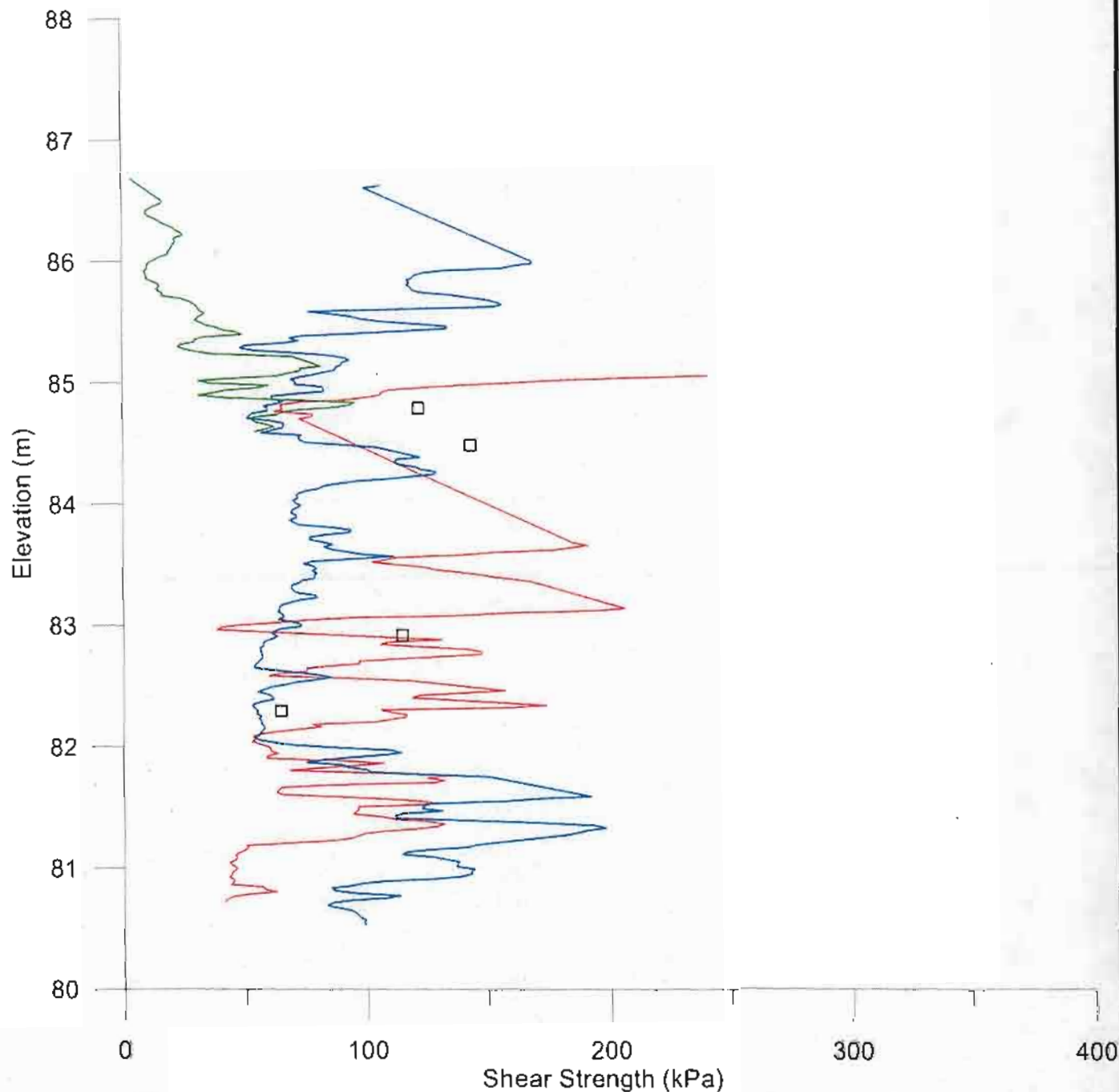
PLASTICITY CHART



Golder Associates
 LONDON, ONTARIO

PROJECT No	08-1132-0013-1	FILE No	0811320131-R01045
DRAWN	LMK	JUL 8/09	SCALE N/A
CHECK	247	JUL 21/09	REV

FIGURE A-5



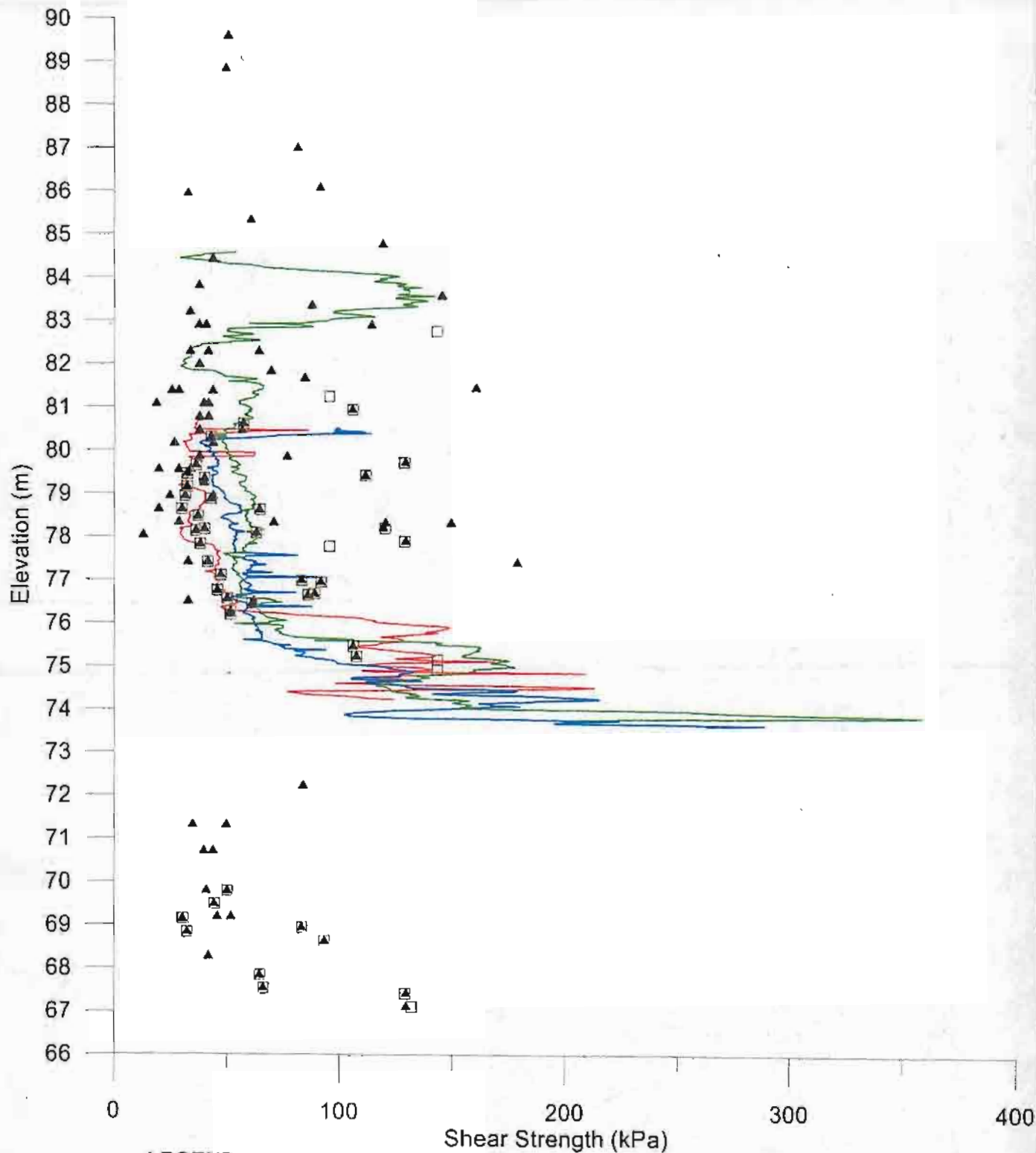
PROJECT
 HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
 HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
 HIGHWAY 403 BRIDGE REHABILITATION

TITLE

**SHEAR STRENGTH PROFILE
 (Fill)**



PROJECT NO. 08-1132-013-1		FILE NO. 08-1320131-RD-CAR	
DRAWN DUP		SCALE N/A REV. 0	
CHECK <i>md</i>		JUL 21/08	
		FIGURE A-6	



PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE
SHEAR STRENGTH PROFILE
(Clayey Silt and Silty Clay)



PROJECT No.	08-1132-013-1	FILE No.	0811320131-R010A7
DRAWN	DUP	JUL 21/09	SCALE
CHECK			N/A REV. 0

FIGURE A-7



APPENDIX B

Previous Borehole, Laboratory and Field Data
(Geocres No. 30M5-31)

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

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

BORE HOLE NO. 61

JOB 60-F-14

STATION B/35 Ramp 'D' 20' Rt

DATUM G.S.C.

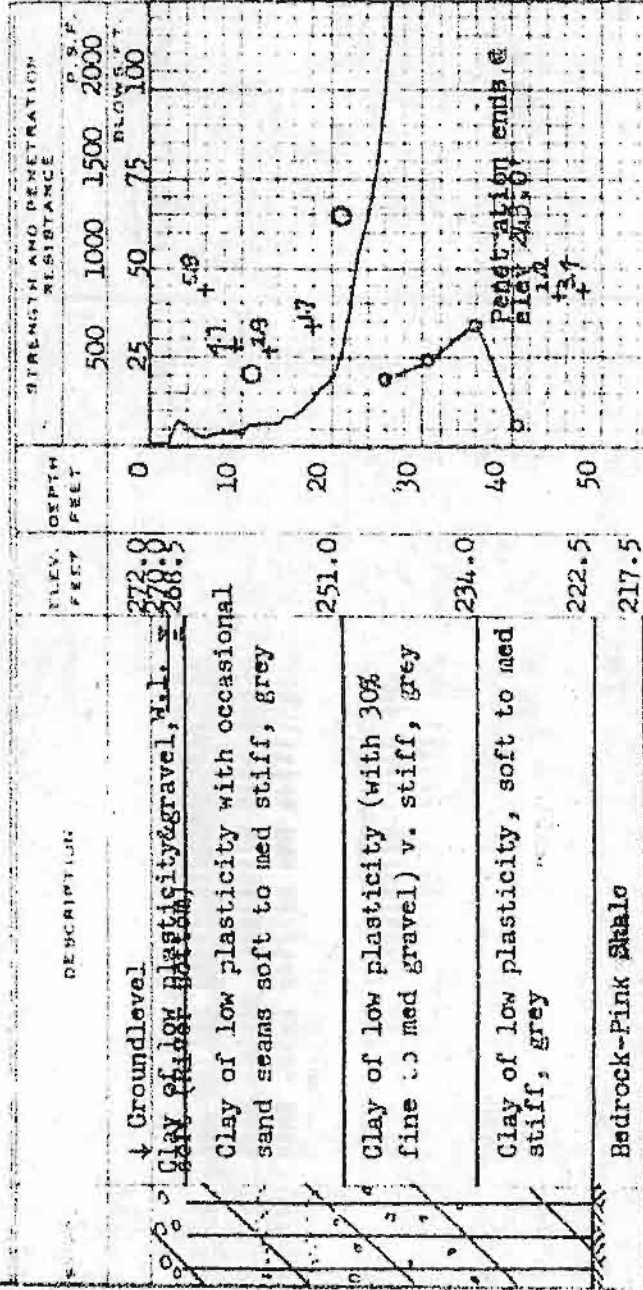
COMPILED BY B.K.

BORING DATE Apr. 4/60

CHECKED BY K.S. & MD

LEGEND

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

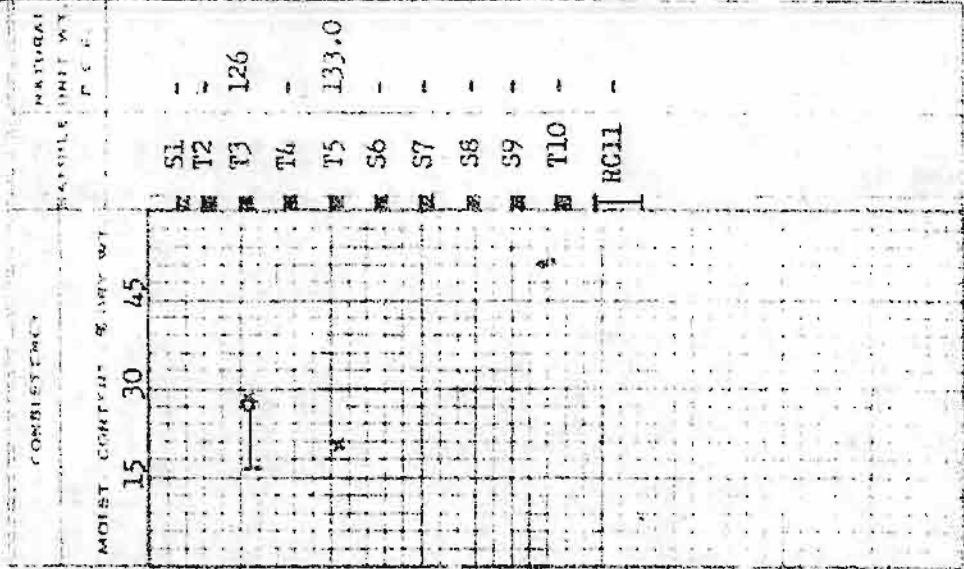


End of borehole

Note: Artesian Head at 222.5 approx. 20' head at G.L.

Normal water table = 270.0

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



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

140-57-1
231-58-3
231-58-2

BORE HOLE NO. 62

JOB 60-P-24

STATION 9+00 Ramp 'D' 90'

DATUM G.S.C.

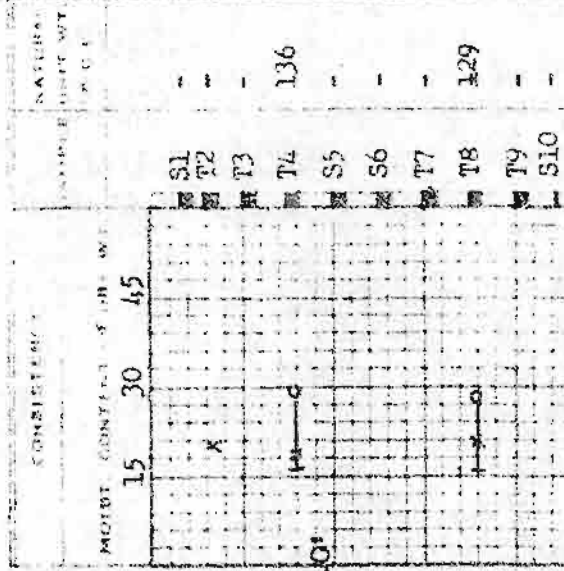
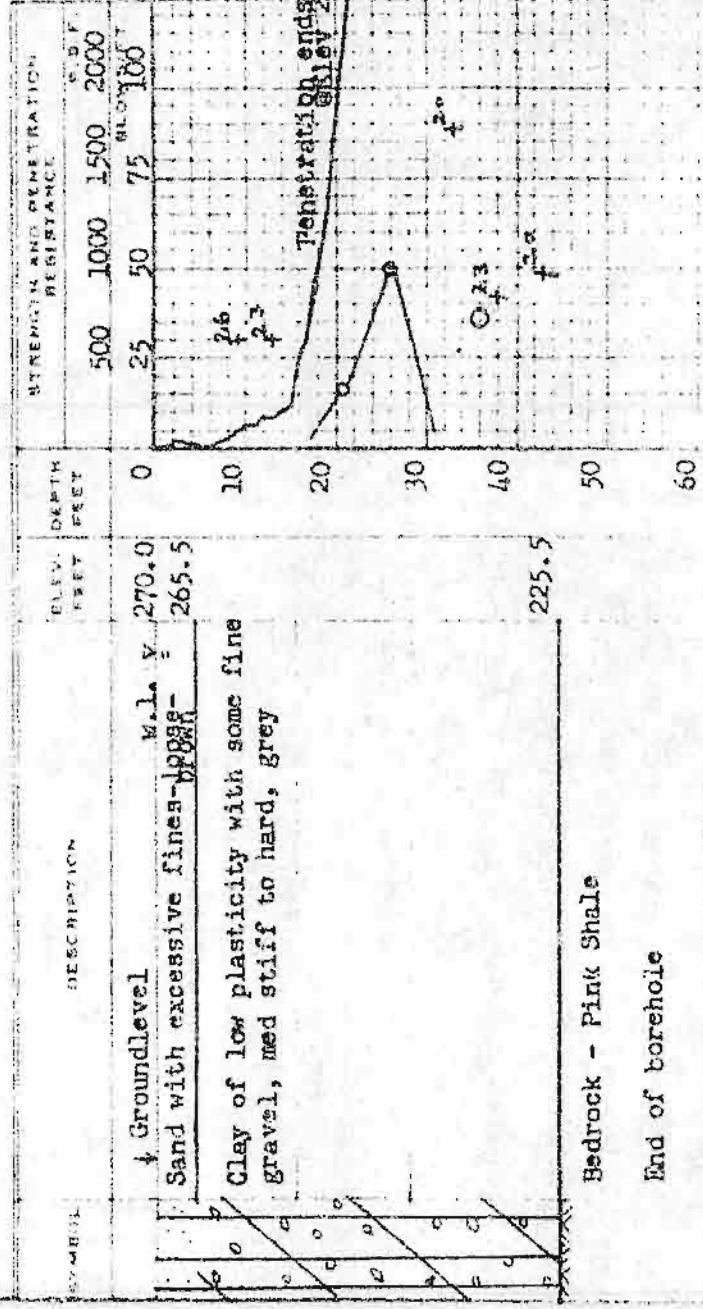
COMPILED BY B.K.

BORING DATE Apr. 7/60

CHECKED BY K.D.

LEGEND

- 1/2 UNCONFINED COMPRESSION (QU) - O
- VANE TEST (C) AND SENSITIVITY (S) - T
- NATURAL MOISTURE AND LIQUIDITY INDEX - L
- LIQUID LIMIT - LL
- PLASTIC LIMIT - PL



Bedrock - Pink Shale

End of borehole

Artesian Head at EL. 225.5

Normal Water Table 270.0

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

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

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

BORE HOLE NO. 63

JOB 60-F-14

STATION 478+80 E.B.L. 130' RT 2" DIA. SPLIT TUBE

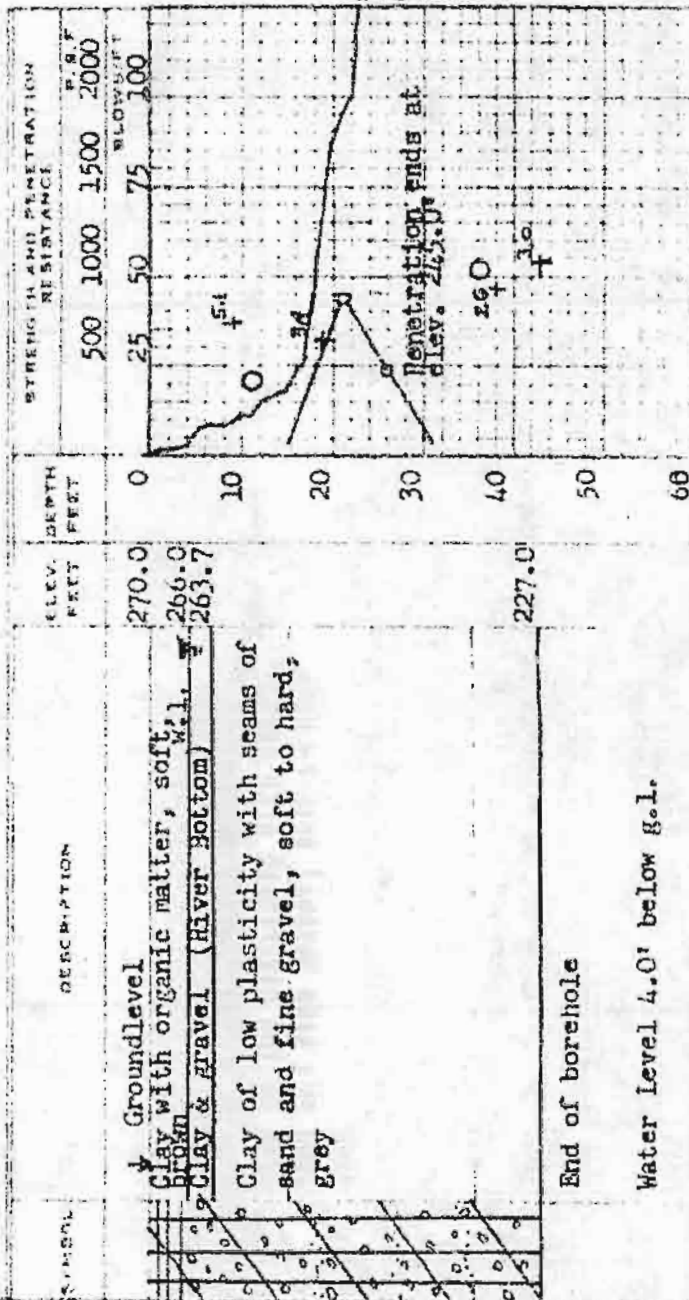
DATUM U.S.C.

COMPILED BY B.K.

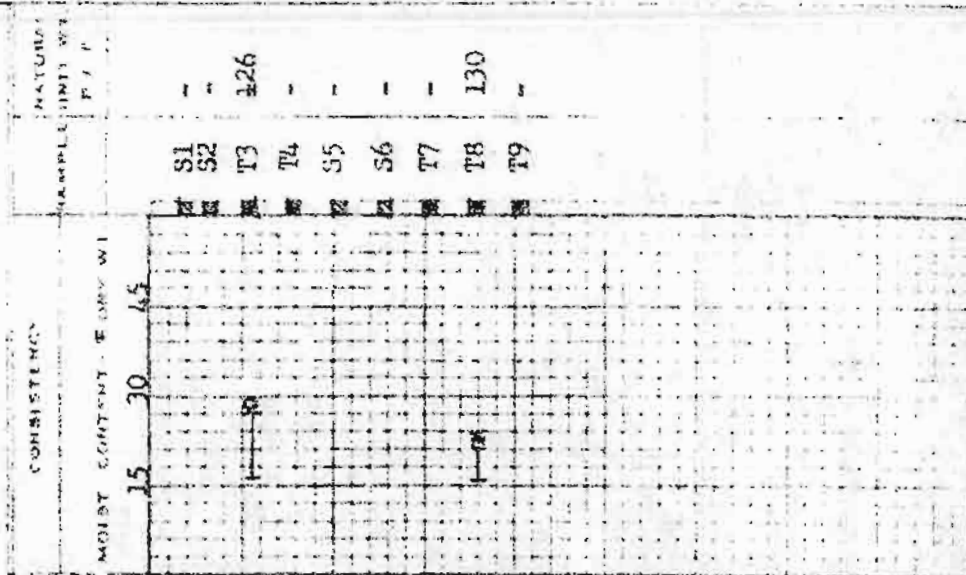
BORING DATE APR. 19/60 CHECKED BY K.S. & K.D.

LEGEND

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



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

HOLE NO	SAMP NO	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETRM RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH (psi)	UNIT WEIGHT (pcf)	REMARKS
61	S1	3'-4.5'	Clay of low plasticity, with thin seams of sand-M stiff-grey	P	-	-	-	-	-	
	VANE	6'		-	-	-	-	880	-	Sens: 5.9
	T2	6'-7.5'	Clay of low plasticity, soft, grey	P	-	-	-	-	-	
	VANE	9'		-	-	-	-	560	-	Sens: 4.7
	T3	10'-11.5'	Clay of low plasticity, soft, grey	P	28.2	16.2	27.7	410	126	
	VANE	13'		-	-	-	-	520	-	Sens: 2.9
	T4	15'-16.5'	" " " "	P	-	-	-	-	-	
	VANE	18'		-	-	-	-	680	-	Sens: 1.7
	T5	20'-21.5'	Clay of low plasticity, Med stiff, grey	P	20.5	-	-	1290	133	
	S6	25'-26.5'	Clay of low plasticity containing some fine gravel V. stiff, grey	19	-	-	-	-	-	
	S7	30'-31.5'	" " " "	23	-	-	-	-	-	
	S8	35'-36.5'	" " " "	33	-	-	-	-	-	
	S9	40'-41.5'	Clay of low plasticity, med stiff, grey	7	-	-	-	-	-	
	VANE	43'		-	-	-	-	860	-	Sens: 2.2
	T10	45'-46.5'	Clay of low plasticity, med stiff, grey	P	-	-	-	-	-	

SUMMARY OF FIELD & LABORATORY TESTS

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

HOLE NO	SAMP NO	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETR RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
61	VARE	48'		-	-	-	-	880	-	Spec: 3.4
	RC11	49.5'-54.5'	Axt core: Shale bed Rock	-	-	-	-	-	-	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-14
140-57-1
W.P. 231-56-2
231-58-3

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETN RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHRINK. STRENGTH %	UNIT WEIGHT pcf	REMARKS
62	S1	3'-4.5'	Sand with excessive fines and traces of organic matter- brown	F	-	-	-	-	-	
	T2	6'-7.5'	Clay of low plasticity, Med stiff, grey	P	20.2	-	-	-	-	
	VANE 9'			-	-	-	-	600	-	Sens: 2.6
	T3	10'-11.5'	" " "	P	-	-	-	-	-	
	VANE 13'			-	-	-	-	600	-	Sens: 2.3
	T4	15'-16.5'	Clay of low plasticity, with some fine gravel v. stiff, grey	P	18.2	16.3	29.5	3740	136	
	S5	20'-21.5'	" " "	17	-	-	-	-	-	
	S6	25'-26.5'	Clay of low plasticity, hard, grey	50	-	-	-	-	-	
	T7	30'-31.5'	Clay of low plasticity with traces of fine gravel, stiff, grey	P	-	-	-	-	-	
	VANE 33'			-	-	-	-	1760	-	Sens: 2.0
	T8	35'-36.5'	Clay of low plasticity, med stiff, grey	P	20.8	16.0	28.1	730	128	
	VANE 38'			-	-	-	-	840	-	Sens: 2.3
	T9	40'-41.5'	" " "	P	-	-	-	-	-	
	VANE 43'			-	-	-	-	960	-	Sens: 3.0
	S10	44.5'-44.6'	Weathered shale, red	Refusal	-	-	-	-	-	

JOB 60-F-14
LOG-57-1
W.P. 211-58-2
231-58-3

SUMMARY OF FIELD & LABORATORY TESTS

HOLE NO	SAMP NO	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETN RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT PCF	REMARKS
63	S1	3'-4.5'	Clay of low plasticity containing 50% gravel	P	-	-	-	-	-	
	S2	6'-7.5'	Clay of low plasticity with some fine gravel med stiff, grey	P	-	-	-	-	-	
	VANE	9'		-	-	-	-	720	-	Sens: 5.1
	T3	10'-11.5'	Clay of low plasticity with some fine gravel, soft, grey	P	26.6	16.2	28.1	420	126	
	VANE	19'		-	-	-	-	680	-	Sens: 3.4
	T4	15'-16.5'	Clay of low plasticity with seams of sand, med stiff, grey	P	-	-	-	-	-	
	S5	20'-21.5'	Clay of low plasticity with pockets of silt, hard, grey	43	-	-	-	-	-	
	S6	25'-26.5'	Clay of low plasticity, v. stiff, grey	23	-	-	-	-	-	
	T7	30'-31.5'	Clay of low plasticity, stiff, grey	P	-	-	-	-	-	
	T8	35'-36.5'	" " " "	P	22.8	15.9	22.9	1040	120	
	VANE	38'		-	-	-	-	920	-	Sens: 2.6
	T9	40'-41.5'	Clay of low plasticity, med stiff, grey	P	-	-	-	-	-	
	VANE	43'		-	-	-	-	1080	-	Sens: 3.0



**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGH FILLS FOR TEMPORARY DETOUR AT HWY 403 ABERDEEN AVE**

APPENDIX C

Photographs



APPENDIX C HIGH FILLS FOR TEMPORARY DETOUR AT HIGHWAY 403 ABERDEEN AVE INTERCHANGE



Photo 1: Temporary detour area west of Ramp 'D'. Highway 403 and Ramp 'D' underpass in background.



APPENDIX C
HIGH FILLS FOR TEMPORARY DETOUR AT HIGHWAY 403
ABERDEEN AVE INTERCHANGE



Photo 2: 2-3 millimetre wide crack on wall, 25 metres upstream/west of outlet of Ramp 'D' Chedoke Creek culvert.



Photo 3: North wall of Ramp 'D' Chedoke Creek culvert approximately 41 metres upstream/west of outlet. Crack is about 10 millimetres wide.



APPENDIX C
HIGH FILLS FOR TEMPORARY DETOUR AT HIGHWAY 403
ABERDEEN AVE INTERCHANGE



Photo 4: South wall of Ramp 'D' Chedoke Creek culvert,
about 41 metres upstream/west of outlet.



APPENDIX C
HIGH FILLS FOR TEMPORARY DETOUR AT HIGHWAY 403
ABERDEEN AVE INTERCHANGE



Photo 5: Inlet of Ramp 'D' Chedoke Creek culvert.



Photo 6: Sinkhole due to erosion/loss of ground just northeast of Ramp 'D' Chedoke Creek culvert.



APPENDIX C HIGH FILLS FOR TEMPORARY DETOUR AT HIGHWAY 403 ABERDEEN AVE INTERCHANGE



Photo 7: Outlet of Ramp 'D' Chedoke Creek culvert.

n:\active\2008\1132 - geotechnical\1132-000-0\08-1132-013-1 mh - addnl fdns - hwy 403 & 6\reports\0811320131-r01 - high fills\0811320131-r01 aug 20 09 - (final) appendix c - aberdeen
fdn alts high fill.docx

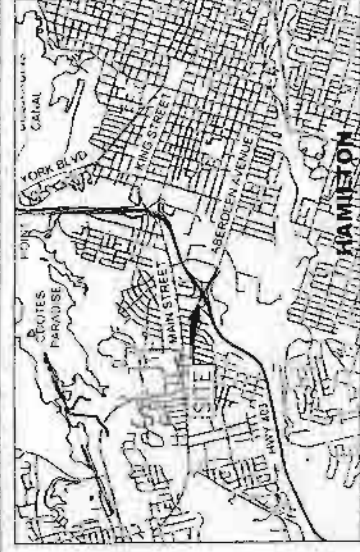
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2172-06-00

HIGHWAY 403 ABERDEEN AVENUE
INTERCHANGE
HIGH TILLS FOR DETOUR -
RAPID BRIDGE REPLACEMENT
SOIL STRATA



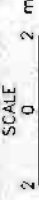
Golder Associates Ltd.
LONDON, ONTARIO, CANADA






KEY PLAN



SECTION ALONG A-A



LEGEND

- | | |
|---|--|
|  | Borehole -- Current Investigation |
|  | Standard Penetration Test Value |
| N | Borehole and Piezocone -- Current Investigation |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 lbf/ft) |
|  | WL upon completion of drilling |
| DRY | Borehole dry during drilling |

No.	ELEVATION	CO-ORDINATES (MIM. ZONE 10)	
		NORTHING	EASTING
101	86.67	4 790 785.8	271 737.0
102	88.00	4 790 807.0	271 739.3
104	87.63	4 790 786.6	271 680.3
105	87.17	4 790 777.8	271 675.4
106	87.72	4 790 762.2	271 668.1



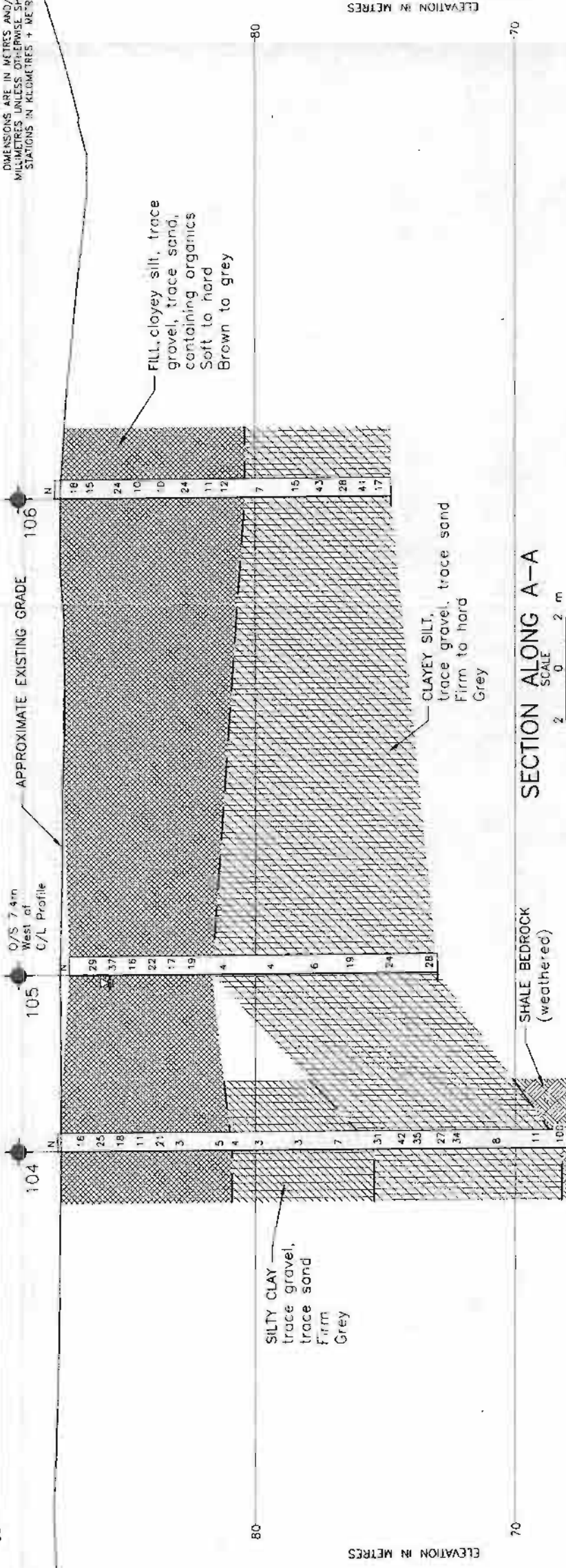
NOTES

The new word is not a complete interference from any. So far, nothing has been added to the existing vocabulary.

REFERENCE

$\frac{d}{dt} \left(\frac{\partial L}{\partial \dot{x}} \right) = \frac{\partial L}{\partial x}$

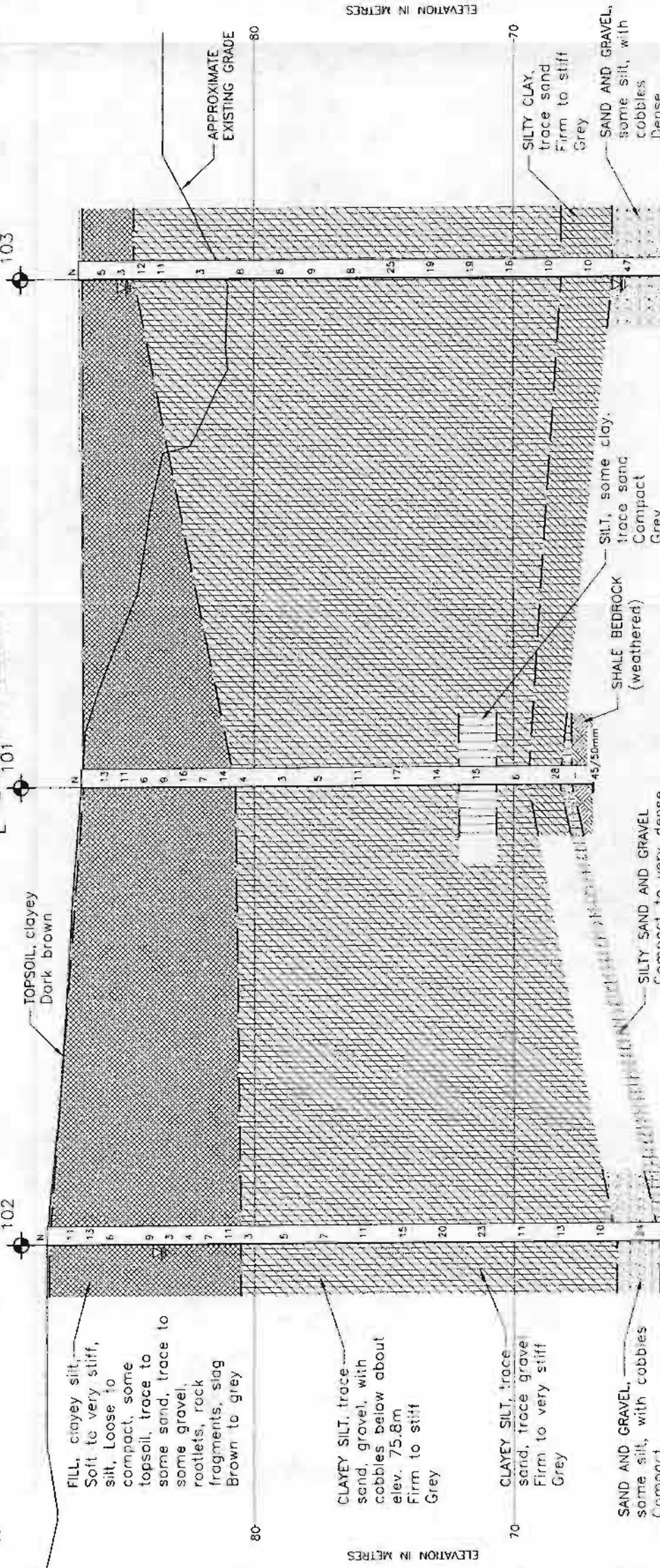
NO.	DATE	BY	REVISION
Geotris No. 30MDS-277			
HWY	403		
SUBWD	DUP	CHKD	PROJECT NO. 08-1132-015-1
DRAWN	LMK	CHKD	DATE JULY 21/08
		APPR	SITE
			DWG. 2



~~Q TEMPORARY DETOUR~~



- TOPSOIL, clayey
Dark brown



SECTION ALONG B-B



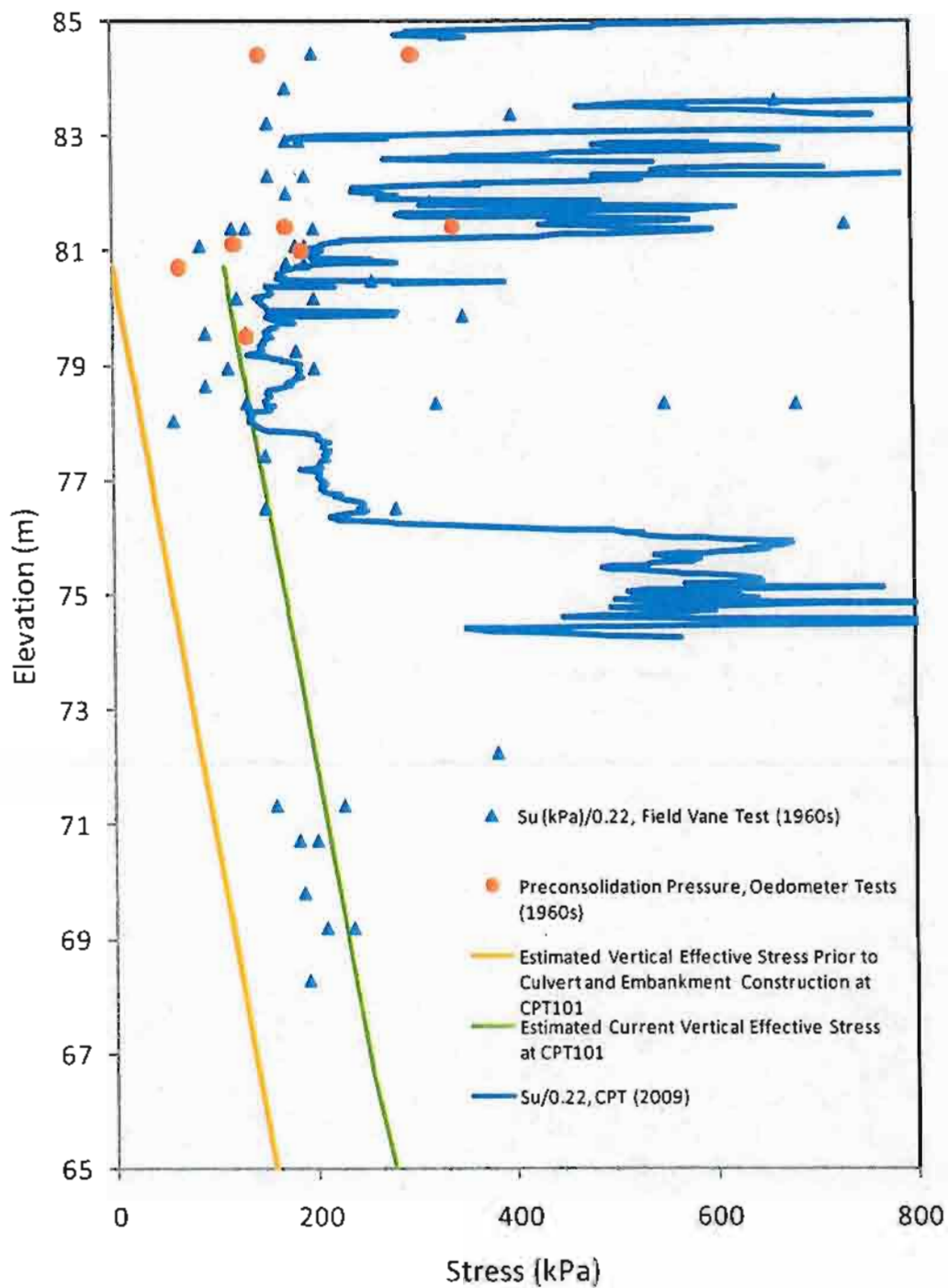
Compact to very delicate
Brown to grey



**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGH FILLS FOR TEMPORARY DETOUR AT HWY 403 ABERDEEN AVE**

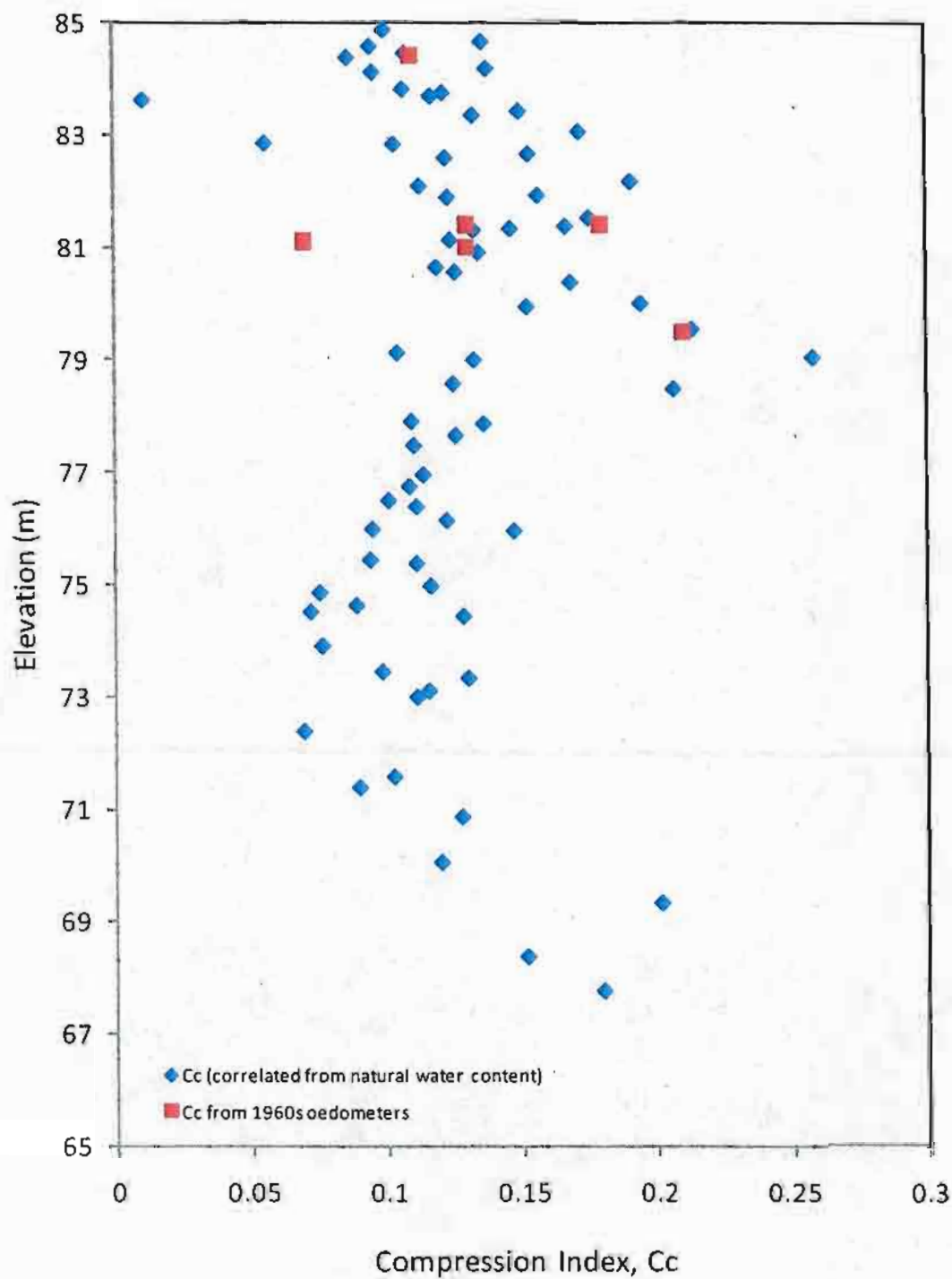
APPENDIX D

Interpreted Field Data



PROJECT HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE HIGHWAY 403 BRIDGE REHABILITATION			
TITLE PRECONSOLIDATION DATA, BEFORE CULVERTS CONSTRUCTED (CLAYEY SILT)			
PROJECT No. 08-1132-013-1		FILE No. 0811320131-R01001	
CADD 1.1MK	DATE July 5/09	SCALE AS SHOWN	REV. 0
CHECK <i>[Signature]</i>	DATE <i>July 15/09</i>	FIGURE D-1	

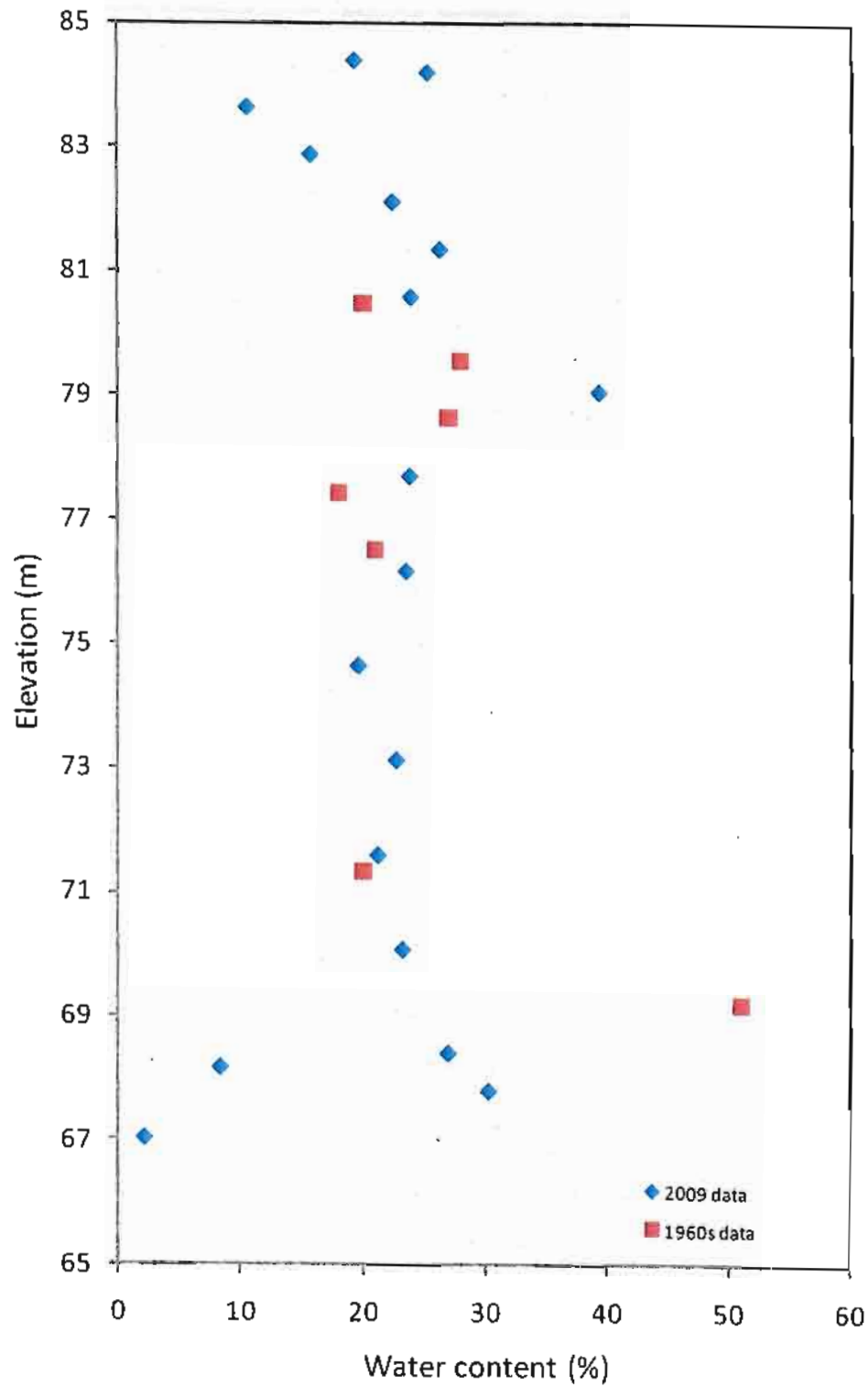





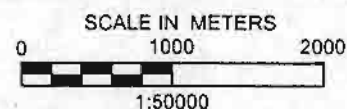
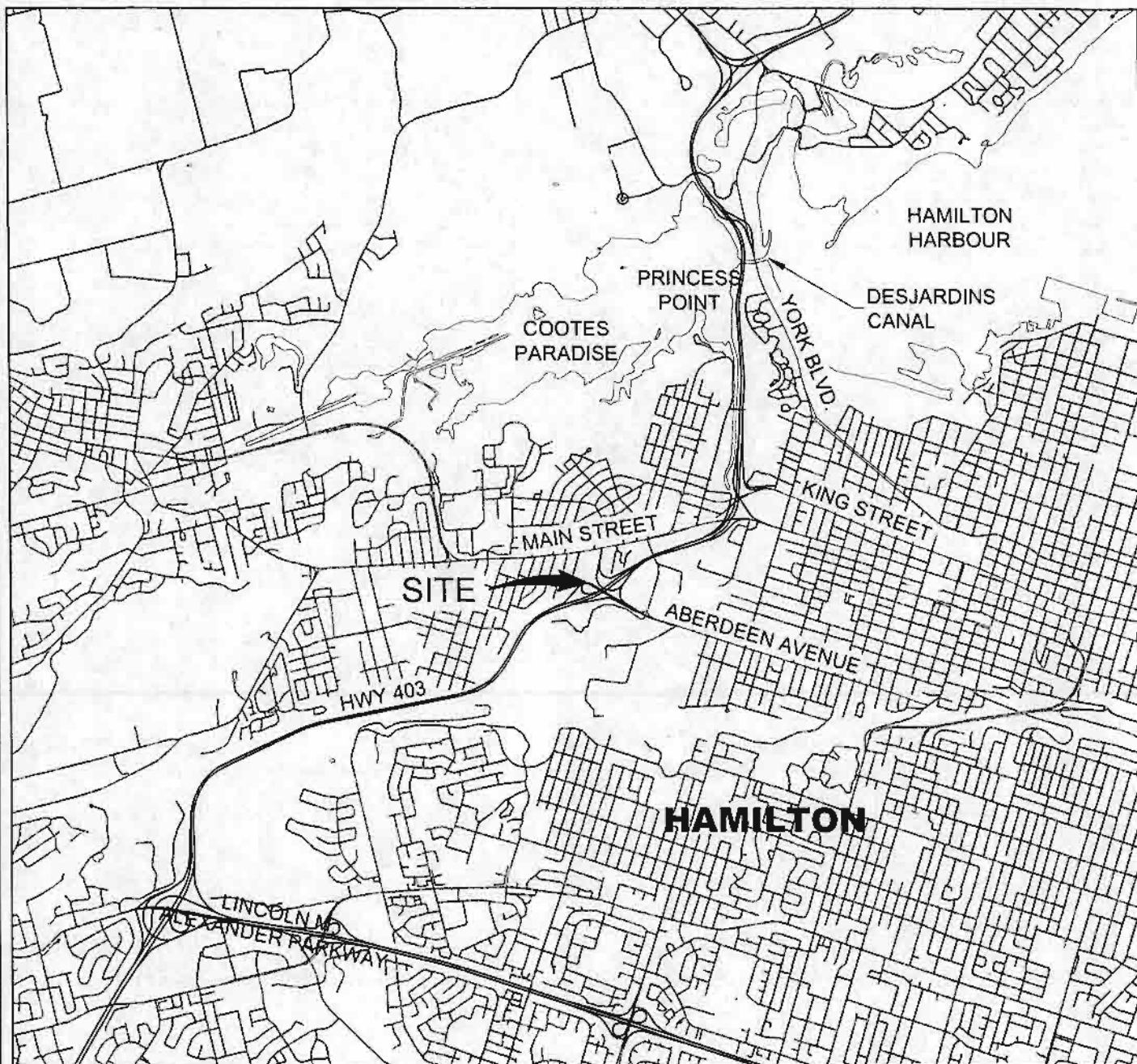
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PROJECT			
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE HIGHWAY 403 BRIDGE REHABILITATION			
TITLE			
C_c DETERMINATION (CLAYEY SILT)			
PROJECTING		FILE NO.	
08-1132-013-1		0811320131-R010D1	
SCALE		AS SHOWN	
REV.		0	
GOLDER ASSOCIATES LONDON, ONTARIO		CHECKED: <i>[Signature]</i> DATE: JULY 2009	

FIGURE D-2



PROJECT			
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE HIGHWAY 403 BRIDGE REHABILITATION			
TITLE			
ELEVATION VS WATER CONTENT (CLAYEY SILT)			
PROJECT No. 08-1132-013-1		FILE No. 0811320131-R01601	
CADD	LMK	July 9/09	SCALE AS SHOWN
CHECK	[Signature]		REV. 0
 Golder Associates LONDON, ONTARIO		FIGURE D-3	



PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE

KEY PLAN

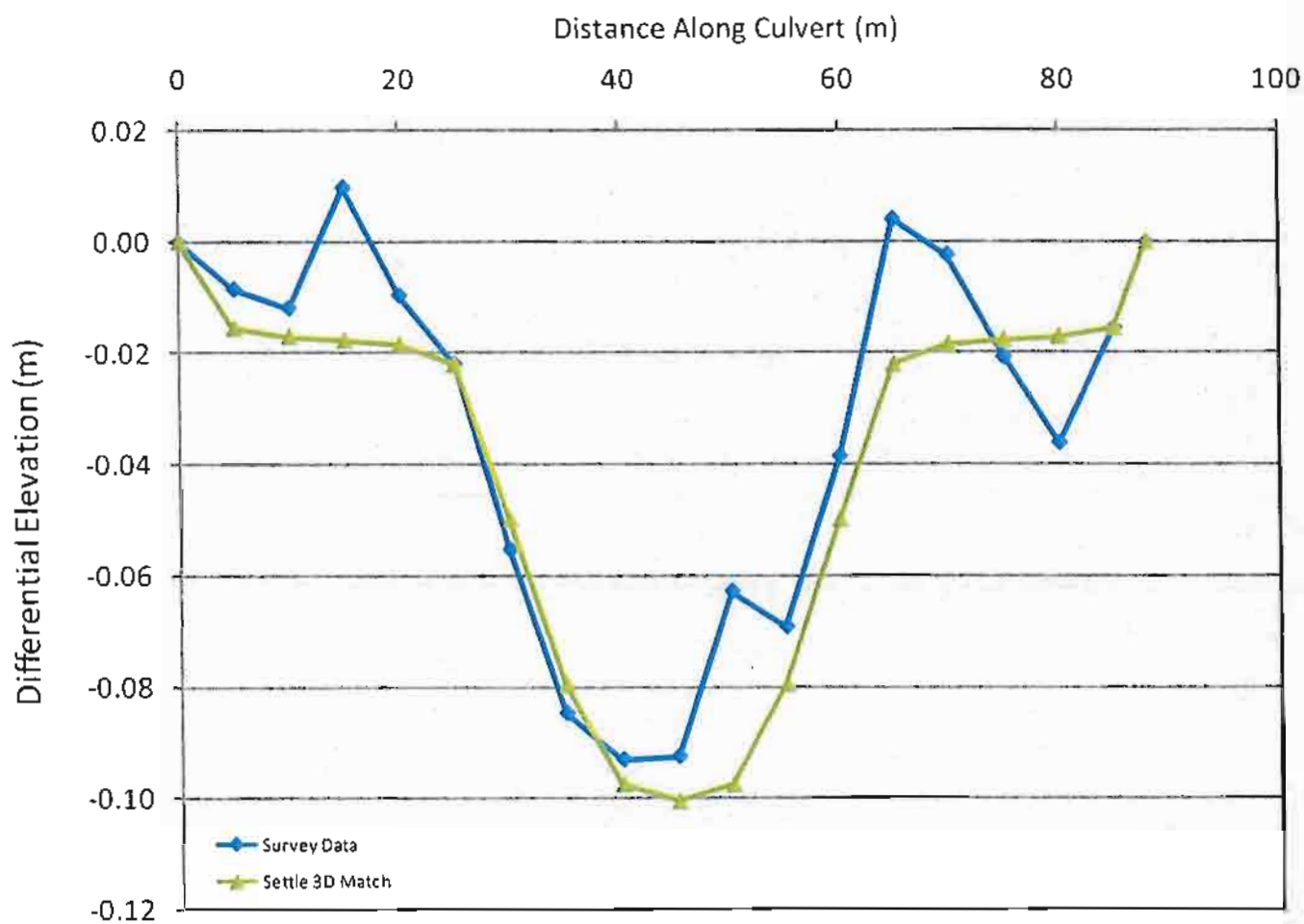


PROJECT No. 08-1132-013-1			FILE No. 0811320131-F01001		
CADD LMK July 5/09			SCALE AS SHOWN	REV. 0	
CHECK <i>[Signature]</i> Jul 15/09			FIGURE 1		




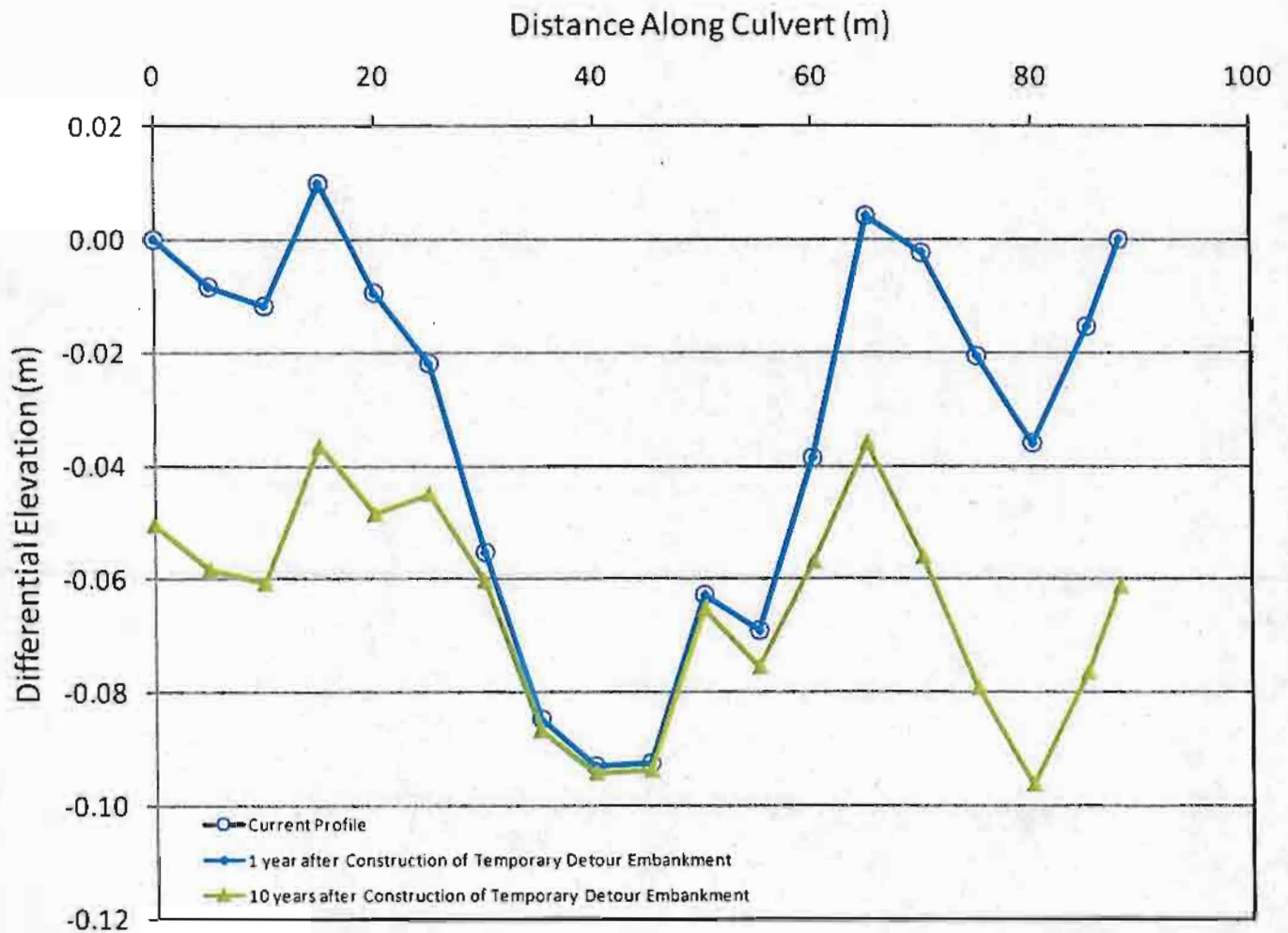
APPENDIX E

Results of Settlement Analyses



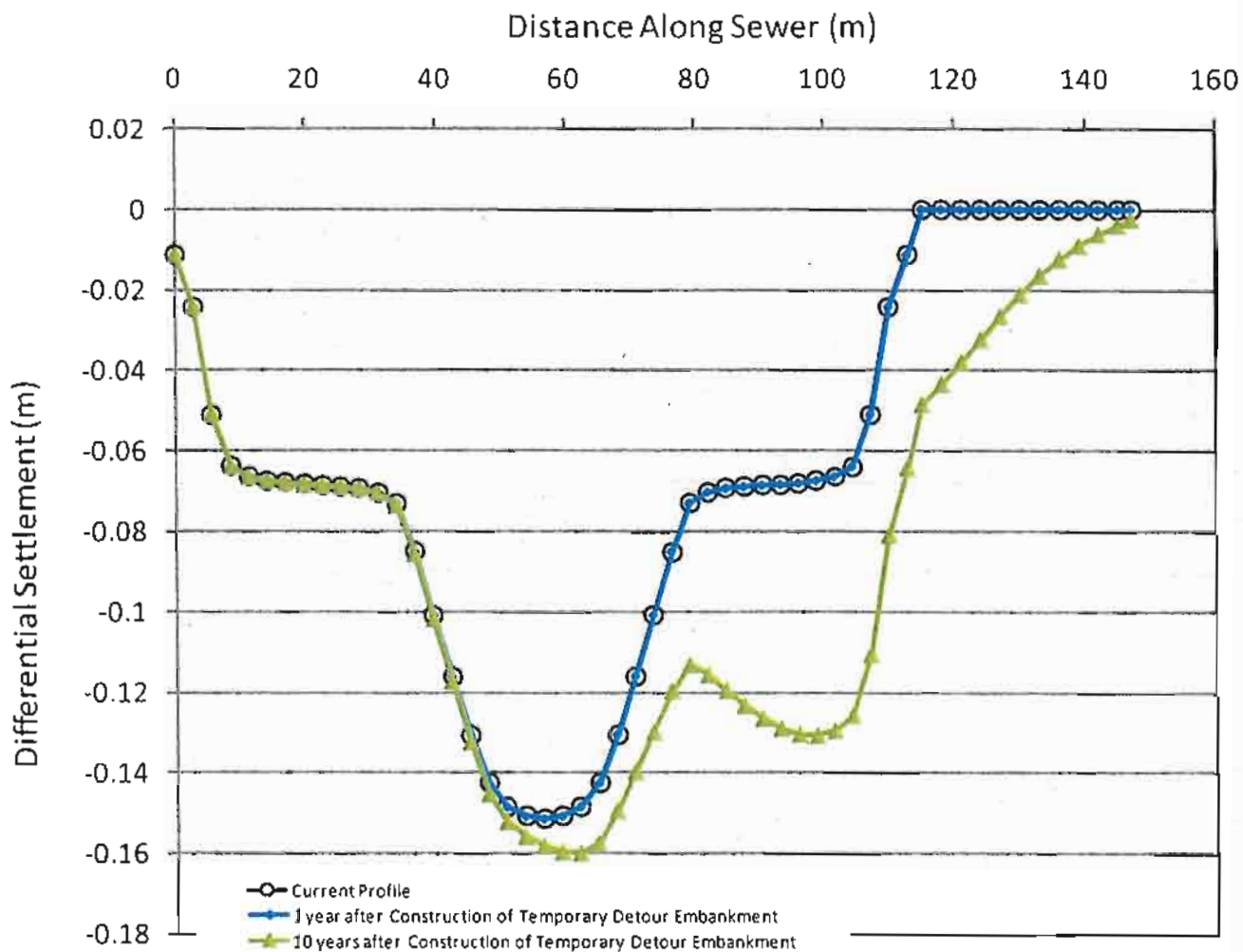
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PROJECT			
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE HIGHWAY 403 BRIDGE REHABILITATION			
TITLE			
CURRENT DIFFERENTIAL SETTLEMENT ALONG CULVERT			
PROJECT No. 08-1132-013-1		FILE No. 0811320131-R010E1	
CADD	CHK	July 07/09	SCALE AS SHOWN
CHECK	alp	July 07/09	REV. 0
 Golder Associates LONDON, ONTARIO			FIGURE E-1



Drawing file: 0811320131-R010E1.dwg Jul 09, 2009 - 2:22pm

PROJECT HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE HIGHWAY 403 BRIDGE REHABILITATION															
TITLE PREDICTED FUTURE SETTLEMENT ALONG CULVERT															
Golder Associates LONDON, ONTARIO		<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td colspan="2" style="font-size: 8px;">PROJECT No. 08-1132-013-1</td> <td colspan="2" style="font-size: 8px;">FILE No. 0811320131-R010E1</td> </tr> <tr> <td style="font-size: 8px;">CADD</td> <td style="font-size: 8px;">LMK</td> <td style="font-size: 8px;">July 9/09</td> <td style="font-size: 8px;">SCALE AS SHOWN</td> </tr> <tr> <td style="font-size: 8px;">CHECK</td> <td style="font-size: 8px;"><i>[Signature]</i></td> <td style="font-size: 8px;"><i>[Signature]</i> Jul 13/09</td> <td style="font-size: 8px;">REV. 0</td> </tr> </table>		PROJECT No. 08-1132-013-1		FILE No. 0811320131-R010E1		CADD	LMK	July 9/09	SCALE AS SHOWN	CHECK	<i>[Signature]</i>	<i>[Signature]</i> Jul 13/09	REV. 0
PROJECT No. 08-1132-013-1		FILE No. 0811320131-R010E1													
CADD	LMK	July 9/09	SCALE AS SHOWN												
CHECK	<i>[Signature]</i>	<i>[Signature]</i> Jul 13/09	REV. 0												
		FIGURE E-2													



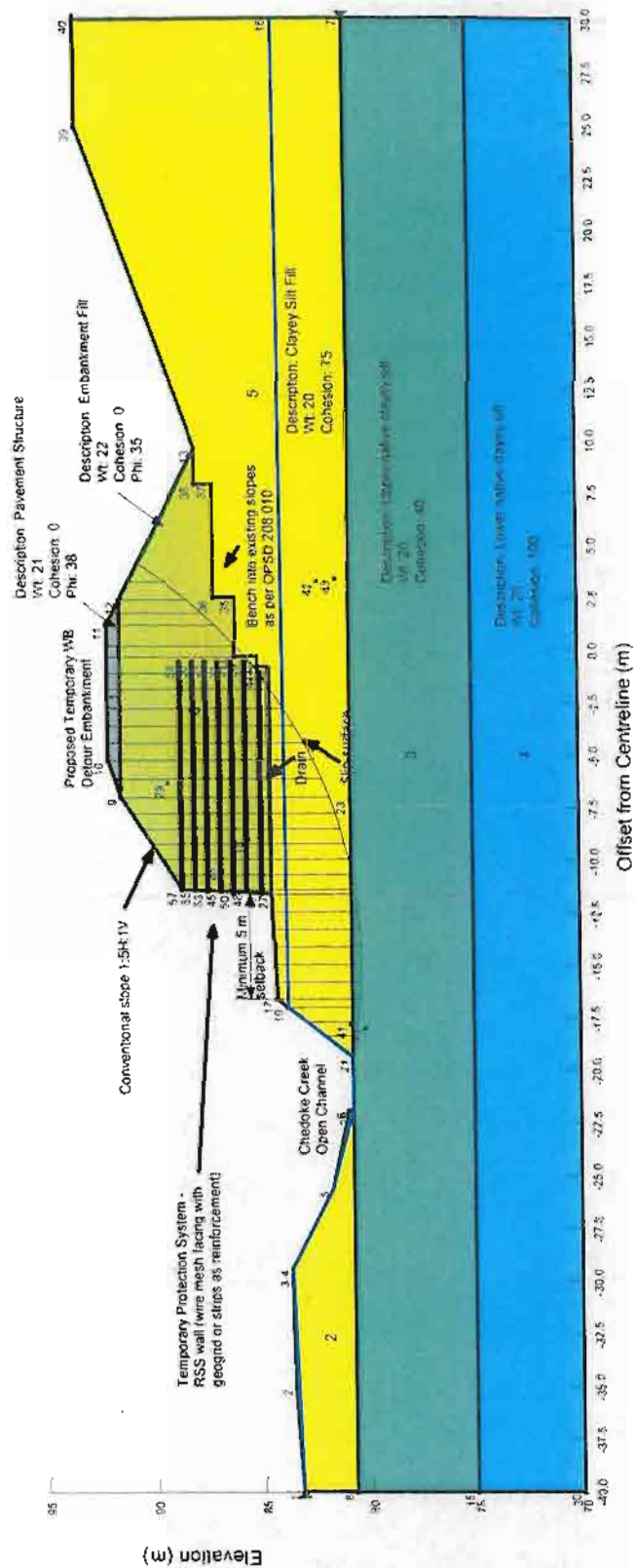
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PROJECT HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT HIGHWAY 403 - ABERDEEN AVENUE INTERCHANGE HIGHWAY 403 BRIDGE REHABILITATION			
TITLE PREDICTED FUTURE SETTLEMENT ALONG SEWER			
Golder Associates LONDON, ONTARIO		PROJECT No. 08-1132-013-1 CADD LMK July 2009 CHECK <i>[Signature]</i> Jul 8/09	FILE No. 0811320131-R010E1 SCALE AS SHOWN REV. 0
			FIGURE E-3



APPENDIX F

Results of Slope Stability Analyses



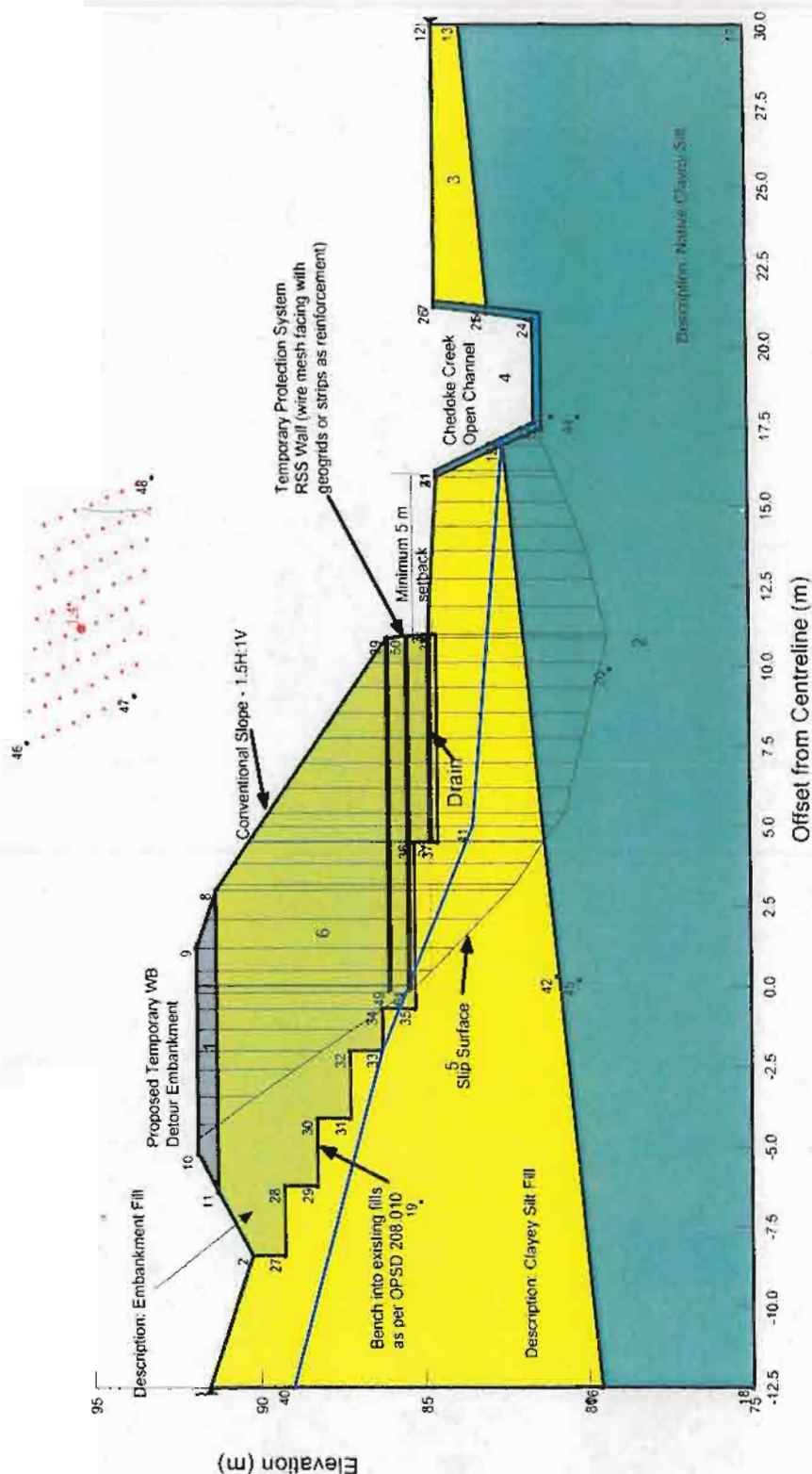
PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE
RESULTS OF SLOPE STABILITY ANALYSIS
STATION 0+070 - RSS wall with 1.5H to 1V Sideslope



PROJECT No.	08-1132-013-1	FILE No.	0811320131-R010P1
DRAWN	DUP	Aug 21/09	SCALE AS SHOWN
CHECK			REV. 0

Figure F-1



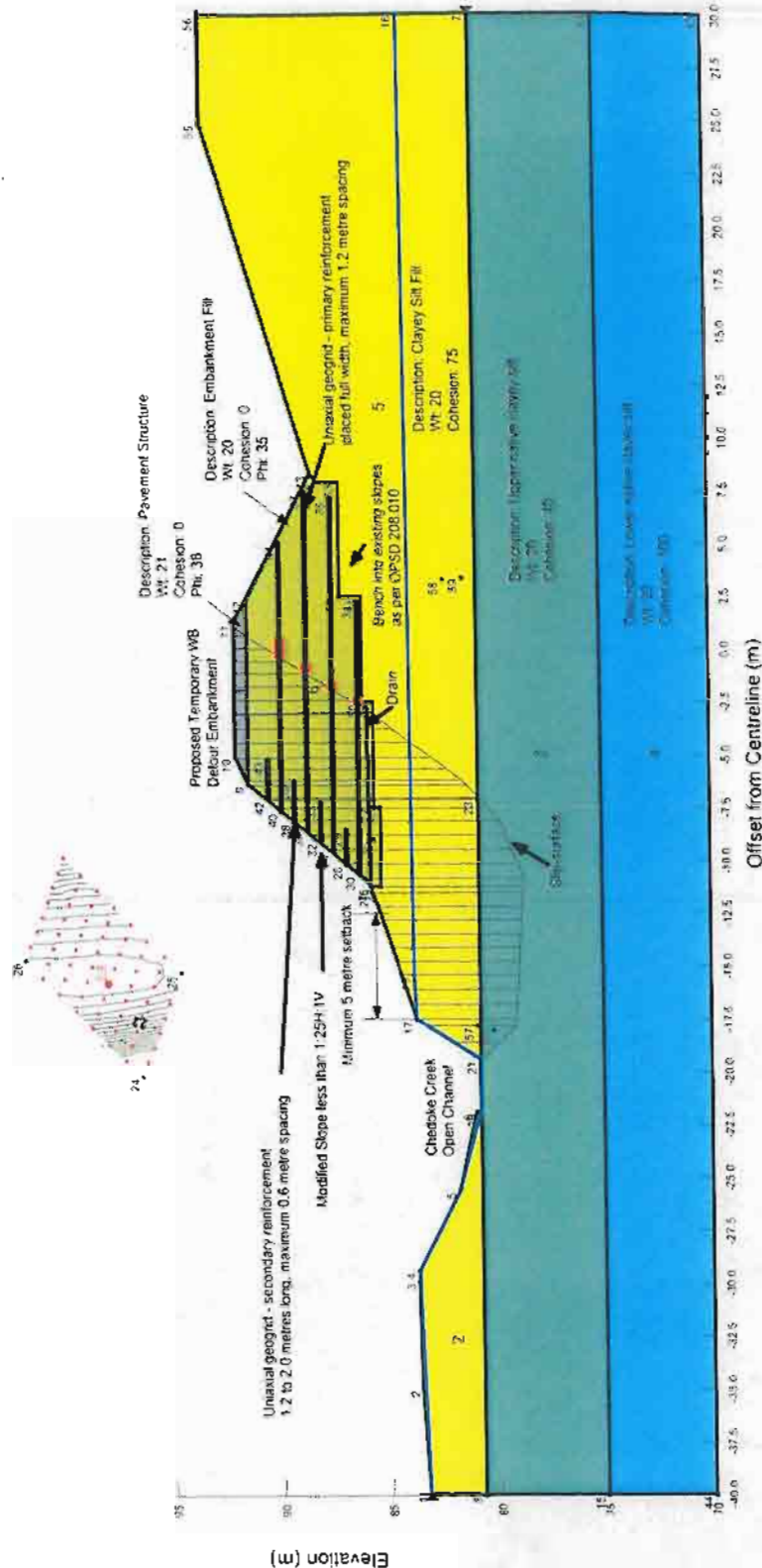
PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE
RESULTS OF SLOPE STABILITY ANALYSIS
STATION 0+160 - RSS wall with 1.5H to 1V Sideslope



PROJECT No. 06-1132-013-1		FILE No. 0811320131-R010F2	
DRAWN	DUP	Aug 21/09	SCALE AS SHOWN
CHECK			REV. 0

Figure F-2

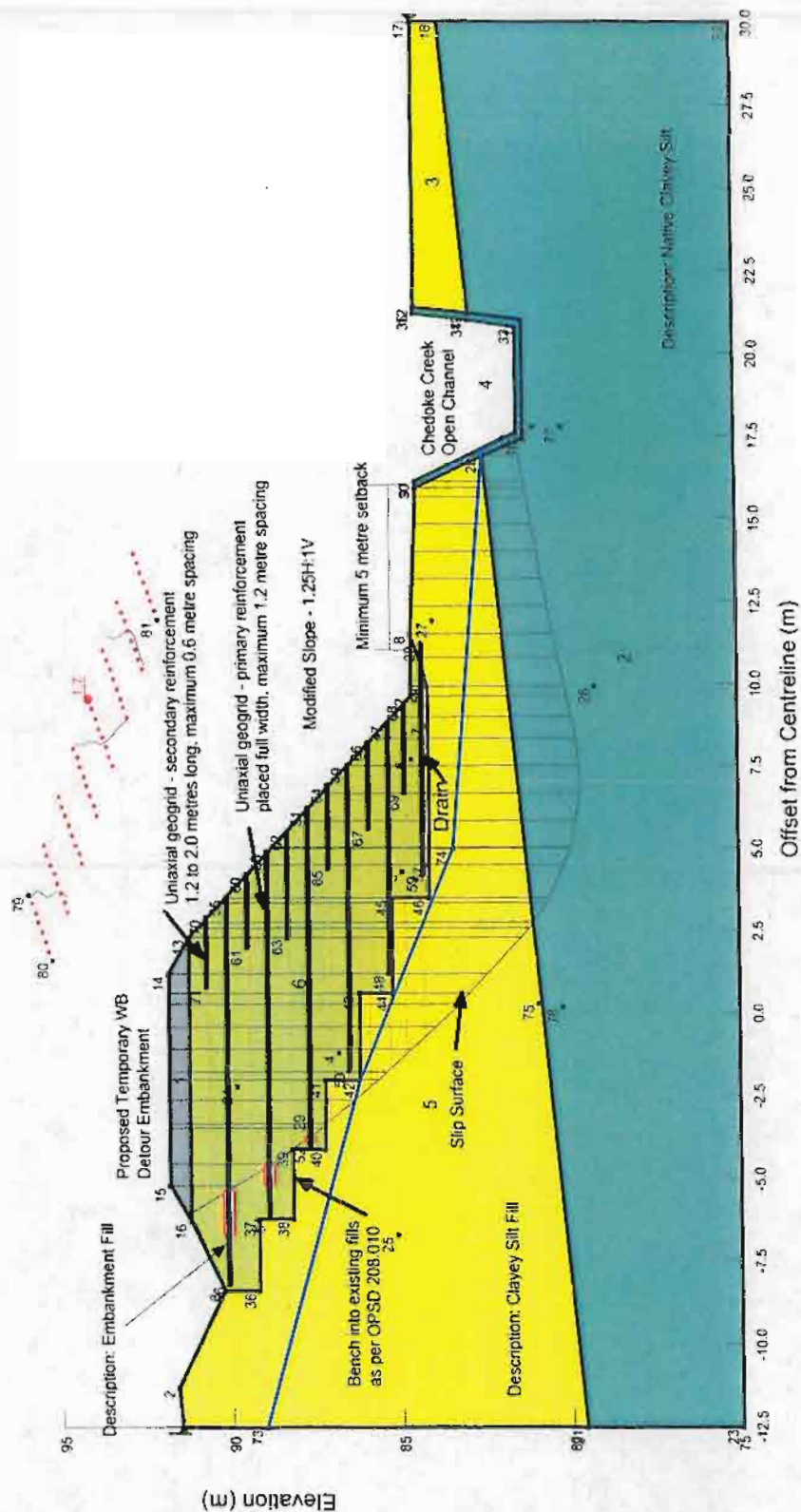


PROJECT
 HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
 HIGHWAY 403 ABERDEEN AVENUE INTERCHANGE
 HIGHWAY 403 BRIDGE REHABILITATION

TITLE
 RESULTS OF SLOPE STABILITY ANALYSIS
 STATION 0+060 - Proposed 0.75H: 1V Sideslope



PROJECT No. 08-1132-013-1		FILE No. 08-1132-013-1-RC\0313	
DRAWN	DUP	Aug 21/09	SCALE AS SHOWN/REV. 0
CHECK	\$		Figure F-3



PROJECT
HIGH FILLS FOR DETOUR - RAPID BRIDGE REPLACEMENT
HIGHWAY 403 ABERDEEN AVENUE INTERCHANGE
HIGHWAY 403 BRIDGE REHABILITATION

TITLE
RESULTS OF SLOPE STABILITY ANALYSIS
STATION 0+160 - Proposed 1H: 1V Sideslope



PROJECT No. 08-1132-013-1		FILE No. 0811320131-R010F4	
DRAWN DUP		Aug. 21/06	
CHECK		SCALE AS SHOWN/REV. 0	
		Figure F-4	



