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**FOUNDATION INVESTIGATION AND DESIGN REPORT
LAMBTON COUNTY ROAD 79 UNDERPASS (SITE 14-355)
STRUCTURAL REHABILITATION
HIGHWAY 402 AND LAMBTON COUNTY ROAD 79 IMPROVEMENTS
GWP 3158-06-00
WATFORD, ONTARIO**

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PART A – FOUNDATION INVESTIGATION REPORT

**LAMBTON COUNTY ROAD 79 UNDERPASS (SITE 14-355)
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HIGHWAY 402 AND LAMBTON COUNTY ROAD 79 IMPROVEMENTS
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WATFORD, ONTARIO**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Delcan Corporation (Delcan) on behalf of Waste Management (WM) to carry out foundation investigations as part of the design package for improvements for the Highway 402/Lambton County Road 79 (Nauvoo Road) interchange and Lambton County Road 79 south of the interchange to the new entrance of the Warwick Landfill in the Township of Warwick, Ontario.

The proposed works are being undertaken in conjunction with the Warwick Landfill Expansion Project. The design package is to be completed in accordance with Ministry of Transportation, Ontario (MTO) standards. The scope of work for this project consists of the geotechnical field investigation and design of the following components of the project:

- rehabilitation of the Lambton County Road 79 Underpass Structure (Site 14-355);
- profile grade adjustments (filling) on Lambton County Road 79;
- profile grade adjustments on portions of the existing E-N/S, S-W, N-E and W-N/S ramps;
- replacement of the existing S-E and N-W ramps with new ramps;
- possible pavement upgrades on the existing E-N/S ramp;
- paved shoulders along Lambton County Road 79 from Highway 402 to the landfill entrance;
- roadway improvements along Lambton County Road 79 at the new landfill entrance;
- culvert extensions on Lambton County Road 79; and
- a culvert extension on Highway 402.

This report addresses the foundations engineering aspects of the proposed abutment modifications and the rehabilitation of the embankment side slopes at the Highway 402/Lambton County Road 79 Underpass structure. The foundation investigation and reporting was conducted in accordance with MTO standards for detail design.

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 WM Project Terms of Reference, in our proposal P71-3118 dated June 26, 2007, and our letter pertaining to additional foundation engineering services (07-1130-128-4) dated September 14, 2007. The work was conducted in accordance with our letter dated September 14, 2007.

Delcan provided Golder with preliminary drawings for this project in digital format for the two span structure. The preliminary general arrangement drawing indicates that new semi-integral abutment systems and concrete barrier walls on the deck and wingwalls are to be constructed with reconstruction of the approach slabs. It also indicates that the deck and soffit are to be rehabilitated.

The Department of Transportation and Communications Ontario Drawings for the Highway 79 Underpass W.P. 42-66-11 dated March 1972 provided by Delcan shows that the pier is founded on a strip footing 3.2 metres wide. The underside of footing is at about elevation 229.8 metres. The abutments are founded on 325 millimetre nominal diameter, concrete filled, steel tube piles. The drawings indicate that these are friction piles which were not to be driven below elevation 228.6 metres. The approximate bottom of pile cap elevation is at 234.2 metres. The approximate top of deck elevation is 238.75 metres at the centreline of the pier. The underpass structure is 18.29 metres wide with a total span length of 81.38 metres.

2.0 SITE DESCRIPTION

This project consists of the upgrading of the Highway 402/Lambton County Road 79 (Nauvoo Road) interchange with rehabilitation of the underpass structure, profile adjustments on Lambton County Road 79 and the affected ramps, construction of a new S-E ramp and N-W ramp and possible upgrading of the pavement on the E-N/S ramp. In addition, short span and structural culverts in the areas of the roadway improvements are to be extended and Lambton County Road 79 is to be upgraded in the vicinity of the entrance to the new Waste Management landfill. The location of the interchange is shown on the Key Plan, Figure 1.

The surrounding area is predominately agricultural lands with woodlots immediately north of the interchange. A former construction yard is located immediately north of the interchange and a former gas storage yard is located south of the interchange, both to the west of Lambton County Road 79. The adjacent topography is generally flat with a ground surface elevation ranging from 232 metres to 235 metres based on the topographical mapping for Watford, Map No. 40-I/13e.

Lambton County Road 79 is a two lane road with 3.35 metre wide lanes and variable width lanes leading to the Highway 402 ramps. The existing Lambton County Road 79 underpass structure is a two span simply supported steel box girder bridge constructed in 1978 and designated as Site Number 14-355. The bridge carries traffic on Lambton County Road 79 over Highway 402. Both spans are 41.76 metres long and the existing deck has an overall width of 18.29 metres. The subject section of Highway 402 is a divided rural freeway with two 3.65 metre wide lanes and one outer variable width speed change lane and paved shoulders in each direction. The interchange has ramps in the northeast and southwest quadrants only. Photographs of the structure are shown in Appendix C.

During a site reconnaissance carried out on September 13, 2007 by the Project Manager and Senior Field Supervisor, it was noted that signs of instability and post construction movements were present in the form of tension cracks in portions of the approach fills. The distress was documented in our September 14, 2007 letter to Delcan.

The side slopes parallel to Lambton County Road 79 and the abutment face slopes parallel to Highway 402 were measured at both approaches using an Abney hand level. At the north approach, the side slopes were found to be 25 and 23 degrees in the northwest and northeast quadrants, respectively. However, the slopes facing Highway 402 were found to be 30 and 33 to 34 degrees in the northwest and northeast quadrants, respectively. At the southern approach, the both side slopes were approximately 25 degrees. The slopes facing Highway 402 were found to be 30 and 29 degrees in the southwest and southeast quadrants, respectively.

Cracks or distresses behind the slope crest were not observed. Cracking of the slope face was noted in the northeast and southeast quadrants of the approach fills as shown in Photographs 3 to 7 in Appendix C. Differential settlements between the structure and the curbs and gutters on the

approach slabs and approach fills was noted at all four corners of the bridge. These settlements were most apparent at the southeast corner of the bridge. The slope face is moderately well vegetated with grass. However, several bare spots were evident. Beneath the structure, the slopes are covered with rip rap.

2.1 Site Geology

The project is located in the physiographic region of southern Ontario known as the Horseshoe Moraines as identified in "The Physiography of Southern Ontario", by Chapman and Putnam (1984). The southwestern limb of the region consists of two, and in some places three, morainic ridges composed of pale brown, hard, calcareous, fine-textured till with a moderate degree of stoniness.

Based on the Ontario Department of Mines and Northern Affairs Preliminary Map P.1972 entitled "Quaternary Geology of the Strathroy Area", the project area is reportedly located in predominantly clayey silt to silty clay till.

The surface of the subcropping rock is reported to be about 27 metres below ground surface based on Ontario Geological Survey Preliminary Map P.2453. According to Geological Survey of Canada Map 1263A, "Geology Toronto – Windsor Area", the rock belongs to the Kettle Point formation and consists of black bituminous shale with greenish-grey silty shale interbeds.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on November 21 and November 28, 2007 at which time four boreholes were drilled at the locations indicated on Drawing 1.

The as-drilled borehole locations, ground surface elevations and depths of the boreholes drilled by Golder are as follows:

<u>BOREHOLE</u>	<u>LOCATION (m)</u>		<u>GROUND SURFACE ELEVATION</u>	<u>BOREHOLE DEPTH</u>
	<u>Northing</u>	<u>Easting</u>	(m)	(m)
201	4,761,446.1	355,406.5	234.44	3.35
202	4,761,609.5	355,406.1	234.51	3.35
203	4,761,461.1	355,397.4	237.96	8.84
204	4,761,624.5	355,397.9	237.89	8.84

The soil stratigraphy encountered in the boreholes is shown on the attached Record of Borehole sheets. The boreholes at the top of the embankment were drilled using a truck mounted CME 45

power auger supplied and operated by a specialist drilling contractor. The boreholes on the embankment side slopes were advanced by Golder staff using manual drilling techniques.

Standard penetration testing and sampling was carried out at suitable intervals of depth in each of the boreholes using 35 millimetre inside diameter split spoon sampling equipment. In the manually drilled boreholes, hand augering equipment was used to advance the holes, the testing was carried out using a 31.8 kilogram hammer and the driving resistances recorded were adjusted to represent approximate N values. Groundwater conditions in the boreholes were observed throughout the drilling operations and the boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 128/03. Standpipes were installed in boreholes 203 and 204 to monitor groundwater conditions.

The field work was supervised on a full-time basis by an experienced member of our engineering staff who directed the drilling, sampling and in situ testing operations, logged the boreholes and determined the ground surface elevations and borehole locations. The borehole elevations are referenced to benchmarks provided by Callon Dietz Inc. It is understood that the benchmark elevations are referenced to geodetic datum.

The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and routine classification testing. Index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

In addition, the results of six previous boreholes and cone penetration tests by others have been included in this report. The previous boreholes were included in the following report:

- Department of Highways, Ontario, Geocres 40I13-31 entitled "Foundation Investigation Report for Proposed Crossing at Hwy. 79 and C.A.H. 402, Line 'C', Twp. Of Warwick, Co. of Lambton, District #1 (Chatham). W.O. 71-11042 – W.P. 42-66-11" dated April, 1976.

The locations of the previous boreholes are shown on Drawing 1 and the Records of Boreholes, together with the laboratory results, are provided in Appendix B.

The locations of the boreholes drilled for the previous investigation are as follows:

<u>BOREHOLE</u>	<u>LOCATION (m)</u>		<u>GROUND SURFACE ELEVATION</u>	<u>BOREHOLE DEPTH</u>
	<u>Northing</u>	<u>Easting</u>	(m)	(m)
1	4,761,589.7	355,389.6	232.29	20.12
2	4,761,590.7	355,379.7	232.23	6.55
3	4,761,547.1	355,398.9	232.41	6.55
4	4,761,548.6	355,378.8	232.17	22.92
5	4,761,507.1	355,398.9	232.32	21.52
6	4,761,506.2	355,379.4	232.17	6.55

The locations of the previous boreholes are approximate and have been inferred based on the borehole locations provided in Geocres Report No. 40I13-31.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the 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 Records of Boreholes from the previous investigation are included in Appendix B along with the pertinent laboratory data. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site encountered thin layers of surficial topsoil or the pavement structure and silty clay fill associated with the existing embankment underlain by extensive deposits of silty clay and clayey silt. Silt layers were encountered within the silty clay and clayey silt deposits.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, 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 in the following sections.

4.1.1 Pavement Structure

Borehole 204 was drilled on the shoulder of Lambton County Road 79 and encountered 240 millimetres of asphalt at the surface. The asphalt was underlain by approximately 220 millimetres of granular roadbase materials.

4.1.2 Topsoil and Fill

Surficial topsoil layers, between 90 and 150 millimetres thick, were found at the ground surface in boreholes 201 to 203. Layers of buried silty topsoil were encountered in boreholes 201, 203 and 204 from elevations 232.2 to 232.4 metres.

Silty clay fill was encountered in all of the boreholes drilled at the site beneath the pavement structure or surficial topsoil. The thickness of the silty clay fill varied from 2.2 to 5.2 metres. The silty clay fill was firm to very stiff with N values of 6 to 23 blows per 0.3 metres and had water contents ranging from 12 to 25 per cent with an average water content of about 19 per cent. The average plastic and liquid limits of the silty clay fill, based on the results of four Atterberg limits determinations, were 20 per cent and 38 per cent, respectively, with an average plasticity

index of 18 per cent. The results of the Atterberg limits testing are presented on the Plasticity Chart, Figure A-3, and indicate a clayey soil of intermediate plasticity.

The results of the grain size testing of four samples of silty clay fill recovered from the standard penetration testing are presented on Figure A-1.

4.1.3 Silty Clay

Silty clay was encountered near elevation 232.0 metres beneath the clayey fill in borehole 202 and below the buried topsoil in boreholes 201, 203 and 204 drilled during the current investigation, and at ground surface in the previous boreholes. Boreholes 201 to 204 were terminated in the silty clay after exploring it for some 0.6 to 1.0 metres.

The silty clay was stiff to hard with N values of 12 to 35 blows per 0.3 metres and water contents of 18 to 29 per cent with an average water content of about 22 per cent. The plastic and liquid limits, based on a single Atterberg limits determination, were 20 per cent and 37 per cent, respectively, with a plasticity index of 17 per cent. The results of the Atterberg limits testing are presented on the Plasticity Chart, Figure A-3, and indicate an inorganic clayey soil of intermediate plasticity. The results of grain size testing of a sample of silty clay recovered from the standard penetration testing are presented on Figure A-2 of Appendix A.

Soil described as clayey silt with traces of sand was found in the boreholes from the previous investigation. The grain size distribution envelope and plasticity chart for the clayey silt materials are shown in Figures 1 and 2, respectively, of Appendix B. Based on comparison of the Atterberg limits and gradation of the samples from the previous boreholes with those from the current investigation, it is interpreted that this material is predominantly clayey silt with an approximately 5 metre thick surficial zone of slightly higher plasticity to about elevation 227.5 metres. In all boreholes except borehole 1, the samples above elevation 227.5 metres generally had liquid limits of 35 per cent or greater and clay and silt contents of approximately 60 per cent and 40 per cent, respectively. The following discussion will consider soils in the previous boreholes with intermediate plasticity above elevation 227.5 metres to be silty clay.

Silty clay is inferred to be present in boreholes 2 to 6 from the ground surface to approximately elevation 227.5 metres. Partings of silt and/or fine sand were found in the silty clay. A layer of dense silt was encountered between elevations 227.9 and 229.5 metres.

The silty clay in the previous boreholes is very stiff to hard with N values ranging from 17 to 62 blows per 0.3 metres. Water contents of 18 to 25 per cent were measured in the silty clay. The average plastic and liquid limits of eight samples were 21 and 36 per cent, respectively, with an average plasticity index of 15 per cent.

4.1.4 Silt

Layers of silt 0.7 to 1.0 metres thick was encountered between layers of silty clay near elevation 230.0 metres in boreholes 203 and 204 and between elevations 227.9 and 229.5 metres in boreholes 3 to 5.

The silt was compact to dense but typically dense with N values of 21 to 32 blows per 0.3 metres and a water content of 25 per cent. The results of grain size testing of a single sample of the silt recovered from the standard penetration testing conducted in borehole 204 are presented on Figure A-4 of Appendix A.

4.1.5 Clayey Silt

As discussed in Section 4.1.3, clayey silt was found in borehole 1 and from elevation 227.5 metres in boreholes 2 to 6. Clayey silt was not found in any of the shallower current boreholes.

The clayey silt was very stiff to hard with N values of 15 to 75 blows per 0.3 metres and water contents ranging from 14 to 22 percent with an average of 17 per cent. Two field vane tests conducted in softer zones indicated undrained shear strengths of 108 and 153 kilopascals with sensitivities of 2.2 and 1.8, respectively. Geocres Report 40I13-31 cites the results of two unspecified laboratory tests that were conducted to determine shear strength. The test results are not identified on the logs, but the report notes that undrained strengths of 86.2 and 205.9 kilopascals were obtained.

The average plastic and liquid limits of the clayey silt, based on 12 samples recovered from the boreholes, were 18 per cent and 28 per cent, respectively, with an average plasticity index of 10 per cent. The grain size distribution envelope and plasticity chart for the clayey silt materials are shown in Figures 1 and 2 of Appendix B.

4.2 Bedrock

Shale to shaley limestone bedrock was found in the deeper boreholes of the previous investigation beneath the clayey silt from elevation 212.5 metres in borehole 1, from elevation 210.7 metres in borehole 4 and from elevation 210.8 metres in borehole 5. Weathered samples were retrieved from the split spoon samples in boreholes 1 and 5. In borehole 5, a 1.4 metre length of BX core was obtained with a reported core recovery of 75 per cent.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling. Boreholes 201 and 202 were both dry during drilling.

Standpipes were installed in boreholes 203 and 204 prior to backfilling. In borehole 203, groundwater was encountered during drilling within the silt layer at about elevation 230.0 metres. The most recent groundwater level readings were obtained on January 30, 2008. On that date, the groundwater level measured in the standpipe was at elevation 232.0 metres or 5.9 metres below the embankment surface. In borehole 204, groundwater was encountered during drilling within the silt layer at about elevation 230.0 metres. The most recent groundwater level readings were obtained on January 30, 2008. On that date, a groundwater level at elevation 231.0 metres or 6.8 metres below the embankment surface was measured in the standpipe.

During the previous investigation, the boreholes encountered groundwater from about elevation 230.7 metres to about 231.6 metres.

Details of the groundwater conditions encountered and subsequently measured in the installations are provided on the Record of Borehole sheets and are summarized below.

BOREHOLE	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL		INSTALLATION	MEASURED GROUNDWATER LEVEL					
		Depth (m)	Elevation (m)		November 28, 2007		December 19, 2007		January 30, 2008	
					Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
201	234.44	Dry	Dry	-	-	-	-	-	-	-
202	234.51	Dry	Dry	-	-	-	-	-	-	-
203	237.96	8.0	230.0	Standpipe	8.56	229.40	5.24	232.72	5.94	232.02
204	237.89	7.9	230.0	Standpipe	8.79	229.10	6.17	231.79	6.80	231.09
1	232.29	0.9	231.4	-	-	-	-	-	-	-
2	232.23	0.9	231.3	-	-	-	-	-	-	-
3	232.41	0.8	231.6	-	-	-	-	-	-	-
4	232.17	1.1	231.1	-	-	-	-	-	-	-
5	232.32	0.9	231.4	-	-	-	-	-	-	-
6	232.17	1.3	230.9	-	-	-	-	-	-	-

The inferred groundwater level based on the measured and encountered groundwater levels is about elevation 231.5 metres, approximately 1.5 metres above the interface between the brown and grey clayey silt and silty clay.

The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., which is an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Daniel R. P. Babcock under the direction of Mr. David J. Mitchell. The routine laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates.

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

GOLDER ASSOCIATES LTD.

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DB/DUP/PRB/FJH/cr
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PART B – FOUNDATION DESIGN REPORT

**LAMBTON COUNTY ROAD 79 UNDERPASS (SITE 14-355)
STRUCTURAL REHABILITATION
HIGHWAY 402 AND LAMBTON COUNTY ROAD 79 IMPROVEMENTS
GWP 3158-06-00
WATFORD, ONTARIO**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the proposed rehabilitation of the approach fills and retrofitting of the Highway 402/Lambton County Road 79 Underpass structure based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing Lambton County Road 79 underpass structure is a two span, simply supported steel box girder bridge constructed in 1978 and designated as site number 14-355. The bridge carries traffic on Lambton County Road 79 over Highway 402. The span lengths are 41.76 metres and the existing deck has an overall width of 18.29 metres. The abutments are founded on piles and the pier is founded on a spread footing. The piles were to be driven to about elevation 228.6 metres and the founding elevation of the pier footing is approximately 229.8 metres. The elevation of the current bridge deck is about 238.75 metres. The Highway 402 pavement beneath the underpass structure is at elevation 232 metres.

The retrofitting of the bridge structure will include the reconstruction of approach slabs, construction of new barrier walls on the deck and wingwalls, repair of bridge deck, conversion of the existing abutments to semi-integral abutments and the removal of existing deck and deck joint assemblies at the pier and abutments. Vertical profile grade changes and excavations are anticipated during the retrofitting works.

The rehabilitation of the approach fills is required to stabilize the slope and to remediate distresses currently present in the approach fills adjacent to the abutments and to reduce the risk of on going slope failures and deformations.

6.2 Foundations

The boreholes drilled at the site encountered surficial topsoil or the pavement structure at the surface underlain by silty clay fill associated with the existing embankment, followed by a thin layer of buried topsoil above a stratum of generally very stiff to hard silty clay to elevation 227.5 metres. The silty clay is underlain by an extensive stratum of very stiff to hard clayey silt extending to the shale bedrock at elevations 210.7 to 212.5 metres.

The existing structure has pile foundations for the two abutment locations and a spread footing for the pier. The piles are short friction piles consisting of 325 millimetre diameter, concrete filled, steel tubes which were to be driven into the very stiff to hard clayey silt to about elevation 228.6 metres. The underside of footing for the shallow pier footing is at elevation 229.8 metres in the very stiff to hard clayey silt.

6.2.1 Existing Deep Foundations

The soil conditions encountered during the previous and current investigations indicate that the existing piles are suitable for the proposed retrofitting of the bridge structure. Details of ultimate pile capacity developed during construction are not known; however, a cursory examination of the structure did not detect any deformation related distress related to settlement of the foundations. Although evidence of post construction differential settlements related to distresses in the embankments in the approach area were noted at all four corners of the bridge, it is considered that such settlement is unrelated to the foundations. Therefore, semi-integral abutments could be supported on the existing steel tube piles. An estimated factored geotechnical resistance of 300 kilonewtons is available at ULS and a geotechnical resistance of 250 kilonewtons at SLS can be used for design. The SLS value is based on 25 millimetres of settlement. The existing piles were designed for an allowable design load of 250 kilonewtons per pile (working stress design).

6.2.2 Resistance to Lateral Loads

The lateral loading on the existing piles is considered to be resisted fully or partially by the existing battered piles. In this case, and for verification purposes, the horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

$$K_s = \frac{67S_u}{d} \text{ for cohesive soils}$$

d = pile width or diameter (m)

S_u = undrained shear strength of the soil (MPa)

Embankment fill (cohesive)
(Embankment Surface to 232 metres)

$S_u = 0.040 - 0.150$ MPa

Stiff to hard silty clay and clayey silt
(elevation 232 to 228 metres)

$S_u = 0.080 - 0.200$ MPa

Very stiff to hard clayey silt
(elevation 230 to 211 metres)

$S_u = 0.100 - 0.200$ MPa

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

<i>Pile Spacing in Direction of Loading, d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R^1</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The following maximum lateral resistances are recommended for 325 millimetre diameter tube piles:

<u>SOIL</u>	<u>UNFACTORED ULS (kPa)</u>	<u>SLS (kPa)</u>
Fill (Silty Clay)	220	125
Silty Clay	290	220
Clayey Silt	290	220

The SLS values assume a lateral movement of 10 millimetres.

6.2.3 Existing Shallow Foundations

The existing pier is founded on a shallow footing about 3.2 metres wide at approximately elevation 229.8 metres. A factored geotechnical resistance of 450 kilopascals at ULS and a geotechnical resistance of 300 kilopascals at SLS may be used in order to verify the structural capacity of the existing foundations.

6.3 Embankment Slope Stability

6.3.1 Site Reconnaissance

During a site reconnaissance carried out on September 13, 2007, by the Project Manager and Senior Field Supervisor, it was noted that signs of instability and post construction movements were present in the form of tension cracks in portions of the approach fills. The distress was documented in our September 14, 2007 letter to Delcan.

¹ Foundations and Earth Structures - Design Manual 7.2, NAVFAL DM-7.2 Department of the Navy, Naval Facilities Engineering Command (1982).

The side slopes parallel to Lambton County Road 79 and the abutment face slopes parallel to Highway 402 were measured at both approaches using an Abney hand level. At the north approach, the side slopes were found to be 25 and 23 degrees in the northwest and northeast quadrants, respectively. However, the slopes facing Highway 402 were found to be 30 and 33 to 34 degrees in the northwest and northeast quadrants, respectively. At the southern approach, both side slopes were approximately 25 degrees. The slopes facing Highway 402 were found to be 30 and 29 degrees in the southwest and southeast quadrants, respectively.

Cracks or distresses behind the slope crest were not observed. Cracking of the slope face was noted in the northeast and southeast quadrants of the approach fills as shown in Photographs 3 to 7 in Appendix C. Differential settlements between the structure and the curbs and gutters, on the approach slabs and approach fills, was noted at all four corners of the bridge. These settlements were most apparent at the southeast corner of the bridge. The slope face is moderately vegetated with grass. However, several bare spots were evident. Beneath the structure, the slopes are covered with rip rap.

6.3.2 Embankment Configuration

The Highway 79 Interchange Underpass structure was constructed in the late 1970s. According to Department of Transportation and Communications, Ontario Drawing No. D-7063-1 entitled “General Layout – Highway 79 Interchange Underpass” dated March 1972, the approach embankments were to have 2 horizontal to 1 vertical (2:1) slopes in the northwest and southeast quadrants and 2.5 horizontal to 1 vertical slopes in the northeast and southwest quadrants. Abutment slopes facing Highway 402 were to be 2:1 with varying slopes expected in areas of transition between the 2.5:1 and 2:1 slopes. At the abutment locations, the embankments were to be approximately 6 metres high. The slope faces under the overpass structure were to be protected with crushed rock. The drawing depicts a typically 2.1 metre wide shoulder at the toe of each abutment slope adjacent to the Highway 402 variable width speed change lanes. The distance between the centerline of the abutment and the centerline of Highway 402 was to be 41.8 metres. The median was to be 39.6 metres wide. The distance between the Highway 402 centreline and outer edge of pavement was to be about 30.8 metres.

As-built drawings of the structure are not available to allow verification of the actual configuration of the approach embankments. Based on review of the Contract Drawings for Contract No. 78-40, dated February 23, 1978 provided by the County of Lambton Public Works Department, the layout of the lanes, median and shoulders of Highway 402 and the length of the structure was consistent with the design drawings. Insufficient information was available to determine the size and slopes of the approach embankments. However, it was noted that the embankment toe was depicted within the 2.44 metre wide shoulder area. The results of the field investigation revealed that the existing embankments are composed of firm to very stiff but generally very stiff silty clay fill constructed on stiff to hard silty clay overlying an extensive

deposit of very stiff to hard clayey silt. Layers of buried topsoil, 60 to 390 millimetres thick were found below the fills in boreholes 201, 203 and 204.

Delcan provided cross sections of the approach embankments based on survey information obtained by Callon Dietz Incorporated. The survey confirmed that the embankment height, layout of the Highway 402 median, shoulders and lanes were consistent with the original design. However, as noted during the site reconnaissance, the gradients of the approach slopes were steeper at some locations. Measured slope inclinations which are equal to or flatter than those indicated on the original design drawing are considered to meet the design. The design and measured slope inclinations are compared in the following table:

LOCATION	SLOPE INCLINATION (horizontal: vertical / degrees)			
	DESIGN	MEASURED (Site Reconnaissance)	MEASURED (CROSS-SECTIONS)	MEETS DESIGN SLOPE (x - no, ✓ - yes)
NE Quadrant – approach slope	2.0:1 / 26.5	1.5:1 / 33 / 34	1.6:1 / 32	x
NE Quadrant – side slope	2.5:1 / 21.8	2.4:1 / 23	2.8:1 / 20	✓
SE Quadrant – approach slope	2.0:1 / 26.5	1.8:1 / 29	1.9:1 / 27	x
SE Quadrant – side slope	2.0:1 / 26.5	2.1:1 / 25	2.4:1 / 22	✓
SW Quadrant – approach slope	2.0:1 / 26.5	1.7:1 / 30	2.0:1 / 27	x
SW Quadrant – side slope	2.5:1 / 21.8	2.1:1 / 25	2.6:1 / 21	✓
NW Quadrant – approach slope	2.0:1 / 26.5	1.7:1 / 30	1.8:1 / 29	x
NW Quadrant – side slope	2.0:1 / 26.5	2.1:1 / 25	2.6:1 / 21	✓
Average – approach slope	-	1.7:1 / 31.2	1.8:1 / 29	-
Average – side slope	-	2.2:1 / 25	2.6:1 / 21	-

The information from the site survey suggests that, in the vicinity of the approaches, the embankment side slopes (parallel to Lambton County Road 79) range from 20 to 22 degrees and are generally flatter than design. Based on the approximate measurements taken during the site reconnaissance, localized areas of side slopes which are significantly steeper than design are present in the northeast and southwest quadrants.

According the site survey information, the approach slopes (facing Highway 402) immediately adjacent to the abutments are steeper than the design inclination of 2.0:1 (26.5 degrees). The slope of the existing side slopes (parallel to Lambton County Road 79) were 2.4 to 2.8 horizontal to 1 vertical (20 to 22 degrees).

6.3.3 Check of Existing Stability

Several slope stability analyses were conducted to assess the stability of the existing embankment and to examine the remedial options which included slope flattening and addition of toe berms. The analyses were conducted using SLOPE/W 2004, a limit equilibrium software program produced by GEO-SLOPE/W International Ltd. Slope stability analyses were carried out using

the soil characteristics and groundwater levels determined in the boreholes and the survey information provided by Delcan.

The stability of the existing approach slopes was checked assuming that the embankment fill slope will fail locally with a shallow failure through the toe and globally with a deeper circle within the upper silty clay deposits.

For shallow failures, occurring in the embankment fill, the embankment side slopes that are inclined at 20 to 21 degrees are stable and have local factors of safety greater than 1.3. The face of the approaches are sloped at 27 to 32 degrees. The approach slopes are considered to be unstable to marginally unstable with local factors of safety varying from 0.92 for a 32 degree slope to 1.05 for a 27 degree slope. Evidence of instability, in the form of tension cracks and differential movement between the structure embankment fills, was observed in the field.

A global factor of safety for deep seated failures of 1.3 or greater is available for slopes inclined at 26 degrees (2:1) or flatter. The main embankment side slopes are assumed to be globally stable since the all measured side slopes are flatter than 26 degrees. The approach slopes facing Highway 402 have slopes of 27 to 32 degrees with associated global factors of safety of 1.22 to 1.24. Although the existing approach slopes can be considered stable, the existing factors of safety are less than 1.3, a value that represents a typical minimum factor of safety for a highway embankment. These values of about 1.2 are associated with slope movements/slumping.

6.3.4 Remediation of Approach Slopes

A variety of options were examined with respect to remediation of the approach slopes. One key limitation of the adoption of some options will be limited space. According to the drawings provided, for each approach, there is likely 2.4 metres or less available between the existing edge of pavement and toe of the approach slope. The following remediation options which result in factors of safety of 1.3 or greater were considered:

1. Slope flattening
2. Slope flattening with toe wall
3. Toe berm
4. Slope reconstruction and reprofiling
5. Slope reinforcement (using geogrids)
6. Soil Nailing

A comparison of the remediation alternatives is give in Table I.

Slope Flattening

The slope can be stabilized by flattening and extending the toe of the slope. An inclination of 23 degrees or less will be required to produce a factor of safety of 1.3 or greater for both local and global stability. This remediation option is simple to implement; however, it may not be feasible since there is insufficient space to extend the toe of the slope into the areas currently used as the outer shoulders and variable speed change lanes.

It was noted that the original design drawings show a 39.6 metre wide median of which the outermost 3.65 metres on each side are reserved for future lanes. Therefore it may be possible to do some realignment of Highway 402 within the interchange area to accommodate the extended slope. Realignment of the road involves additional costs associated with traffic delays and disruption and may not be compatible with the current and proposed configurations of the interchange ramps.

Slope Flattening With Toe Wall

The slopes could be reduced to a stable inclination of 23 degrees by placement of additional fill which is restrained by retaining walls. The retaining walls would be situated at the location of the existing toes. The required maximum wall height is in the order of 2 metres where the original slope was 32 degrees and 1.6 metres where the original slope was 29 degrees. The wall segments would follow the toes of the existing embankments and would diminish in height with increasing distance from the structure. It is anticipated that no wall segments will be required beyond a distance of 15 metres from the centreline of the structure where lower stable slopes inclined at approximately 26 degrees are present. Wall segments need not be constructed under the structure where the crushed rock slope protection is in place.

This rehabilitation strategy is a preferred technical solution and has the advantages of the previous option but does not encroach upon the travelled area.

Toe Berm

A toe berm consisting of Granular A could be added to the base of the slope. For example, a Granular A toe berm, sloped at 26 degrees with a height approximately two-thirds of the embankment height, could achieve the minimum factor of safety of 1.3 provided that the upper slope is cut back to 1 metre behind the crest. Addition of a toe berm will encroach onto the road as with slope flattening.

Slope Reconstruction and Reprofiling

The upper 2 metres of fill material could be removed and replaced with Granular A fill at a 26 degree (2.1 horizontal to 1 vertical) slope. The advantages of this strategy are a reduction of the likelihood of surficial failures which could occur if only a toe berm is applied and preservation of the footprint of the existing approach fills. It is therefore the preferred solution from a foundations engineering perspective.

Excavations are not to expose the underside of the abutment pile caps. Should this occur, temporary shoring must be installed. Generally it is not anticipated that excavations will expose the pile caps except for a very small area at the toe of the abutment wall where the base of a 2 metre deep excavation will be at or slightly below the underside of pile cap elevation. Although this condition is not expected to adversely affect the structure, temporary shoring will be required as a precaution.

These excavations will be in very close proximity to the existing abutment footings/pile caps, and care must be taken not to undermine the pile caps.

Slope Reinforcement (Using Geogrid)

The oversteepened and distressed slope segments could be reconstructed using geogrid reinforcement. The chief advantages of this method is that the existing embankment fill could be reused and the slope could be rebuilt using an inclination of 2 horizontal to 1 vertical which allows for preservation of the existing right-of-way width. Construction using granular material such as Granular A will result in increased costs but a more reliable design. The appearance of the slope can be maintained with the reestablishment of vegetation upon completion of construction.

It is recommended that the length of the reinforcing strip be approximately 7 metres long plus an additional minimum length of 1 metre for embedment. A factor of safety of 2 is recommended for pull out resistance if the cohesive fill is reused and 1.5 if they are replaced with granular fills. Following the proper storage and installation procedures recommended by the manufacturer are critical when using geotextiles. In addition, there are concerns about the long-term durability of buried polymer reinforcement. The slope must be provided with subsurface drains and a diversion ditch or drain at the crest of the wall prevent runoff from eroding the slope face.

Soil Nailing

Consideration was given to the use of soil nails to stabilize the embankment slopes. However, pre-fabricated or cast-in-place concrete panels would have to be added to the shotcrete facing to

improve aesthetics, durability and accommodate drainage and insulation would be required for frost protection.

In addition, the clayey embankment fill would require only low capacity nails and costs would be high.

6.3.5 Comparison of Remediation Options

A comparison of the advantages, disadvantages, estimated costs and risks/consequences of each remedial option is presented in Table I. The estimated costs are preliminary and are intended to provide a cost comparison from a relative rather than an absolute basis. The costs of traffic protection, and slope restoration (sodding and seeding, provision of erosion protection, etc.) have not been included.

Options which allow for preservation of the existing right-of-way, that minimize construction costs and improve overall global and local stability are preferred from a technical standpoint. These options are slope reconstruction and reprofiling, slope reinforcement and soil nailing. Of these three, slope reconstruction with Granular A may prove to be the most cost effective due to the relatively small area affected. Although slope reinforcement is more attractive than soil nailing and may be attractive from an aesthetic point of view, there are concerns about the longevity of the polymer reinforcement which is subject to ultraviolet, chemical and biological degradation especially if handled poorly during construction.

6.3.6 Construction Considerations

All remedial grading of the embankments is to conform with the requirements of SP206S03 using clean earth fill as required by OPSS 212 or granular fill which satisfies OPSS 1010. Buried topsoil material are present at the base of the embankment fills. If the buried topsoil materials are exposed during remedial works, they should be removed. However, noting the limited thickness of topsoil, 90 to 390 millimetres, and the extensive work required to remove these materials, their removal from the entire footprints of the embankment fills is not considered warranted. Additional fill added to the existing embankment for remedial purposes must be benched into the existing fill in accordance with OPSD 208.010. All embankment fill should be placed and compacted in regular lifts no thicker than 300 millimetres loose.

Irrespective of the selected remedial option, adequate erosion protection must be provided to the earthen portion of the slopes in order to mitigate against shallow failures. The slopes should be topsoiled, sodded and seeded in accordance with current MTO standards.

6.4 Retaining Wall For Slope Remediation

One of the technical solutions for remediation of the existing unstable approach faces considers placement of additional fill to reduce the inclination and construction of a toe wall. If this option is selected, the toe wall could consist of an RSS wall, concrete cantilever wall or soldier pile and lagging wall.

The wall could be supported on spread footings founded on the very stiff to hard silty clay near elevation 231.0 metres. A factored geotechnical resistance at ULS of 450 kilopascals and 300 kilopascals at SLS may be used for design purposes. The SLS value allows for 25 millimetres of settlement.

Alternatively, an RSS wall footing designed with the geotechnical resistances given above may be founded on a 0.3 metre thick compacted Granular A leveling pad constructed on the surface of the silty clay. It is anticipated that the reinforced width of the RSS wall would be approximately 75 per cent of the wall height.

6.5 Lateral Earth Pressures

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

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawings (OPSD) 3101.150, 3190.100 and 3121.150.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Figure 6.9.3. Compaction equipment should be used in accordance with SP 105S10.
- In accordance with CHBDC Clause C6.9.1, the granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).

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

Soil unit weight:	20 kN/m ³
Angle of internal friction, ϕ	26°
Coefficients of lateral earth pressure:	
Active, K_a	0.39
At rest, K_o	0.56

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

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>(Type III)</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. For sloping backfill/ground surface, these parameters should be adjusted as indicated in CHBDC C6.9.1(e).

6.6 Excavations and Temporary Roadway Protection

Excavations for the semi-integral abutment conversions will only penetrate the embankment fill behind the abutments to about 1.5 metres below the surface of the existing deck. Remediation of the approach fill slopes will require excavation of some of the existing embankment fill and will penetrate the underlying silty clay subgrade if the remedial works incorporate a retaining wall. If any of the buried topsoil layers such as those encountered in boreholes 201, 203 and 204 are encountered, they should be removed from the limits of the excavation.

Excavations for the abutment conversions and rehabilitation are not expected to intercept the groundwater table. Excavations which extend into the native silty clay may penetrate below the groundwater level, however, minimal groundwater seepage is expected. Any seepage from the fill layers may be controlled using properly filtered sumps. All surface water must be directed away from the excavations. 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 and the silty clay, clayey silt and silt materials would be Type 1 or 2 depending on consistency or density.

Temporary Roadway Protection

Temporary roadway protection (shoring) will have to be provided to support the existing embankment fills and excavations in accordance with SP 105S19 when construction of the new semi-integral abutments is carried out or if required as part of the scheme for remediation of the approach slopes. The shoring should be designed to Performance Level 2.

The temporary support system could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds or driven steel sheet piling. Support to the system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection.

The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

The support systems may be designed using the following parameters:

<u>SOIL TYPE</u>	<u>COEFFICIENT OF EARTH PRESSURE</u>			<u>INTERNAL ANGLE OF FRICTION (degrees)</u>	<u>UNIT WEIGHT (kN/m³)</u>
	<u>Active, K_a</u>	<u>At Rest, K_0</u>	<u>Passive, K_p</u>		
Clayey Fill	0.39	0.56	2.6	26	20
Clayey Silt	0.33	0.50	3.0	30	20
Silty Clay	0.39	0.56	2.6	26	20

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

7.0 MISCELLANEOUS

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

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DB/DUP/PRB/FJH/cr
n:\active\2007\1130 - geotechnical\1130-1000\07-1130-128-4 delcan - fnds new n to w ramp - watford\reports\2 underpass site stn 14+355\apr 15 08 - (final) parts a & b - cty rd
79 underpass site 14-355.doc

TABLE I

COMPARISON OF REMEDIATION ALTERNATIVES FOR APPROACH SLOPES

Site 14-355
Lambton County Road 79 Underpass
Structural Rehabilitation
Highway 402 and Lambton County Road 79 Improvements
GWP 3158-06-00

REMEDICATION OPTION	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Slope Flattening	<ul style="list-style-type: none"> Improves global and local stability 	<ul style="list-style-type: none"> This strategy will result in encroachment onto Highway 402 High potential to be costly particularly if realignment affects design of interchange on each side of structure Potential for lengthy construction 	<ul style="list-style-type: none"> Supply and place fills: \$30 to \$50 per m³; lower end for earth fill, higher end for granular fills 	<ul style="list-style-type: none"> May require costly realignment of Highway 402 and ramps Highly disruptive to traffic This option may not be compatible with the existing and proposed configuration of the interchange
Slope Flattening with Toe Wall	<ul style="list-style-type: none"> Improves global and local stability Does not encroach into travelled area Preferred technical option. 	<ul style="list-style-type: none"> Viewed as having poor aesthetics 	<ul style="list-style-type: none"> Supply and place fills: \$30 to \$50 per m³; lower end for earth fill, higher end for granular fills Wall construction: \$420 to \$550 per m² for concrete cantilever wall and \$300 to \$600 per m² for RSS wall 	<ul style="list-style-type: none"> Use of architectural wall facings can improve aesthetics
Toe Berm (in combination with slope flattening)	<ul style="list-style-type: none"> Improves overall stability Avoids wall construction 	<ul style="list-style-type: none"> Encroaches onto travelled area Surficial failures could still occur above berm May be difficult to establish vegetation on granular berm 	<ul style="list-style-type: none"> Sub-excavation of existing fills and supply and place fills: \$65 to \$75 per m³ 	<ul style="list-style-type: none"> Does not resolve surficial failures occurring on upper slope

COMPARISON OF REMEDIATION ALTERNATIVES FOR APPROACH SLOPES

REMEDICATION OPTION	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Slope Reconstruction and Reprofilng	<ul style="list-style-type: none"> Improves global and local stability Does not encroach into travelled area 	<ul style="list-style-type: none"> Excavations in close proximity to the existing abutment may require temporary shoring May be difficult to establish vegetation on granular berm Requires disposal of sub-excavated fills 	<ul style="list-style-type: none"> Remove existing fill and supply and place Granular A fills: \$50 to \$70 per m³ Shoring: \$300 per m² (if required) 	<ul style="list-style-type: none"> If excavations adjacent to the structure are not properly supported, the abutment foundations may experience excessive movement
Slope reinforcement (using geogrid)	<ul style="list-style-type: none"> Improves global and local stability with steeper slope than for slope flattening Finished appearance similar to existing slope Possibility of reusing existing fill Does not encroach into travelled area 	<ul style="list-style-type: none"> Uncertainties with long-term performance of geogrid reinforcement If slope is steeper than 2 horizontal to 1 vertical, increased difficulty with performing maintenance activities (grass cutting etc.) on a steeper slope 	<ul style="list-style-type: none"> \$200 to \$300 per m². 	<ul style="list-style-type: none"> Must use reinforcement that is proven to be effective for the long-term Stresses imposed on reinforcement during construction must be considered during design
Soil Nailing	<ul style="list-style-type: none"> Improves global and local stability Flexible method 	<ul style="list-style-type: none"> Operation of heavy equipment, trucks, etc. near the wall must be limited during construction Requires experienced specialist contractor Placement of nails may be limited by obstructions or buried utilities 	<ul style="list-style-type: none"> \$215 to 325 per m² 	<ul style="list-style-type: none"> Requires clear area behind wall for installation of nails May need to limit truck traffic on Underpass structure

- NOTES:
1. Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs. Cost of traffic protection and slope restoration not included.
 2. Table to be read in conjunction with accompanying report.

Prepared By: DUP
Checked By: PRB

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

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.)

Consistency

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

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

PROJECT <u>07-1130-128-4-2</u>		RECORD OF BOREHOLE No 201		1 OF 1		METRIC	
G.W.P. <u>3158-06-00</u>		LOCATION <u>N 4761446.1 ; E 355406.5</u>		ORIGINATED BY <u>DB</u>			
DIST <u> </u> HWY <u>402</u>		BOREHOLE TYPE <u>MANUAL AUGER (UNCASED)</u>		COMPILED BY <u>JAS</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 21, 2007</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
234.44	GROUND SURFACE																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				</

LDN_MTO_01 07-1130-128-4-2.GPJ LDN_MTO.GDT 4/16/08

PROJECT <u>07-1130-128-4-2</u>		RECORD OF BOREHOLE No 202		1 OF 1		METRIC	
G.W.P. <u>3158-06-00</u>		LOCATION <u>N 4761609.5 ; E 355406.1</u>		ORIGINATED BY <u>DB</u>			
DIST <u> </u> HWY <u>402</u>		BOREHOLE TYPE <u>MANUAL AUGER (UNCASED)</u>		COMPILED BY <u>JAS</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 21, 2007</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						W _p W W _L				GR	SA	SI	CL	
								20	40	60	80	100	20	40	60	80						100
234.51	GROUND SURFACE																					
0.09	TOPSOIL, silty Brown FILL, silty clay, trace to some sand, trace gravel, trace topsoil Stiff to very Stiff Brown						234															
			1	SS	9																	
			2	SS	15		233															
232.22																						
2.29	SILTY CLAY, trace sand, trace gravel, trace topsoil Stiff Brown		3	SS	13		232															
231.77																						
2.74	SILTY CLAY, trace sand, silt lenses Very stiff Brown		4	SS	25																	
231.16																						
3.35	END OF BOREHOLE																					
	Borehole dry during drilling on November 21, 2007																					

PROJECT 07-1130-128-4-2		RECORD OF BOREHOLE No 203		1 OF 1	METRIC
G.W.P. 3158-06-00		LOCATION N 4761461.1 ; E 355397.4		ORIGINATED BY DB	
DIST HWY 402		BOREHOLE TYPE POWER AUGER (UNCASED)		COMPILED BY JAS	
DATUM GEODETIC		DATE November 28, 2007		CHECKED BY	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE									
237.96	GROUND SURFACE																			
0.00	TOPSOIL, silty																			
0.15	Brown																			
237.50	FILL, sand and gravel, trace silt																			
0.46	(crushed)																			
	Brown																			
	FILL, silty clay, trace sand, trace		1	SS	12															
	gravel																			
	Stiff to very stiff		2	SS	10															
	Brown																			
			3	SS	18															
			4	SS	19															
			5	SS	14															
			6	SS	21															
232.41			7	SS	20															
5.55	TOPSOIL, silty																			
232.02	Compact Brown																			
5.94	SILTY CLAY, trace to some sand,																			
	trace gravel		8	SS	12															
	Stiff																			
	Brown																			
231.25																				
6.71	SILTY CLAY, trace sand		9	SS	35															
	Hard																			
	Brown																			
			10	SS	35															
230.04																				
7.92	SILT, trace clay																			
229.73	Dense Brown																			
8.23	SILTY CLAY, trace sand																			
	Very stiff																			
	Grey		11	SS	20															
229.12																				
8.84	END OF BOREHOLE																			
	Groundwater encountered at about elev. 230.0m during drilling on November 28, 2007.																			
	Water level measured in standpipe at elev. 229.40m on November 28, 2007.																			
	Water level measured in standpipe at elev. 232.72m on December 19, 2007.																			
	Water level measured in standpipe at elev. 232.37m on January 7, 2008.																			
	Water level measured in standpipe at elev. 232.05m on January 17, 2008.																			
	Water level measured in standpipe at elev. 232.02m on January 30, 2008.																			

LDN_MTO_01 07-1130-128-4-2.GPJ LDN_MTO.GDT 4/16/08

RECORD OF BOREHOLE No 204

1 OF 1

METRIC

PROJECT 07-1130-128-4-2
G.W.P. 3158-06-00 LOCATION N 4761624.5 ; E 355397.9 ORIGINATED BY DB
DIST HWY 402 BOREHOLE TYPE POWER AUGER (UNCASED) COMPILED BY JAS
DATUM GEODETIC DATE November 28, 2007 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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)					
237.89	GROUND SURFACE						20	40	60	80	100	10	20	30		GR SA SI CL	
0.00	ASPHALT																
0.24	FILL, sand and gravel, trace silt (crushed) Brown																
0.46	FILL, silty clay, trace sand, trace gravel, sand pockets Stiff to very stiff Brown		1	SS	16								○				
			2	SS	13								○	—————		1 5 57 37	
			3	SS	20								○				
			4	SS	10								○	—————		0 3 58 39	
			5	SS	21								○				
			6	SS	20								○				
			7	SS	17								○				
232.25	TOPSOIL, silty Compact Brown		8	SS	22												
5.64	SILTY CLAY, trace sand Very stiff Brown		9	SS	23												
231.95			10	SS	31								○			0 1 91 8	
5.94			11	SS	14												
230.03	SILT, trace clay Dense Brown																
7.86	SILTY CLAY, with silt lenses Stiff Grey																
229.66																	
8.23																	
229.05	END OF BOREHOLE																
8.84	Groundwater encountered at about elev. 230.0m during drilling on November 28, 2007. Water level measured in standpipe at elev. 229.10m on November 28, 2007. Water level measured in standpipe at elev. 231.79m on December 19, 2007. Water level measured in standpipe at elev. 231.03m on January 7, 2008. Water level measured in standpipe at elev. 231.18m on January 17, 2008. Water level measured in standpipe at elev. 231.09m on January 30, 2008.																



REFERENCE

DRAWING BASED ON CANMAP STREETFILES
V2005.4

NOTES

THIS DRAWING IS SCHEMATIC ONLY AND IS TO BE READ
IN CONJUNCTION WITH ACCOMPANYING TEXT.

ALL LOCATIONS ARE APPROXIMATE ONLY.

PROJECT

LAMBTON COUNTY ROAD 79 UNDERPASS
HIGHWAY 402 & LAMBTON COUNTY ROAD 79 IMPROVEMENTS
GWP 3158-06-00

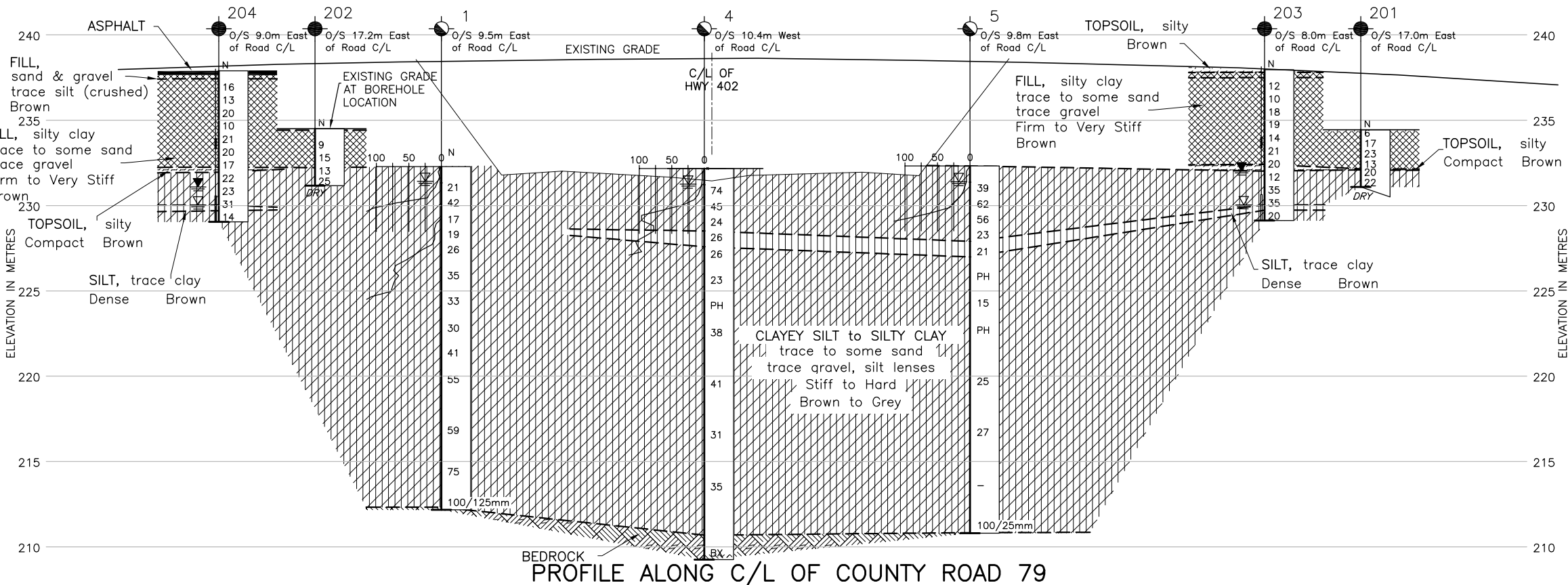
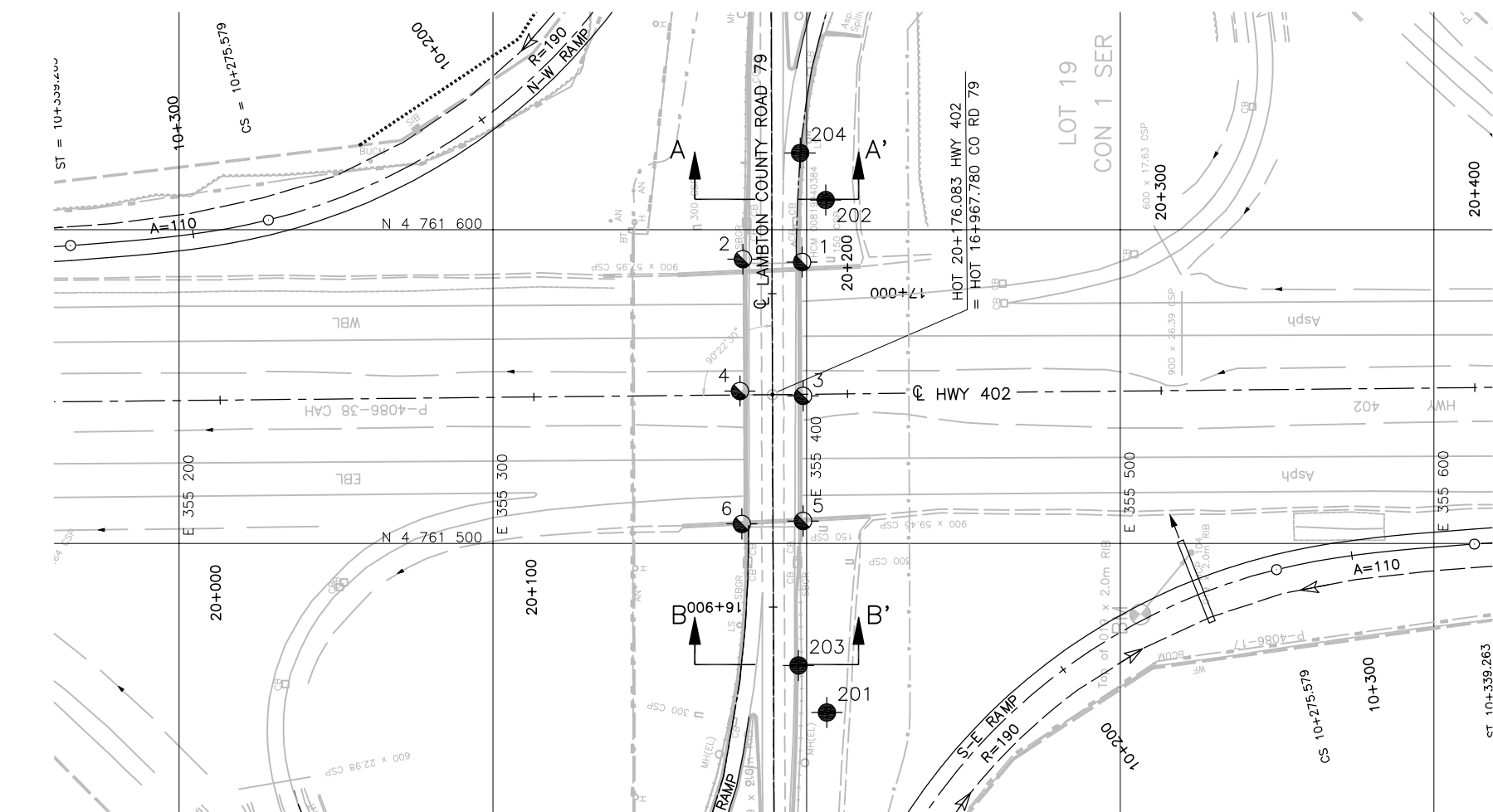
TITLE

KEY PLAN



**Golder
Associates**
LONDON, ONTARIO

PROJECT No.		07-1130-128-4	FILE No.		0711301284-2-F01001
CADD	WDF	Apr. 15/08	SCALE	AS SHOWN	REV. 0
CHECK			FIGURE 1		



PROFILE ALONG C/L OF COUNTY ROAD 79



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

CONT No.
WP No. 3158-06-00

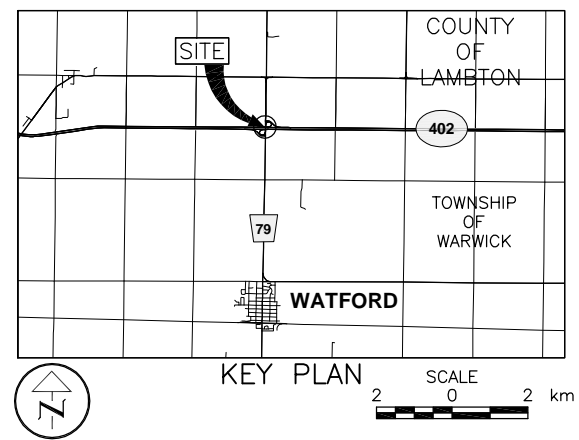


SHEET

HWY 402/LAMBTON CTY RD 79
IMPROVEMENTS
LAMBTON COUNTY ROAD 79 UNDERPASS
BOREHOLE LOCATION AND SOIL STRATA



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Borehole and Cone - Previous Investigation (by Others)
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in standpipe, measured on JAN 30, 2008.
- WL encountered during drilling
- DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM Zone 11)	
		NORTHING	EASTING
201	234.44	4 761 446.1	355 406.5
202	234.51	4 761 609.5	355 406.1
203	237.96	4 761 461.1	355 397.4
204	237.89	4 761 624.5	355 397.9
Borehole Elevation & Co-ordinates (inferred from Previous Data)			
1	232.29	4 761 589.7	355 398.6
2	232.23	4 761 590.7	355 379.7
3	232.41	4 761 547.1	355 398.9
4	232.17	4 761 548.6	355 378.8
5	232.32	4 761 507.1	355 398.9
6	232.17	4 761 506.2	355 379.4

NOTES

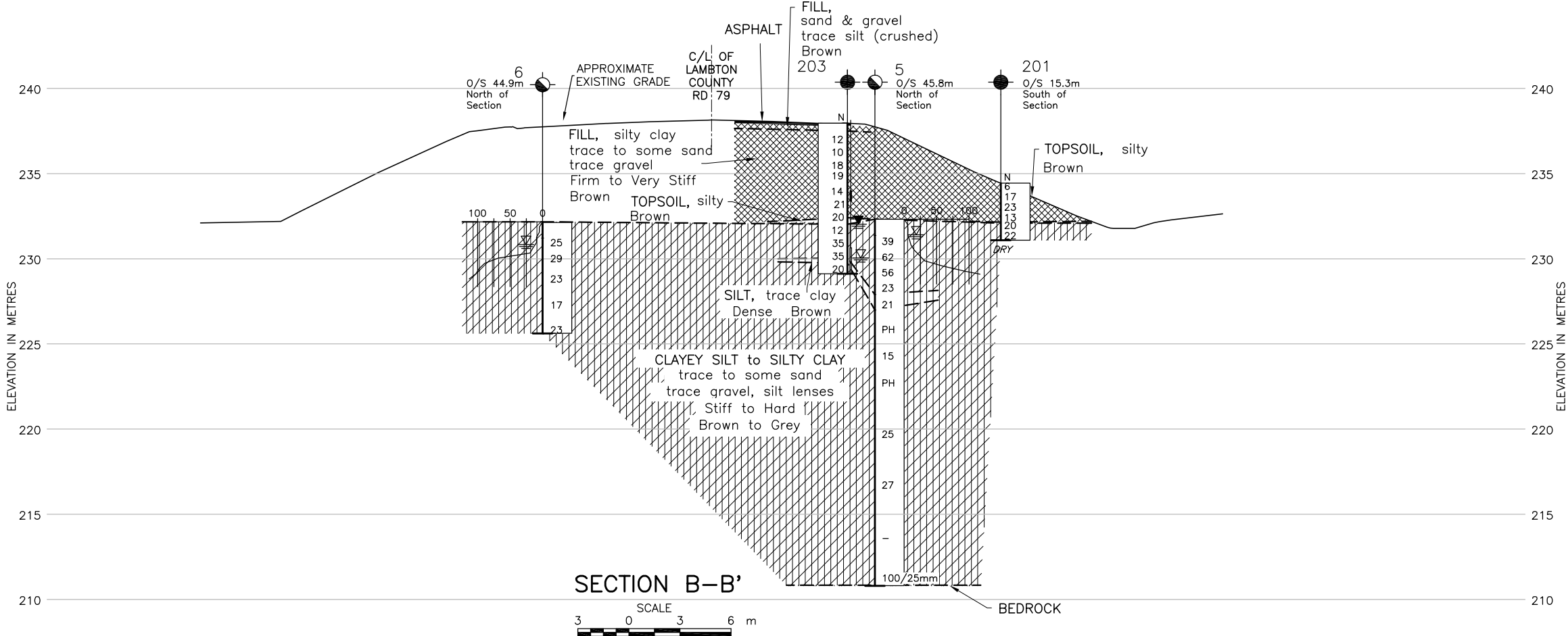
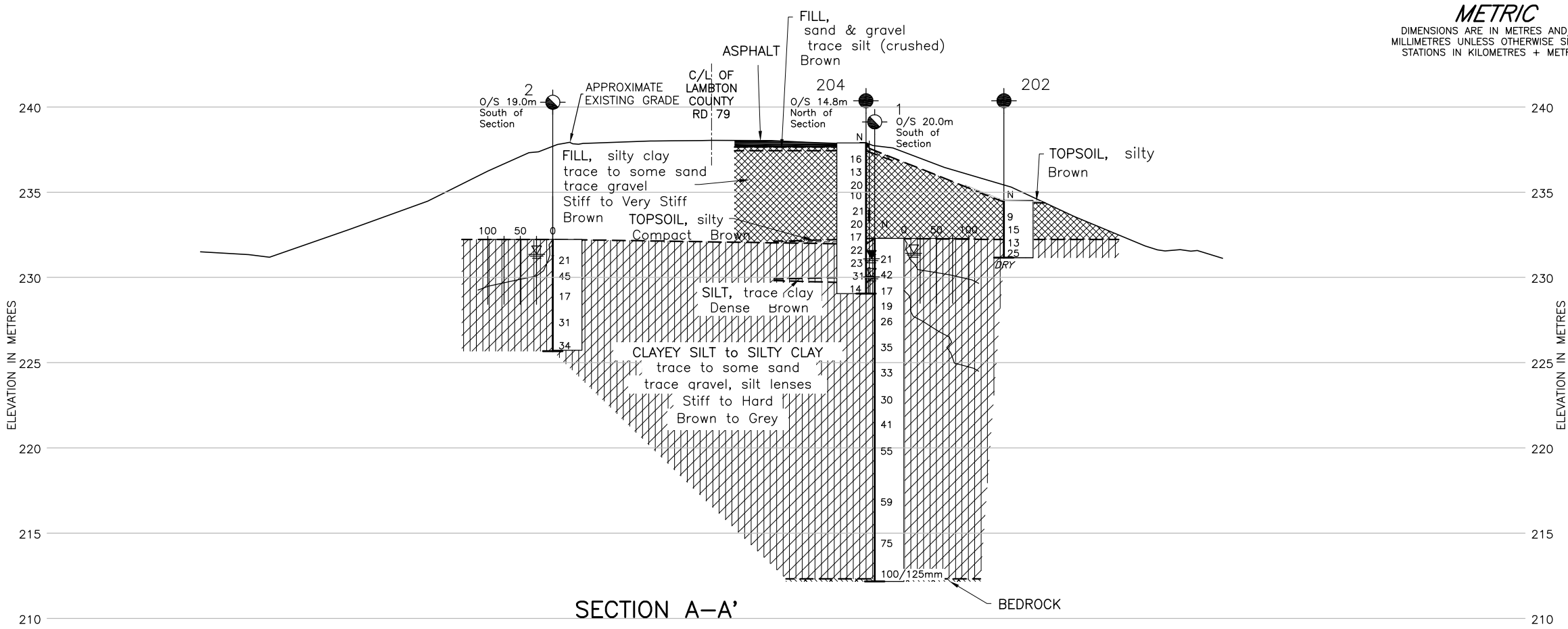
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Base plans provided in digital format by DELCAN

NO.	DATE	BY	REVISION
Geocres No. 40113-54			
HWY.	402	PROJECT NO.	07-1130-128-4
SUBM'D.	DB	CHKD.	DUP
DRAWN:	WDF	CHKD.	APPD.
DIST.		SITE: 14-355	
DATE: APR 15/08		DWG. 1	



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

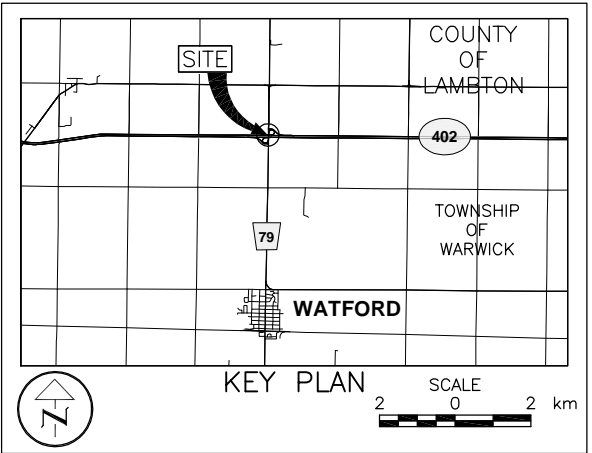
CONT No.
WP No. 3158-06-00

HWY 402/LAMBTON CTY RD 79 IMPROVEMENTS
LAMBTON COUNTY ROAD 79 UNDERPASS
SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Borehole and Cone - Previous Investigation (by Others)
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in standpipe, measured on JAN 30, 2008.
- WL encountered during drilling
- DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM Zone 11)	
		NORTHING	EASTING
201	234.44	4 761 446.1	355 406.5
202	234.51	4 761 609.5	355 406.1
203	237.96	4 761 461.1	355 397.4
204	237.89	4 761 624.5	355 397.9
Borehole Elevation & Co-ordinates (inferred from Previous Data)			
1	232.29	4 761 589.7	355 398.6
2	232.23	4 761 590.7	355 379.7
3	232.41	4 761 547.1	355 398.9
4	232.17	4 761 548.6	355 378.8
5	232.32	4 761 507.1	355 398.9
6	232.17	4 761 506.2	355 379.4

NOTES

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

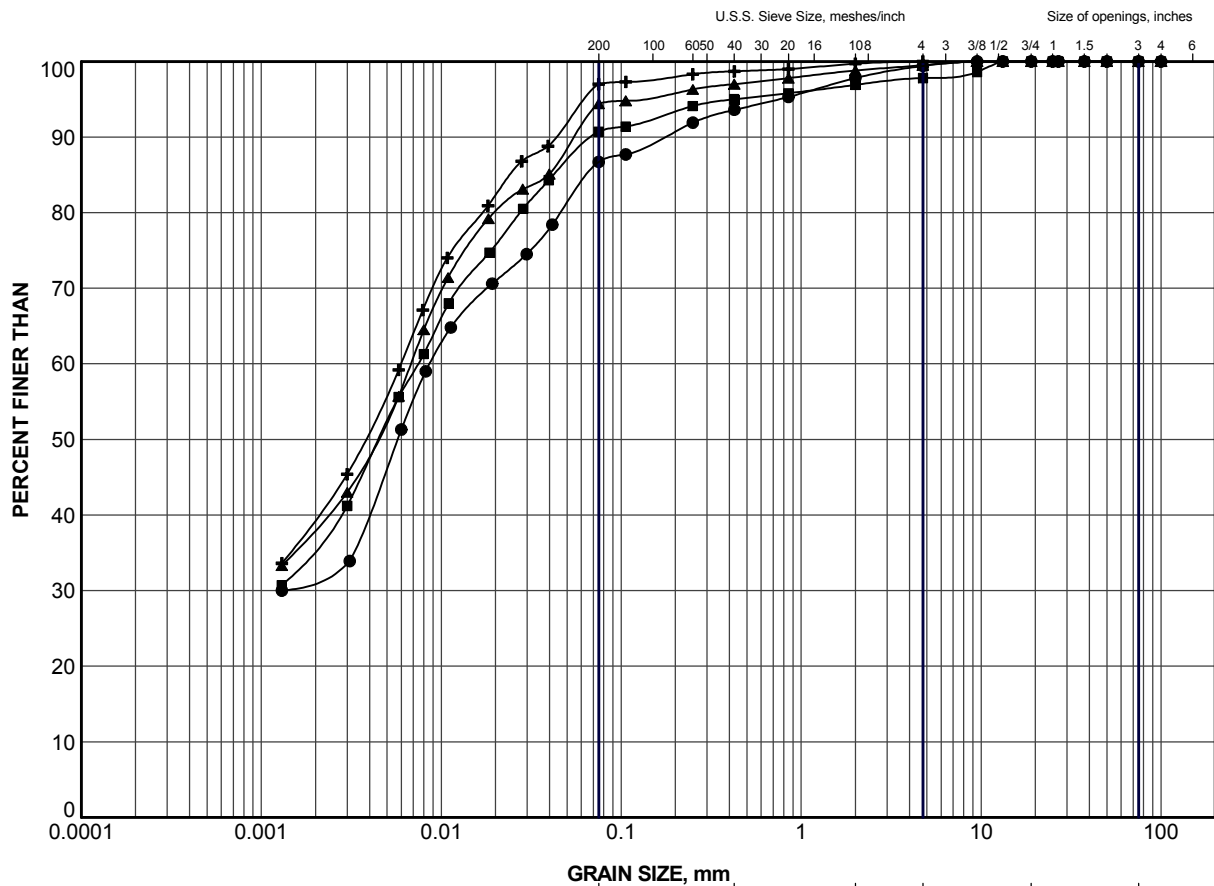
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by DELCAN

NO.	DATE	BY	REVISION
Geocres No. 40113-54			
HWY.	402	PROJECT NO.	07-1130-128-4
SUBM'D.	DB	CHKD.	DUP
DRAWN:	WDF	CHKD.	APPD.
DIST.		SITE: 14-355	
DWG.		2	


APPENDIX A
LABORATORY TEST DATA

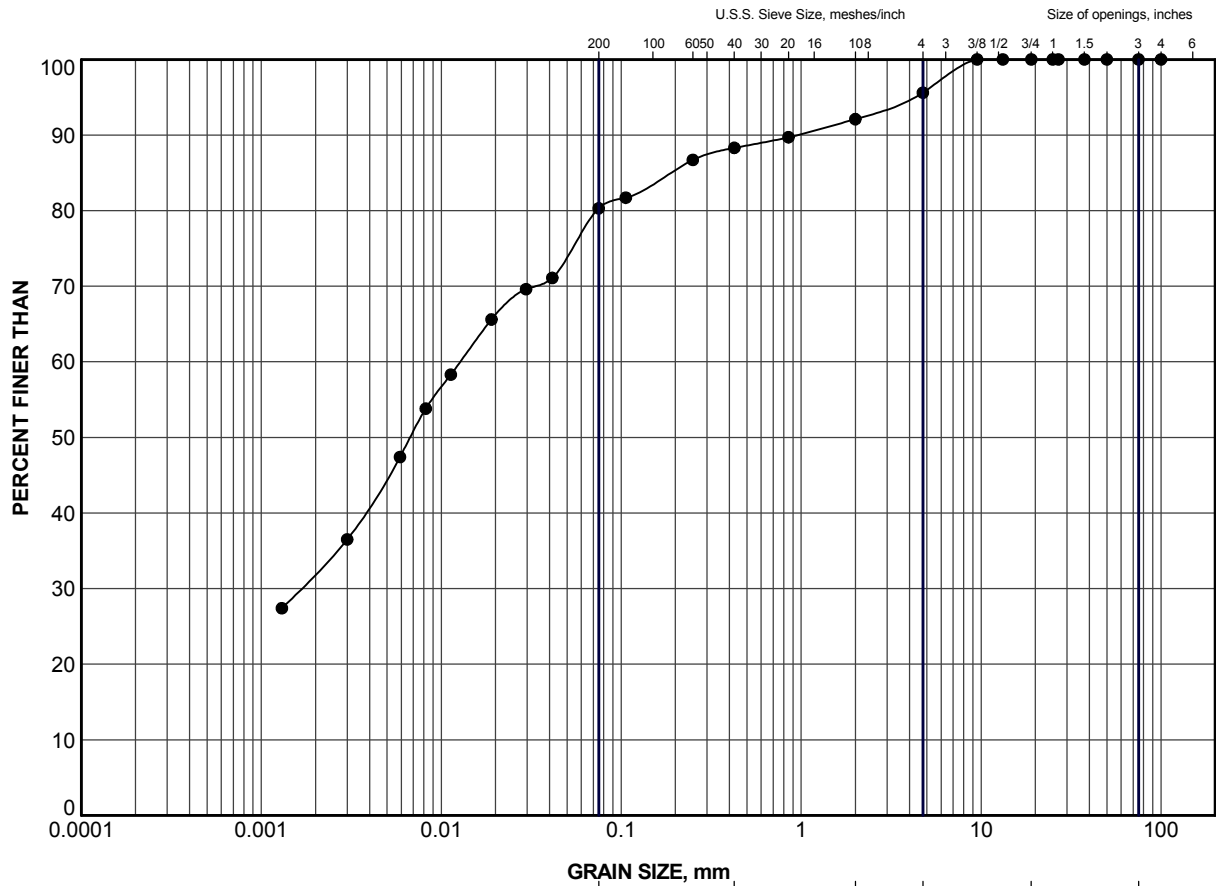


CLAY AND SILT	SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	3	233.0
■	203	4	234.7
▲	204	2	236.2
✚	204	4	234.6

PROJECT				LAMBTON COUNTY ROAD 79 UNDERPASS HIGHWAY 402 & LAMBTON COUNTY ROAD 79 IMPROVEMENTS GWP 3158-06-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL, silty clay			
PROJECT No.		07-1130-128-4		FILE No.		0711301284-2-R010a1	
DRAWN		WDF		SCALE		N/A	
CHECK		Feb. 28/08		REV.			
 Golder Associates LONDON, ONTARIO				FIGURE A-1			

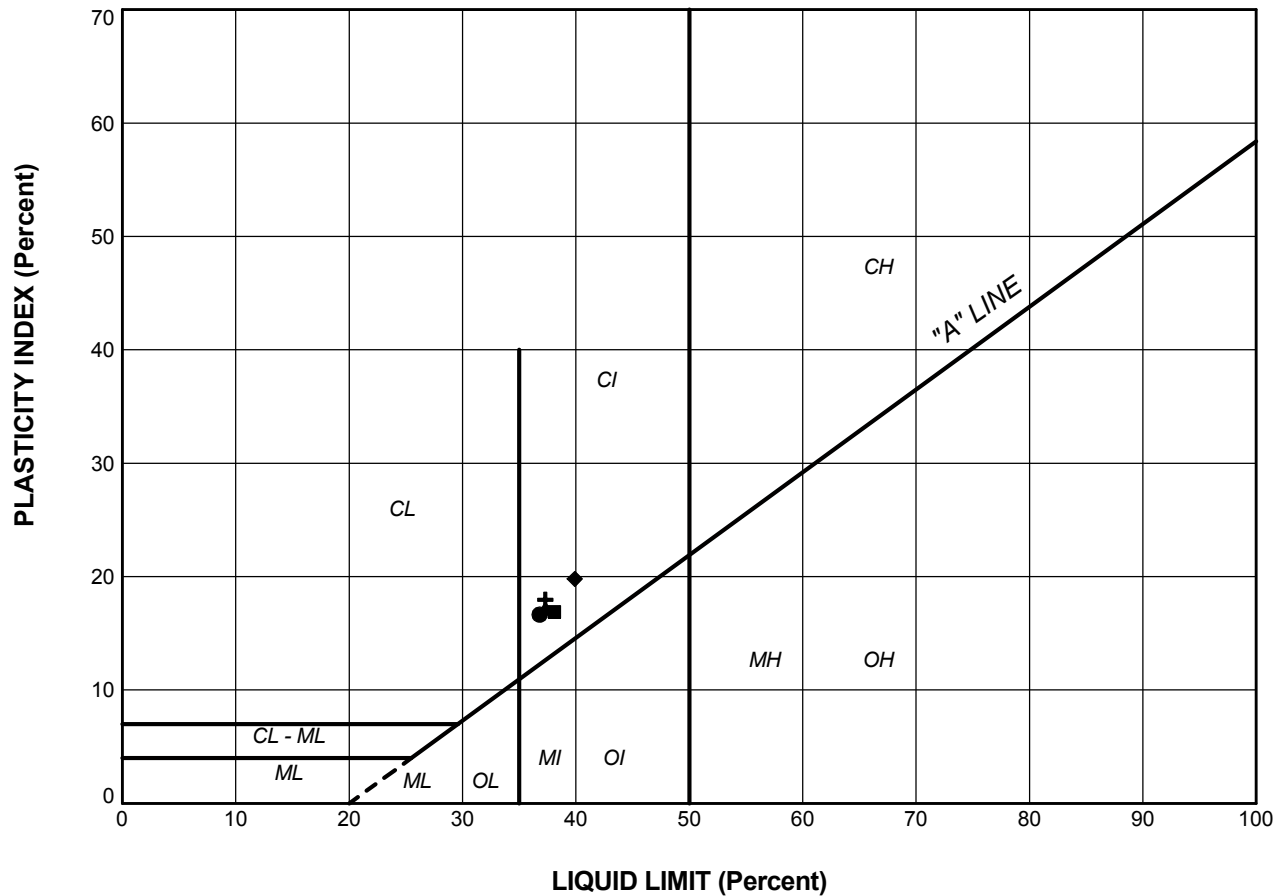


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	203	8	231.7


PROJECT				LAMBTON COUNTY ROAD 79 UNDERPASS HIGHWAY 402 & LAMBTON COUNTY ROAD 79 IMPROVEMENTS GWP 3158-06-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.		07-1130-128-4		FILE No.		0711301284-2-R010a2	
DRAWN		WDF		SCALE		N/A	
CHECK		Feb. 28/08		REV.			
 Golder Associates LONDON, ONTARIO				FIGURE A-2			

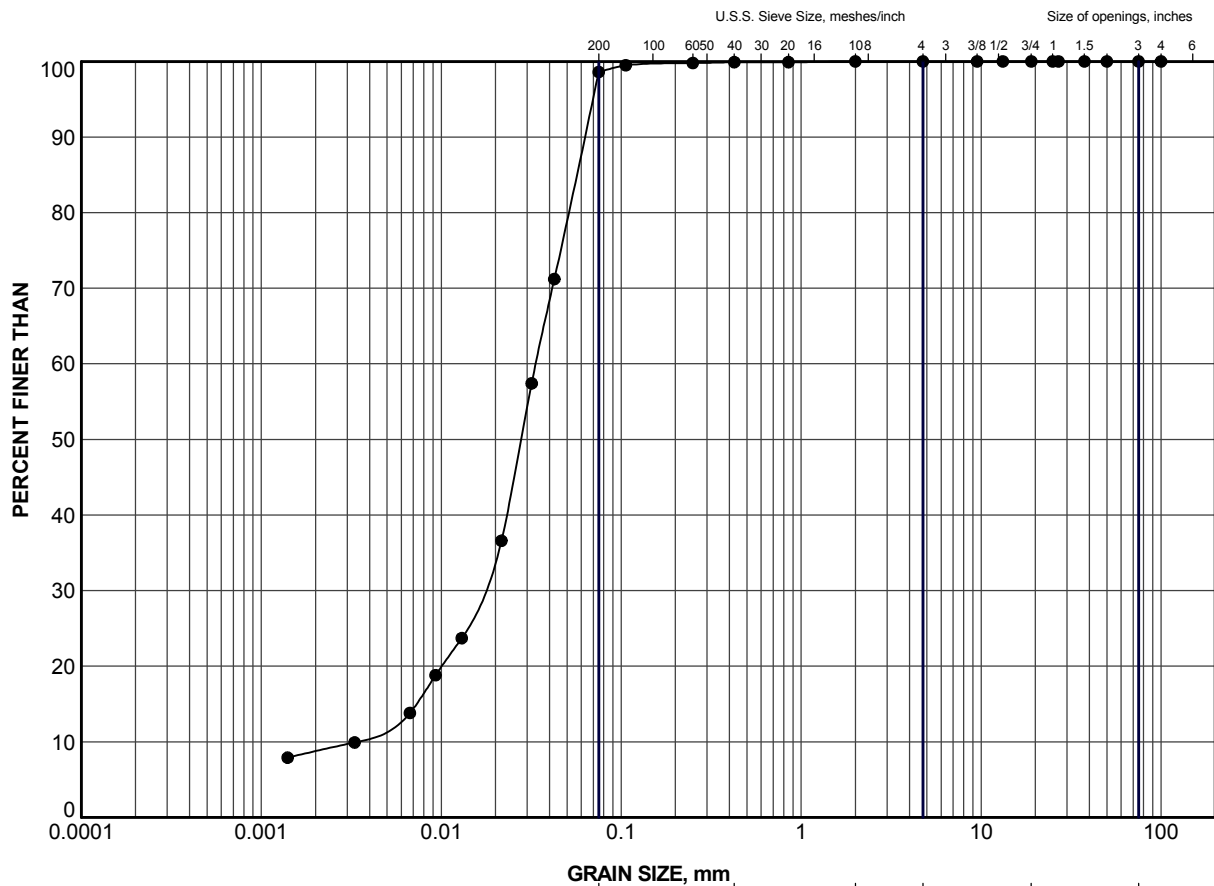
LDN_MTO_NEW_GLDR_LDN.GDT



LEGEND


SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
FILL, silty clay					
●	201	3	36.8	20.2	16.7
■	203	4	38.1	21.2	16.9
+	204	2	37.3	19.4	18.0
◆	204	4	39.9	20.1	19.8
SILTY CLAY					
▲	203	8	37.3	20.1	17.2

PROJECT			
LAMBTON COUNTY ROAD 79 UNDERPASS HIGHWAY 402 & LAMBTON COUNTY ROAD 79 IMPROVEMENTS GWP 3158-06-00			
TITLE			
PLASTICITY CHART			
PROJECT No. 07-1130-128-4		FILE No. 0711301284-2-R010a3	
DRAWN	WDF	Feb. 28/08	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	204	10	230.0

PROJECT				LAMBTON COUNTY ROAD 79 UNDERPASS HIGHWAY 402 & LAMBTON COUNTY ROAD 79 IMPROVEMENTS GWP 3158-06-00			
TITLE				GRAIN SIZE DISTRIBUTION SILT			
PROJECT No.		07-1130-128-4		FILE No.		0711301284-2-R010a4	
DRAWN		WDF		SCALE		N/A	
CHECK		Feb. 28/08		REV.			
 Golder Associates LONDON, ONTARIO				FIGURE A-4			

APPENDIX B

RECORDS OF PREVIOUS BOREHOLES
(GEOCRES REPORT NO. 40113-31)

RECORD OF BOREHOLE NO 1

WP 42-66-11 LOCATION Hwy. 79 Sta. 330 + 01 31' Rt. ORIGINATED BY AP
 DIST 1 HWY 402 BORING DATE May 19, 1971 COMPILED BY KW
 DATUM Geodetic BOREHOLE TYPE Bombardier Flight Auger and Cone CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH (232.29m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
752.1	Ground Level															
0.0			1	SS	21	760										
	Brown		2	SS	42											0 10 74 16
	Grey		3	SS	17											
	Clayey silt, traces		4	SS	19	750										0 1 67 32
	of sand		5	SS	26											0 3 73 24
	Occasional silt		6	SS	35	740										0 2 71 27
	and/or fine sand		7	SS	33											
	partings.		8	SS	30	730										
	Very Stiff to hard		9	SS	41											
			10	SS	55	720										0 3 67 30
			11	SS	59	710										
			12	SS	75	700										2 15 57 26
697.1			13	SS	100	690										
652.0	Weathered Shale															
66.0	End of Borehole															
	Probable Bedrock															

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2

WP 42-66-11 LOCATION Hwy. 79 Sta. 330 + 04 31' Lt. ORIGINATED BY AP
 DIST 1 HWY 402 BORING DATE May 25, 1971 COMPILED BY KW
 DATUM Geodetic BOREHOLE TYPE CME Flight Auger & Cone CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w w_p — w — w_L WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH (232.23m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES					
761.9	Ground Level									
0.0	Brown Grey Clayey silt to silty clay, traces of sand Occ. silt and/or fine sand partings. Very Stiff to hard		1	SS	21	760				0 1 62 37
			2	SS	45	750				
			3	SS	17					
			4	SS	31					
740.4					5					
21.5	End of borehole									0 5 69 26

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

WP 42-66-11 LOCATION Hwy 79 Sta. 328 + 61 32' Rt ORIGINATED BY AP
 DIST 1 HWY 402 BORING DATE May 20, 1971 COMPILED BY KW
 DATUM Geodetic BOREHOLE TYPE Bombardier Flight Auger and Cone CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P — W — W_L WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH (232.41m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100			
762.5	Ground Level					760								
			1	SS	30									
			2	SS	42									0 2 64 34
	Brown Grey silt, traces of sand & clay		3	SS	32									0 5 85 10
	Clayey silt, traces of sand. Occ. silt and/or sand partings		4	SS	31	750								1 1 58 40
741.0	Very Stiff to Hard		5	SS	19	740								
21.5	End of borehole													

RECORD OF BOREHOLE NO 4

WP 42-66-11 LOCATION: Hwy. 79 Sta. 328 + 66 34' Lt.
DIST 1 HWY 402 BORING DATE May 26, 1971
DATUM Geodetic BOREHOLE TYPE CME Flight Auger, NX Casing, BX Core and Cone

ORIGINATED BY AP
COMPILED BY KW
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH (232.17m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
761.7	Ground Level															
			1	SS	34	760										
	Brown Grey		2	SS	45											0 3 60 37
			3	SS	24											
	Silt, traces of sand & clay		4	SS	26	750										0 2 90 8
			5	SS	26											0 2 63 35
			6	SS	23	740										
			7	TW	PH											2 13 65 20
	Clayey silt, traces to some sand (increasing with depth) traces of gravel in lower regions (below elev. 740). Occ. silt and/or sand partings.		8	SS	38	730										
	Very Stiff to hard.		9	SS	41	720										3 18 53 26
			10	SS	31	710										
			11	SS	35	700										
691.2																
70.5	Bedrock					690										
686.5	Shaley Limestone		12	BX	75%											
75.2	End of Borehole															
						680										

RECORD OF BOREHOLE NO 5

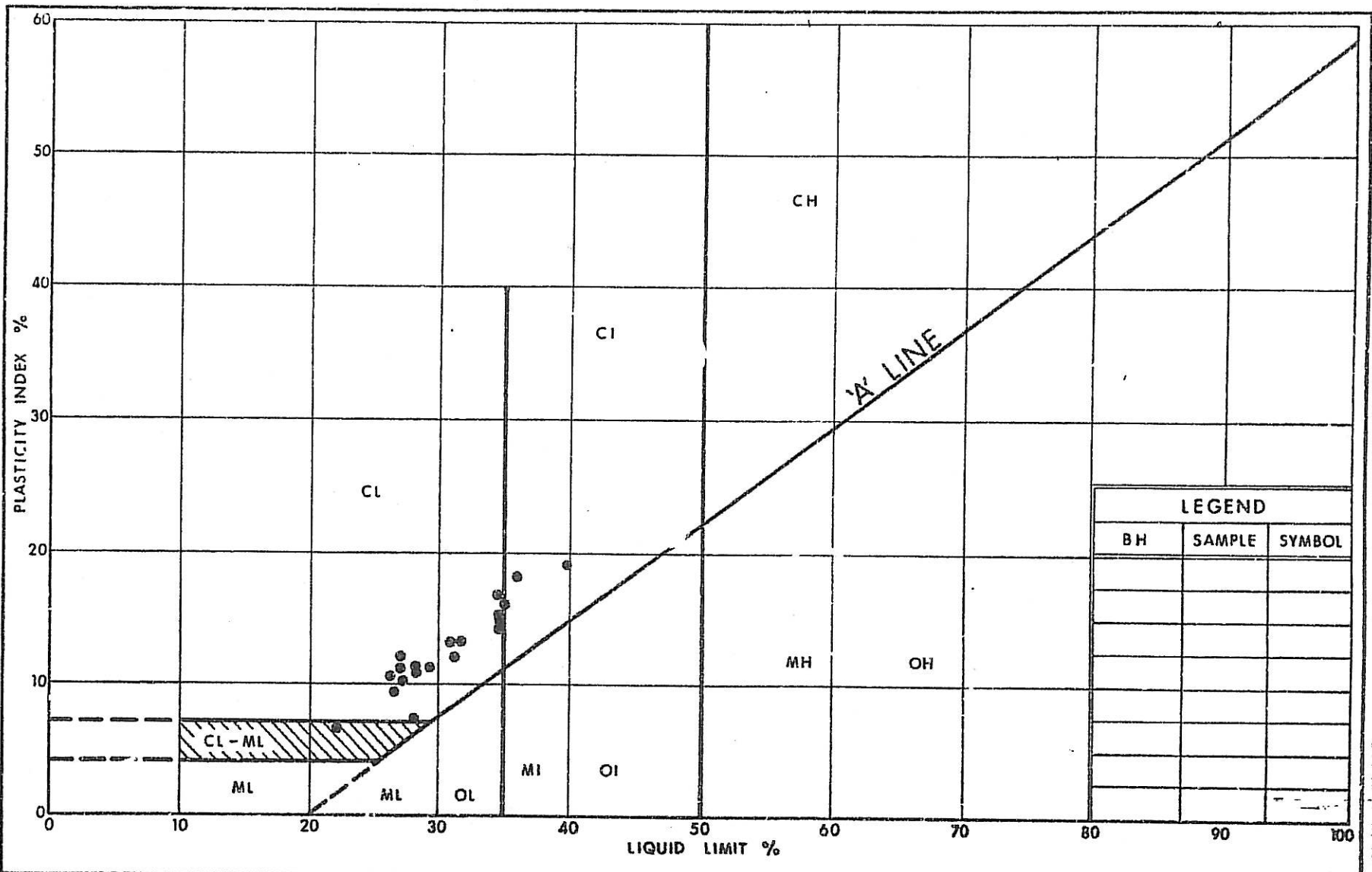
WP 42-66-11 LOCATION Hwy. 79 Sta. 327 + 30 32' Rt. ORIGINATED BY AP
DIST 1 HWY 402 BORING DATE May 21, 1971 COMPILED BY KW
DATUM Geodetic BOREHOLE TYPE Bombardier Flight Auger and Cone CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH (232.32m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	w_p	w	w_L		
762.2	Ground Level															
0.0			1	SS	39	760										
	Brown Grey		2	SS	62											
			3	SS	56											
	Silt, some sand traces of clay		4	SS	23	750										0 2 67 41
			5	SS	21											0 5 81 14
			6	TW	PH	740										0 1 64 35
	Clayey silt, traces to some sand (increasing with depth) traces of gravel in lower regions (below elev. 740). Occ. silt and/or sand partings.		7	SS	15											
			8	TW	PH	730										4 9 62 25
			9	SS	25	720										
	Very stiff to hard		10	SS	27	710										7 20 54 19
			11	SS	-	700										
591.7																
70.5	Weathered Shale		12	SS	10071"											
70.6	End of Borehole Probable Bedrock					690										

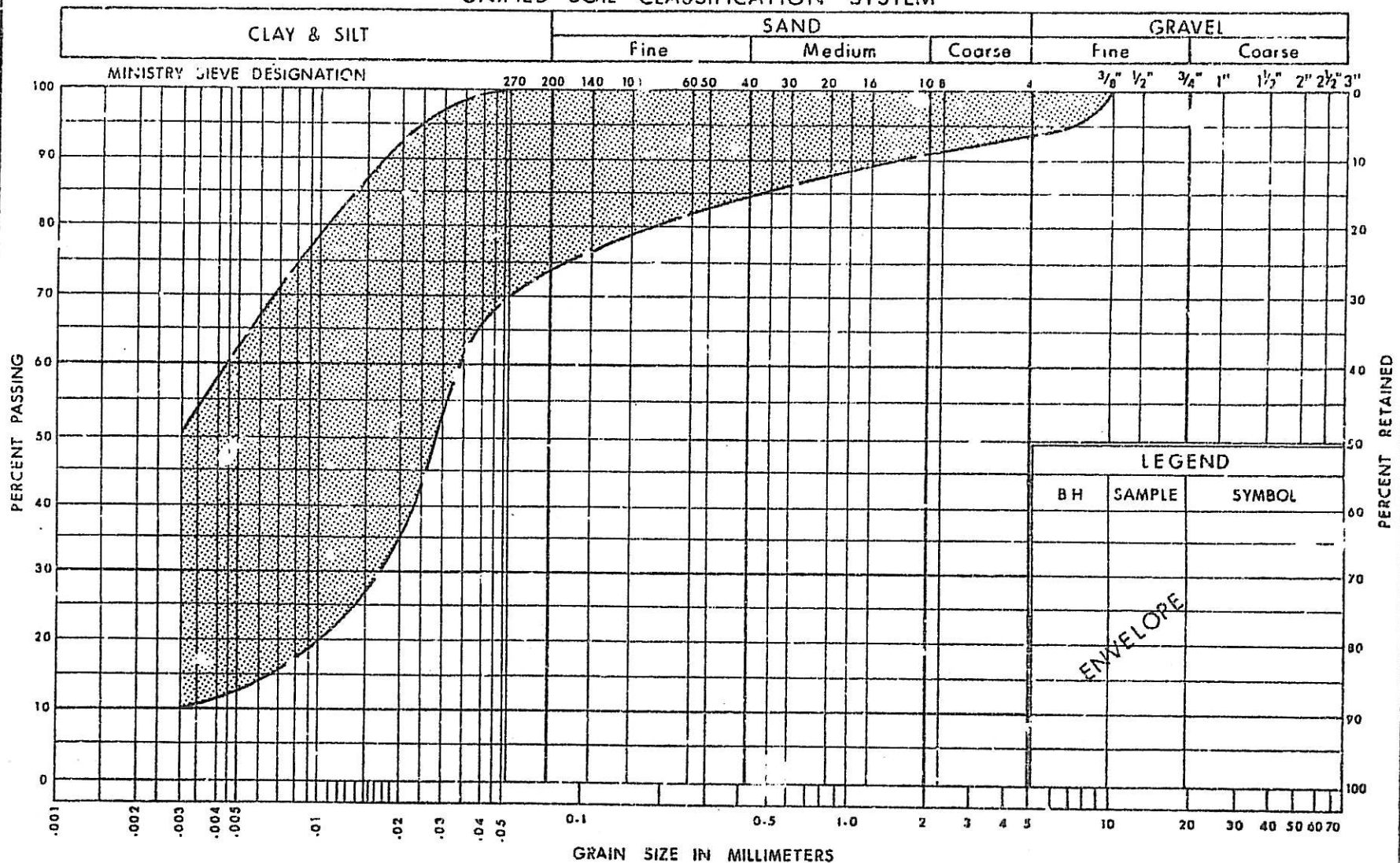
RECORD OF BOREHOLE NO 6

WP 42-66-11 LOCATION Hwy. 79 Sta. 327 + 27 32' LL. ORIGINATED BY AP
DIST 1 HWY 402 BORING DATE May 25, 1971 COMPILED BY KW
DATUM Geodetic BOREHOLE TYPE CME Flight Auger & Cone CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT: w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH (232.17m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100					w_p — w — w_L				
							SHEAR STRENGTH					WATER CONTENT %				
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					20 40 60				
761.7	Ground Level															
0.0	Clayey silt, traces of sand. Occ. silt and/or partings. Very stiff to hard		1	SS	25	760									0 2 58 40	
2			SS	29												
3			SS	23	750											
4			SS	17												
5			SS	23	740											
740.2	End of borehole															
21.5																



UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND

BH SAMPLE SYMBOL

ENVELOPE



Ministry of
Transportation and
Communications

Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
TRACE OF SAND, OCC. SILT &/OR SAND PARTINGS

FIG No 2

W P 42 - 66 - 11

APPENDIX C
PHOTOGRAPHS

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SITE PHOTOGRAPHS



Photo 1: South approach Lambton County Road 79 Underpass Structure.



Photo 2: Lambton County Road 79 Underpass Structure, looking east from shoulder of eastbound lanes. NS-E ramp at right of photo.

Golder Associates

SITE PHOTOGRAPHS



Photo 3: Tension crack along southwest approach fill.



Photo 4: Vertical cracking at abutment in north face of southwest approach fill.

SITE PHOTOGRAPHS



Photo 5: Tension cracks near toe of south face of northwest approach fill.



Photo 6: Loss of vegetation on south face of northwest approach fill.

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SITE PHOTOGRAPHS



Photo 7: Tension cracks near top of south face of northwest approach fill.



Photo 8: General view of soffit over eastbound lanes, looking towards the south abutment. Rip-rap slope protection is visible at bottom of photograph.

Golder Associates