



February 2013

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**Speedsville Road Underpass, Site 33-145  
Reconstruction and Widening of Highway 401  
From 0.5 km West of King Street  
Easterly to 0.5 km East of Hespeler Road - 5.5 KM  
GWP 4-00-00  
Ministry of Transportation - West Region**

**Submitted to:**

Mr. Henry Huotari, P. Eng., Senior Project Manager, Principal  
Delcan Corporation  
214-1069 Wellington Road South  
London, Ontario  
N6E 2H6

REPORT



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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

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**FOUNDATION INVESTIGATION AND DESIGN REPORT  
SPEEDSVILLE ROAD UNDERPASS, SITE 33-145**

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**PART A**

**FOUNDATION INVESTIGATION REPORT**

**SPEEDSVILLE ROAD UNDERPASS, SITE 33-145  
RECONSTRUCTION AND WIDENING OF HIGHWAY 401  
FROM 0.5 KM WEST OF KING STREET EASTERLY TO  
0.5 KM EAST OF HESPELER ROAD – 5.5 KM  
GWP 4-00-00  
MINISTRY OF TRANSPORTATION - WEST REGION**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Delcan Corporation (Delcan) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 4-00-00. The project involves the detail design for the reconstruction and widening of Highway 401 from 0.5 kilometres west of King Street (Waterloo Regional Road 8) easterly to 0.5 kilometres east of Hespeler Road (Waterloo Regional Road 24). This report addresses the replacement of the Speedsville Road Underpass (Site 33-145).

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples of the subsurface materials. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P0-1132-0056 dated July 23, 2010 and the revised scope letter 10-1132-0056-2000-L02 dated May 28, 2012. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated September 2010.

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



## **2.0 SITE DESCRIPTION**

The Speedsville Road Underpass (Site 33-145) is located in the City of Cambridge, Region of Waterloo, Ontario. It is approximately 400 metres west of the Speed River and 1.4 kilometres east of the Grand River Electric Railway line. The location of the project is shown on the Key Plan, Figure 1.

For the purposes of this report, Highway 401 and Speedsville Road are assumed to be oriented in an east-west direction and a north-south direction, respectively. This section of Highway 401 is currently a six lane divided highway with a median wall. The existing underpass is a continuous deck slab structure which was erected in 1960. The existing embankments are approximately 8 to 9 metres high. The road surface at the grade of Speedsville Road is at approximately elevation 285.5 metres. The Highway 401 grade near the centreline of Speedsville Road is near elevation 279.5 metres.

The lands south and east of the structure are primarily used for recreational purposes. A Provincially Significant Wetland (PSW), including property administrated by the Grand River Conservation Authority (GRCA), is located east of Speedsville Road. There are some rural residential, agricultural and industrial/commercial uses of properties located on the north side of Highway 401. Site Photographs are provided in Appendix E.

## **2.1 Site Geology**

This project lies within the physiographic region of southwestern Ontario known as the Waterloo Hills which primarily comprises sandy glacial till ridges or glacial kame moraines with outwash sands in the lower areas. The physiographic mapping indicates that the Speedsville Road underpass site is situated in a former glaciofluvial spillway area<sup>1</sup>.

The quaternary geology mapping indicates that the surficial materials consist of stream deposits containing gravel, sand, silt and clay. Immediately west of the structure is a northeast-southwest trending rock outcrop with a water course. The bedrock reportedly consists of shale and dolomite.<sup>2</sup> The underlying bedrock surface is typically found between the elevations of 278 and 282 metres based on geologic mapping.<sup>3</sup> The rock formation is mapped and described as cream and brown, fine to medium crystalline dolomite of the Guelph Formation.<sup>4</sup>

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

<sup>2</sup> Karrow, P.F., 1987: Quaternary Geology of the Cambridge Area, Southern Ontario. Ontario Geological Survey, Map 2508, scale 1:50,000.

<sup>3</sup> Ontario Department of Mines, 1960: Bedrock Topography, Galt Area, Southern Ontario. Map 2030, scale 1:50,000

<sup>4</sup> Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.



### 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between April 26 and June 3, 2012, during which time 8 boreholes, numbered 301 to 308, were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths:

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
301	4 808 494	235 958	285.14	14.97
302	4 808 577	235 940	285.56	12.10
303	4 808 573	235 934	285.58	12.13
304	4 808 493	235 952	285.19	13.50
305	4 808 478	235 954	284.60	9.21
306	4 808 596	235 936	285.11	7.53
307	4 808 537	235 959	279.49	9.39
308	4 808 536	235 937	279.42	9.75

The investigation was carried out using truck mounted drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.75 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of ASTM D 1586. The bedrock in boreholes 301 to 304, 307 and 308 was cored using NQ-sized rock coring equipment. Core lengths greater than the minimum MTO requirement of 3.0 metres were obtained due to the poor quality of the near surface bedrock. The boreholes were terminated between 7.5 and 15.0 metres below the existing pavement or ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder staff members who also located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Selected samples of rock core were forwarded to our Mississauga laboratory for unconfined compression 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 testing are shown on the Record of Borehole sheets and in Appendix A.

The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.



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In addition, information from the original subsurface investigation for the existing structure was incorporated into this report. Data from boreholes 1 to 4 from Geocres Report No. 40P8-76 entitled "Soils Report for Highway 401 – Waterloo Township #6 Crossing" dated September 24, 1958 was used to supplement the current data.

The Record of Borehole sheets for previous boreholes are presented in Appendix B in their original format. The table below summarizes the locations, ground surface elevations and depths of the previous boreholes.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
1	4 808 550	235 949	277.29	5.03
2	4 808 515	235 942	277.86	4.65
3	4 808 550	235 935	277.52	4.88
4	4 808 516	235 956	277.12	5.69

Boreholes 1 and 3 were drilled in the Highway 401 eastbound lanes within the structure footprint and boreholes 2 and 4 were drilled in the westbound lanes. It should be noted that the boreholes were drilled in 1958 and pre-date the existing overpass structure and Highway 401. The locations of the previous boreholes are shown in plan on Drawing 1; however, these locations should be considered approximate since the locations were referenced to imperial chainages and offsets rather than metric MTM coordinates.





## **4.0 SUBSURFACE CONDITIONS**

### **4.1 Site Stratigraphy**

The detailed subsurface soil, rock 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 provided on the attached Record of Borehole sheets following the text of this report and in Appendix A. 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 and rock 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 generally encountered fill materials overlying buried topsoil, peat, sands and limestone bedrock.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawings 1 to 3. Detailed description of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and are summarized in the following sections.

#### **4.1.1 Topsoil**

Layers of buried topsoil 240 to 760 millimetres thick were encountered at the base of the Speedsville Road embankment fill in boreholes 301, 304 and 305 from elevation 276.5 to 277.0 metres and in borehole 306, at elevation 278.4 metres. The sandy to silty topsoil exhibited SPT N<sup>5</sup> values ranging between 11 and 25 blows per 0.3 metres but generally less than 13 blows per 0.3 metres. The water content of the topsoil samples varied from 21 to 22 per cent.

A layer of topsoil, described in the previous investigation as a sandy loam, approximately 305 to 610 millimetres in thickness, was encountered at the ground surface in boreholes 1 to 4 (40P8-76) which are located within the travelled portion of Highway 401. The topsoil layer is presumed to have been removed during construction of Highway 401 during the early 1960s. The reported ground surface at the locations of the 1958 boreholes ranged from elevation 277.1 to 277.9 metres.

Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

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<sup>5</sup> According to ASTM D-1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres.



#### **4.1.2 Pavement Structure**

Asphaltic concrete pavement was encountered at the ground surface in boreholes 301 to 308. The asphaltic concrete was about 150 to 230 millimetres thick at the borehole locations.

Pavement granular base or subbase materials were encountered beneath the asphaltic concrete in boreholes 301 to 308 from elevations 279.2 to 285.4 metres. The granular base or subbase materials were about 220 to 250 millimetres thick.

#### **4.1.3 Fill**

Boreholes 301 to 306 intercepted embankment fill beneath the pavement structure below elevation 284.2 to 285.2 metres. The embankment fill contained cobbles and predominantly consisted of layers of various granular materials: sand and gravel, sand, silty sand, silty sand and gravel and sandy silt. A 0.3 metre thick layer of clayey silt fill was encountered at elevation 281.0 metres in borehole 306. The embankment fill materials ranged in thickness from 6.3 to 8.2 metres at the borehole locations.

At the Highway 401 grade, the pavement structure was underlain by granular fill with cobbles comprising sand and gravel, sand and sandy silt below elevations 279.0 metres in borehole 307 and 278.0 metres in borehole 308. The fill was found to be 1.4 and 1.5 metres thick in boreholes 307 and 308, respectively.

Standard penetration testing N values of 4 blows per 0.3 metres to 100 blows per 50 millimetres but generally less than 30 blows per 0.3 metres were recorded in the embankment fill layers. The fill samples had water contents of 3 to 15 per cent. Grain size distribution curves for samples of the fill recovered from the standard penetration testing are provided on Figures A-1 and A-2. In general, the samples from the lower portion of the embankments had higher silt and clay contents.

#### **4.1.4 Sand and Gravel**

Layers of compact to very dense sand and gravel were encountered beneath the fill in boreholes 302 and 303. The sand and gravel was encountered at elevations 278.1 and 278.9 metres and was about 0.8 and 1.2 metres thick. The sand and gravel had measured N values of 28 blows per 0.3 metres to 100 blows per 200 millimetres.

#### **4.1.5 Sand**

Layers of compact to very dense sand were encountered beneath the fill in borehole 307 at elevation 277.5 metres, and beneath the buried topsoil in boreholes 301 and 306 at elevation 275.9 and 277.9 metres



respectively. Where fully penetrated, the sand layers were about 0.6 to 1.2 metres thick. Borehole 306 was terminated on inferred bedrock after drilling through 0.3 metres of sand.

The sand had a measured N value of 27 blows per 0.3 metres. Water contents of 15 to 31 per cent were measured in selected samples. The higher water contents were measured in samples from borehole 301 that contained topsoil. A grain size distribution curve for a sample of the sand recovered from the standard penetration testing is provided on Figure A-3.

Layers of sand were encountered beneath the topsoil in boreholes 1 to 4 (40P8-76). These layers were encountered at elevation 276.8 to 277.3 metres and were about 0.3 to 0.8 metres thick. The sand in boreholes 1 and 3 (40P8-76) contained organic matter. The sand in boreholes 2 and 4 (40P8-76) contained rock fragments.

#### **4.1.6 Clayey Silt**

A layer of stiff clayey silt was encountered beneath the buried topsoil in borehole 304 at elevation 276.2 metres. The clayey silt was about 0.4 metres thick and had a water content of 26 per cent.

#### **4.1.7 Silty Sand**

The fill in borehole 308 was underlain by a 0.7 metre thick layer of silty sand from elevation 277.5 metres. The silty sand was very dense and had a measured N value greater than 100 blows per 0.3 metres. The measured water content was 12 per cent. The results of a grain size analysis carried out on the sample of silty sand are presented on Figure A-4.

#### **4.1.8 Peat**

A layer of peat was encountered beneath the buried topsoil in borehole 305, beneath the clayey silt in borehole 304 and beneath the sand in borehole 301. The peat was encountered between elevations 275.8 and 276.4 metres and was about 0.3 to 0.9 metres thick. The peat had measured N values of 9 and 10 blows per 0.3 metres and water contents of 108 to 138 per cent.

#### **4.1.9 Bedrock**

Dolomitic limestone to dolostone bedrock of the Guelph Formation was encountered beneath the peat in boreholes 301 and 304, beneath the sand and gravel in boreholes 302 and 303 and beneath the sand in borehole 307 and 308. The bedrock contained millimetre-scale horizontal interbeds of shale. The bedrock



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surface was inferred beneath the peat in borehole 305 and beneath the sand in borehole 306 due to auger refusal. The rock surface slopes generally southwards from approximate elevation 277.6 metres at the north abutment to 275.2 metres at the south abutment.

The following table summarizes the bedrock surface depths and elevations (proved or inferred) as encountered at the borehole locations.

Location	Borehole	Ground Surface Elevation (m)	Encountered Bedrock Surface	
			Depth (m)	Elevation (m)
North Approach	306*	285.11	7.53	277.58
North Abutment	302	285.56	7.92	277.64
North Abutment	303	285.58	8.23	277.35
Pier	307	279.49	3.14	276.35
Pier	308	279.42	2.62	276.80
South Abutment	301	285.14	9.91	275.23
South Abutment	304	285.19	9.66	275.53
South Approach	305*	284.60	9.14	275.46

\*Bedrock surface was inferred based on auger refusal.

Samples of rock core were obtained in boreholes 301 to 304, 307 and 308 using an NQ-size core barrel. The bedrock was explored for about 3.8 and 7.1 metres before terminating the boreholes. The Rock Quality Designation (RQD), Total Core Recovery (TCR) and Solid Core Recovery (SCR) for each rock core run are summarized in the following table.

Borehole	Elevation (m)		RQD (%)	TCR (%)	SCR (%)
	From	To			
301	274.7	273.2	23	97	93
	273.2	271.7	19	95	95
	271.7	270.2	44	100	100
302	277.2	276.5	0	96	96
	276.5	275.0	27	94	84
	275.0	273.5	38	100	100
303	276.9	276.5	0	90	90
	276.5	275.0	19	93	88
	275.0	273.4	38	98	95
304	275.3	274.7	0	84	74
	274.7	273.2	20	100	100
	273.2	271.7	8	86	75



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Borehole	Elevation (m)		RQD (%)	TCR (%)	SCR (%)
	From	To			
307	276.4	276.2	0	100	100
	276.2	274.7	7	90	82
	274.7	273.2	39	98	98
	273.2	271.6	14	84	82
	271.6	270.1	41	99	94
308	276.5	275.0	7	70	62
	275.0	273.5	13	91	89
	273.5	271.9	7	74	67
	271.9	270.4	26	96	96
	270.4	269.7	51	86	86
Average			20	92	88

The RQD varies between 0 per cent and 51 per cent with an average of 20 per cent indicating very poor to fair quality. The upper 2 to 3 metres is highly fractured and there is a general trend of improving rock quality with increased penetration below the rock surface as indicated by the table below.

Run	Average Thickness (m)	Elevation (m)		Average RQD (%)	Average TCR (%)	Average SCR (%)
		Average Top of Run	Average Base of Run			
1	0.8	276.2	275.4	5	90	86
2	1.5	275.4	273.9	18	94	90
3	1.5	273.9	272.3	29	93	89
4 <sup>#</sup>	1.5	272.6	271.0	20	90	89
5 <sup>#</sup>	1.1	271.0	269.9	46	93	90

<sup>#</sup> Three runs of coring were carried out in boreholes 301 to 304 and five runs in boreholes 307 and 308.

The core samples obtained from the 2012 investigation were noted to have rubbly and calcite-filled vuggy zones and voids. The dolomitic limestone to dolostone is generally massive and thinly laminated with horizontal shale interbeds. The texture varies from fine crystalline to fossiliferous. Rubbly zones were observed between elevations 276.4 and 276.0 metres in boreholes 302 and 303, near elevation 271.7 metres in borehole 304 and between elevations 275.7 and 275.3 metres in borehole 307. Calcite filled voids and/or vugs were observed below elevation 274.6 metres in boreholes 302, 303 and 308 and near 275.3 metres in borehole 307. The strength of the bedrock varies between R2 to R6 according to the Canadian Foundation Engineering Manual (2006) rock strength classification system. The strengths of two bedrock samples were determined by carrying out unconfined compression tests on selected rock core. The intact rock is strong to very strong based on measured unconfined compressive strengths (UCS) of 64 and 111 megapascals (MPa). The rock test data are shown on the Record of Borehole Sheets and are presented in Appendix B. Photographs of the rock core



recovered are shown in Appendix C. Although the intact rock is classified as strong to very strong, the behaviour of the rock mass will also be influenced by fissures, joints and other discontinuities.

The condition of rock cores from the 2012 investigation was found to be similar to that described for the 1958 investigation. The 1958 investigation was carried out presumably using AXT or BX sized equipment and recoveries of 59 to 100 per cent were reported (40P8-76). The bedrock was described to be fissured and faulted “with evidence of deep water penetration”. Bedrock was encountered beneath the sand in boreholes 1 to 4 (40P8-76) from elevation 276.2 to 277.0 metres. The unconfined compressive strength (UCS) of the bedrock, based on tests on six samples was reported to range from 50.5 to 113.4 megapascals but generally 67 megapascals or less (40P8-76). Eliminating the single sample which had a UCS greater than 67 megapascals, the average unconfined compressive strength of intact samples of the bedrock was estimated to be 54 megapascals indicating a medium strong to strong rock.

A test pit was excavated at the time of the original investigation to examine the bedrock surface. The bedrock was found to be layered with 25 to 50 millimetre deep cracks and easily broken up by a shovel. The type of shovel and depth to which the shovel could excavate the surficial bedrock was not noted.

## 4.2 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling. Groundwater was encountered in boreholes 301, 305, 306 and 307 at depths of 2.7 to 9.3 metres or between elevations of 275.8 and 277.9 metres. Boreholes 302, 303, 304 and 308 remained dry during drilling. Groundwater was encountered in previous boreholes 1, 2, 3 and 4 (40P8-76) between elevations 276.5 and 276.7 metres or at depths of 0.6 to 1.4 metres. A summary of the encountered and measured groundwater levels is provided in the table below. The water level in the adjacent reach of the Speed River during the 1958 investigation was between elevations 275.3 to 275.7 metres. The reported water level elevation in the Speed River at a Water Survey of Canada gauge located approximately 1.3 kilometres upstream of Highway 401 was 280.49 metres on October 2, 2012.<sup>6</sup>

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
301	285.14	9.3	275.8
302	285.56	Dry to 12.1	Dry to 273.5
303	285.58	Dry to 12.1	Dry to 273.5
304	285.19	Dry to 13.5	Dry to 271.7
305	284.60	7.5	277.1
306	285.11	7.2	277.9

<sup>6</sup> Grand River Conservation Authority, 2012: Water Survey of Canada Gauge Survey Report < <http://www.grandriver.ca/index/document.cfm?Sec=2&Sub1=6&Sub2=18> > accessed October 2, 2012.



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Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
307	279.49	2.7	276.8
308	279.42	Dry to 9.8	Dry to 269.7
1 (40P8-76)	277.29	0.8	276.5
2 (40P8-76)	277.86	1.4	276.5
3 (40P8-76)	277.52	0.8	276.7
4 (40P8-76)	277.12	0.6	276.5

The above-noted water levels are not considered to be representative of the long-term, stabilized groundwater conditions as the readings were taken only during the relatively short duration of drilling. Based on the measured and encountered groundwater levels and the soil colour change from brown to grey, the inferred groundwater level is at elevation 277 metres. 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. Michael Arthur and Mr. Brett Thorner 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. The unconfined compression tests were conducted in Golder's Mississauga laboratory under the supervision of Dr. J.Paul Dittrich, P.Eng. The Mississauga laboratory is a MTO registered laboratory in the Specialty of Soil and Rock including Testing for Foundation Engineering - Low and High Complexity.

This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, Mr. Storer J. Boone, Ph.D., 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.**

**ORIGINAL SIGNED**

Dirka U. Prout, P.Eng.  
Project Engineer

**ORIGINAL SIGNED**

Storer J. Boone, Ph.D., P.Eng.  
Associate

**ORIGINAL SIGNED**

Fintan J. Heffernan, P.Eng.  
MTO Designated Contact

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**PART B**

**FOUNDATION DESIGN REPORT**

**SPEEDSVILLE ROAD UNDERPASS, SITE 33-145  
RECONSTRUCTION AND WIDENING OF HIGHWAY 401  
FROM 0.5 KM WEST OF KING STREET EASTERLY TO  
0.5 KM EAST OF HESPELER ROAD – 5.5 KM  
GWP 4-00-00  
MINISTRY OF TRANSPORTATION - WEST 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 replacement of the existing Speedsville Road (Waterloo Regional Road 38) Underpass, Site 33-145. The recommendations are based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

Based on the preliminary design information provided by Delcan, the replacement structure will be 13.1 metres wide, 84 metres long and will be erected at the same location as the existing structure. The deck will be designed with two steel box girders with provision for future 4.4 metre wide symmetrical widening by adding an additional girder to each side of the structure. The median pier will consist of three columns. Consideration is being given to supporting the central pier on drilled shafts (caissons). The existing embankments will be modified to provide room for a speed change lane and an additional lane in each direction. The profile grade of Highway 401 at the Speedsville Road underpass structure will remain relatively unchanged. The profile grades of Speedsville Road will be increased by approximately 1.0 metre at the south abutment and 2.0 metres at the north abutment. The resulting grade at the Speedsville Road grade will vary from elevation 287.6 metres at the north abutment, 287.4 metres at the central pier and 286.3 metres at the south abutment. The existing embankment will be symmetrically widened approximately 1.4 metres in the bridge approach area.

### **6.2 Bridge Foundations**

The subsurface soil and rock conditions typically consist of the existing pavement structure and fill materials to approximate elevation 277.5 metres. The embankment fill is underlain by buried topsoil layers 0.3 to 0.8 metres thick at the approach embankments. With the exception of the boreholes completed at the south abutment, the buried topsoil and/or fill was underlain by sand or sand and gravel to the bedrock surface encountered between elevation 276.4 and 277.6 metres. At the south approach embankment, the buried topsoil was underlain in sequence by sand or clayey silt to approximate elevation 276 metres then peat to the bedrock surface which was encountered between elevation 275.2 and 275.5 metres. The groundwater level has been inferred to be at or near approximately elevation 277 metres.

The median pier of the replacement structure could be founded on strip/spread footings or drilled shafts (caissons). Spread footings are the preferred foundation engineering alternative for the pier foundations from a geotechnical viewpoint since bedrock is near the ground surface. The depth of the existing embankment fill and the proposed use of integral abutments preclude the use of shallow foundations for the abutments. Steel tube



piles do not provide the flexibility required for integral abutments, therefore, steel H-piles are the preferred technical alternative for support of the abutments. If conventional abutments are designed, the abutments can be supported on concrete filled steel tube piles or steel H-piles driven to bedrock. Recommendations for each of these foundation systems are provided in subsequent report sections below.

A comparison of foundation alternatives is presented in Table I. The costs provided are estimates meant to provide an order of magnitude comparison for the alternatives for foundation engineering purposes and should not be considered to be indicative of actual construction costs.

### 6.2.1 Shallow Foundations – Median Pier

The three piers of the existing structure are supported by concrete strip footings founded in the bedrock at elevation 274.9 metres according to the Department of Highways Ontario (DHO) Drawing D 4200-1 entitled “General Arrangement Waterloo Township Bridge No. 6”, dated February 13, 1959. The bedrock surface was encountered near elevation 276.5 metres at depths of 2.6 and 3.1 metres below the Highway 401 surface. The founding elevation for the existing footings indicates that the top 1.5 metres of rock was removed to achieve a sound bearing surface. The new piers for the replacement bridge can be founded on spread/strip footings bearing on the bedrock at or below elevation 275 metres. It is preferred that the footings are placed at or below elevation 275 metres since the bedrock above this elevation is of very poor quality. Based on the quality of the rock areas obtained at boreholes 307 and 308, there is a zone of relatively poor quality bedrock based on the RQD, total and solid core recoveries between elevation 272 and 273.5 metres. As such, excavation below elevation 274.5 metres should be avoided. A factored geotechnical resistance at Ultimate Limit States (ULS) of 1,200 kilopascals can be used for design. A geotechnical reaction at Serviceability Limit States (SLS) is not provided since the bedrock is considered to be effectively unyielding (i.e., settlement values typically used to define the serviceability limit should not occur).

The condition of the bearing surface should be assessed by a Quality Verification Engineer (QVE) experienced in geotechnical engineering after removal of the existing foundations and excavation to the design footing elevation. It may be necessary to carry out additional probings to ascertain the elevation of competent bedrock if inspection reveals that the bearing surface is unsuitable at the design footing elevation. Detailed comments pertaining to construction of the footings on bedrock are presented in Section 6.7.1.

### *Resistance to Lateral Forces*

Resistance to lateral forces/sliding between the concrete spread/strip footings and the bedrock should be calculated in accordance with Section 6.7.5 of the CHBDC. Assuming that the surface of the bedrock is adequately prepared during excavation, the following angle of friction between the concrete and the founding bedrock and corresponding unfactored coefficient of friction,  $\tan \delta$ , may be used:

Footings on dolomitic limestone to limestone	angle of friction	38°
	$\tan \delta$	0.78



### **6.2.2 Deep Foundations – Median Pier**

It is understood that cast-in-place concrete drilled shafts (caissons) are being considered for support of the median pier. In this case, the caissons would likely be connected to the piers as a single unit without a pile cap. It has been assumed that large diameter caissons, at least the same size as the columns for the median pier, or with a minimum diameter of 1.2 metres will be used. The use of deep foundations may not be economical for this site since bedrock is 3 metres below the Highway 401 grade and excavation to this level would be required to remove the existing median pier footing.

Special precautions will need to be taken prior to and during construction of the caissons to address the relatively shallow groundwater levels and fractured bedrock. Detailed comments pertaining to construction of the caissons socketed into bedrock are presented in Section 6.7.2.

#### ***Geotechnical Axial Resistance – Caissons***

Assuming an estimated tip elevation of about 271.5 metres to result in caissons being socketed approximately 3 to 4 diameters into the bedrock, the end-bearing resistance for design may be based on a factored axial geotechnical resistance at ULS of 5,000 kilopascals. The following factored axial geotechnical resistances for two sizes of caissons are given in the following table:

<b>Caisson Diameter (m)</b>	<b>Factored Axial Resistance at ULS (kN)</b>
1.2	5,500
1.5	8,800

An SLS value is not provided since the SLS resistance for 25 millimetres of settlement is greater than the factored axial geotechnical resistance at ULS.

#### ***Downdrag Load (Negative Skin Friction)***

Downdrag loads are not applicable for piles (caissons) supporting the median pier since the grade change at the Highway 401 level is negligible, the depth to bedrock is shallow and the overburden materials are primarily granular in nature.

### **6.2.3 Deep Foundations – Abutments**

The abutments may be founded on steel tube piles or steel H-piles driven into the underlying bedrock. Steel H-piles are suitable for both conventional and integral abutments whereas steel tube piles cannot be used for



integral abutments due their stiffness. The existing embankment fill contains cobbles. In addition, the upper surface of the bedrock is highly fractured. Construction-related recommendations for installation of piles at the abutments are discussed in Section 6.7.3.

### ***Geotechnical Axial Resistance – Driven Steel H-piles***

End bearing HP 310 x 110 piles driven to refusal into the bedrock may be designed using the geotechnical resistances noted in the following table. The cut-off elevations have been inferred from the General Arrangement drawing.

<b>Pile Location</b>	<b>Assumed Cut-off Elevation (m)</b>	<b>Proposed Tip Elevation (m)</b>	<b>Proposed Pile Length (m)</b>	<b>Factored Geotechnical Resistance at ULS (kN)</b>
North Abutment	283.0	276.5	6.5	1,500
South Abutment	282.3	274.7	7.0	1,500

A SLS value is not provided since the geotechnical reaction at SLS corresponding to 25 millimetres of settlement is greater than the ULS value. Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement. A Non-Standard Special Provision (NSSP) for CSP Integral Abutments detailing the sand gradation should be included in the Contract Documents.

The steel H-piles should be installed and monitored in accordance with Ontario Provincial Standard Drawing (OPSD) 3000.100, OPSD 3000.150 and OPSS 903. The piles should be equipped with Titus Bearing points or approved equivalent. Once bedrock is reached, the hammer energy should be reduced to 10 per cent of maximum and the pile struck in sets of 20 blows until no further penetration is observed. The hammer energy is then to be increased by 10 per cent and the procedure repeated incrementally to 100 per cent of the maximum hammer energy<sup>7</sup>. The maximum ultimate resistance of two times the factored ULS value shown in the above table should be noted on the foundation drawing. The appropriate pile note should be added to the foundation drawing.

### ***Geotechnical Axial Resistance – Driven Steel Tube Piles***

End bearing, concrete-filled steel tube piles with a 324 millimetre outer diameter (O.D.) and a 9.5 millimetres wall thickness may be driven closed ended to refusal in the bedrock. The cut-off elevations have been inferred from the General Arrangement drawing. The following table summarizes the proposed tip elevations, pile lengths, and factored geotechnical resistances at ULS.

<sup>7</sup> Ministry of Transportation, Ontario 2011: Structural Manual. Provincial Highway Management Division, Highway Standards Branch, Bridge Office.



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Pile Location	Assumed Cut-off Elevation (m)	Proposed Tip Elevation (m)	Proposed Pile Length (m)	Factored Geotechnical Resistance at ULS (kN)
North Abutment	283.0	277.0	6.0	1,300
South Abutment	282.3	275.0	7.3	1,300

The steel tube piles should be installed and monitored in accordance with OPSD 3001.100, OPSD 3001.150 and OPSS 903. The piles should be equipped with Type II driving shoes in accordance with OPSD 3001.100. The appropriate pile note should be added to the foundation drawing.

The maximum ultimate resistance of two times the factored ULS value shown in the above table should be noted on the foundation drawing.

### ***Frost Protection***

If conventional abutments are used, the pile caps should be provided with a frost cover of 1.4 metres of soil or thermal equivalent.

### ***Downdrag Load (Negative Skin Friction)***

Grade increases on the order of 0.5 to 1.0 metres have been proposed at the approach embankments in conjunction with the bridge replacement. Considering the relatively small grade changes and the predominantly granular embankment fill and native overburden materials and the relatively shallow depth to bedrock, negligible negative skin friction is expected to develop on the new piles at both abutments.

## **6.2.4 Resistance to Lateral Loads**

Lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading. The stratigraphy presented in the table below has been simplified for the purposes of this report.

The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

$k_h$  = coefficient of horizontal subgrade reaction (MPa/m) =  $n_h (z/d)$  for cohesionless soils

$d$  = pile width or diameter (m)

$n_h$  = constant of horizontal subgrade reaction (MPa/m)

$z$  = depth below ground surface grade (m)



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The range in values reflects the variability in subsurface conditions as well as the two extremes of design: the requirement for flexibility if integral abutments are selected, and the requirement for lateral support in the cases of non-integral abutments or pier foundations.

Soil Type	North Abutment		South Abutment	
	Elevation (m)	$n_h$ (MPa/m)	Elevation (m)	$n_h$ (MPa/m)
Granular backfill around piles and CSPs (for integral abutments)	Where applicable	8 - 14	Where applicable	5 - 16
Existing granular fill materials - Loose to very dense sand and gravel to sandy silt	285 to 278	2 - 15	285 to 277	4 - 15
Native granular soils - Compact to very dense sand or sand and gravel	278 to 277	8 - 15	277 to 275	2 - 3

Based on the current design information provided by Delcan, it is anticipated that integral abutment H-piles will be between 6.5 and 7.0 metres in length. It is considered that H-piles greater than 5.0 metres in length are not required to be augured in to rock in order to achieve fixity. As the design process proceeds Golder will continue to work with the design engineer and may provide recommendations regarding pile fixity if the pile design lengths are found to be shorter than 5 metres.

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, <math>d</math> = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor <math>R</math></i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The lateral resistance for single piles for HP 310 x 110 and 324 millimetre diameter, 9.5 millimetre wall thickness tube piles, 1.2 and 1.5 metre diameter caissons are summarized in the following table:



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Pile Type	Lateral Resistance	
	Factored ULS (kPa)	SLS (kPa)
HP 310 x 110 (driven to refusal into bedrock)		
- Weak axis bending (Integral abutments)	70	40
- Strong axis bending (all other abutment types)	110	60
324 mm O.D. x 9.5 mm tube (driven to refusal into bedrock)	80	50
1.2 m O.D. caisson (socketed in bedrock)	185	100
1.5 m O.D. caisson (socketed in bedrock)	220	135

The lateral resistances were calculated using Brom's hand calculation method as described in Federal Highways Administration (FHWA) Publication No. FHWA HI 97-013.<sup>8</sup> For all pile types, a free-headed pile was assumed with the vertical load applied at the ground surface. An ultimate compressive strength of 30 megapascals was assumed for the concrete caissons. The SLS values are based on 10 millimetres of deflection at the ground surface.

### 6.3 Liquefaction Potential and Seismic Analysis

#### 6.3.1 Seismic Parameters

The site is located in Cambridge, Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio,  $A$ , applicable to this site is 0.05. The corresponding acceleration-related seismic zone,  $Z_a$  is 1.

The replacement bridge is in Seismic Performance Zone (SPZ) is 1, based on a CHBDC classification as "Emergency Route Bridge". Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient,  $S$ , of 1.0 based on the CHBDC criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

<sup>8</sup> Federal Highway Administration, 1998: *Design and Construction of Driven Pile Foundations, Workshop Manual – Volume 1*. Publication No. FHWA HI 97-013.





### 6.3.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.<sup>9</sup> Although saturated granular layers are present above the bedrock, these layers are generally less than 1 metre thick and were found to have a normalized N value of greater than 22 blows per 0.3 metres. The liquefaction potential is considered to be relatively low based on the soil profile type, age of the deposits, relative density and the historically low seismicity. Therefore a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

## 6.4 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments 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 OPSS Granular A or Granular B Type III but with less than 5 per cent passing the No. 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 OPSD 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with SP 105S10.
- The granular fill may be placed either in a zone with a width equal to at least 1.4 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 a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).

<sup>9</sup> FHWA, 1997: "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



- For Case a, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Soil unit weight:	20 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50
Passive, $K_p$	3.0

For Case b, the unrestrained case, 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 <sup>3</sup>	21 kN/m <sup>3</sup>
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.
- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHDBC.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

## 6.5 Elastic Modulus of Abutment Backfill

The existing abutment backfill materials range from sand and gravel, sand, silty sand, sandy silt, to clayey silt. Various factors affect the value of a soil's Elastic Modulus including the soil loading process, particle organization, and water content. Typical ranges of Elastic Moduli,  $E_s$ , for the various existing backfill materials, as well as potential future use backfill material, have been provided in the following table.



Location	$E_s$ (MPa)		
	Existing Fill	Granular A	Granular B Type III
North Abutment	12	125	37.5
South Abutment	11	125	37.5

## 6.6 Approach Embankments

The embankment fill at both approaches is underlain by buried topsoil 0.2 to 0.8 metres thick. Peat layers 0.3 to 0.9 metres thick were encountered above the bedrock at the southern approach embankment. It is not considered necessary to remove the buried topsoil and peat beneath the existing embankments; however, these materials should be removed from the footprints of the widened areas. The maximum anticipated grade change will be about 2.0 and 1.0 metres at the north and south approach embankments, respectively. The embankment will be widened approximately 2.8 metres (1.4 on each side) at the approaches. The settlement resulting from the grade increase and widening is expected to be negligible.

The existing foreslopes will be modified by removing materials up to 12 to 15 metres behind the existing toe. The preliminary General Arrangement drawing indicates that the foreslopes will be constructed at 2 horizontal to 1 vertical and will feature concrete slope protection. Foreslopes constructed no steeper than 2 horizontal to 1 vertical are expected to be stable.

All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. The exposed subgrade should be proofrolled prior to fill placement under the direction of a geotechnical QVE. All grading and embankment construction should be conducted in accordance with MTO Special Provision 206S03 and 105S10 (amendment to OPSS 501). Except for the top approximately 0.5 metres, where Granular A and B Type III material will be placed for the pavement, the embankment fills should consist of an approved granular borrow such as SSM. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments in accordance with OPSD 208.010 and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of two horizontal to one vertical or flatter.

## 6.7 Construction Considerations

The discussion presents construction related comments and recommendations pertaining to shallow foundations, caissons and driven piles.



### **6.7.1 Shallow Foundations**

If the median pier is supported on shallow foundations bearing on bedrock, dewatering of the overlying sands will be required. The dolomitic limestone to dolostone bedrock is highly fractured and pitted. It is expected that the bedrock at the surface will be weakened by water present in the granular materials. Blasting is not recommended since it may generate more fractures and widen existing ones. If close control of the excavation dimensions is required, then impact hammers or hydraulic splitters may be used. Use of hydraulic splitters and expansion agents; however, is less preferable since it is difficult to control the direction of the cracks in fractured rock with these methods.

Footings must be formed in competent bedrock that is relatively free of defects. The bearing surface should be cleared of all loose and broken rock. The cleaned excavation base should be inspected by a QVE qualified in geotechnical engineering. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical QVE on site and the footing constructed immediately after bedrock inspection. Additional probing and/or excavation may be required if inspection reveals that the bedrock at the design foundation depth is unsuitable or voids are suspected within the bedrock. Due to the presence of relatively very poor rock between elevation 272 and 273.5 metres, footing excavations should be limited in extent and should only proceed as deep as required to remove all loose and excessively broken bedrock.

Bearing surfaces may be improved if open joints or cracks are present by use of non-shrink grout. Use of non-shrink grout depends on the discontinuity width and only suitable if the fault, crack or seam of crushed/faulted bedrock is not more than one third of the footing width. The discontinuity should be cleaned to a depth equal to twice its width and backfilled with non-shrink grout or structural concrete. Grouting through drilled holes may also be used to remediate discontinuities or other natural openings in the bedrock. A Non-Standard Special Provision (NSSP) should be added to the Contract Documents to alert the Contractor to the requirements for constructing shallow foundations on rock.

The new footings may be wider than the original 2.4 metre wide footings. Mass concrete placed during the original construction may be present at the design footing elevation. It may be possible to leave this material in place if inspection reveals that the structural integrity is intact and the dimensions and reinforcement are compatible with the new pier. It is also possible that remnants of temporary structures used during construction of the existing overpass may be buried in place.



### **6.7.2 Caissons**

Caissons socketed into the bedrock may be used to support the median pier. A temporary or permanent liner will be required to support the excavation to the rock surface due to the presence of granular fills and saturated granular native materials. Spoil removal will require a use of augers and coring for overburden and bedrock, respectively. Coring for caisson construction will, however, be problematic because of the tendency for the fractured rock to jam coring equipment. The cleaned excavation base should be inspected by the geotechnical QVE prior to placing reinforcement and pouring concrete.

### **6.7.3 Driven Piles**

The abutments will likely be designed to be supported on driven piles. The currently proposed design includes integral abutments which require steel H-piles. It should be noted that the existing embankment fill contains cobbles which may interfere with advancement of the piles. An NSSP should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with cobbles during pile installation.

## **6.8 Excavations**

### **6.8.1 General**

Excavations for pile caps or strip/spread footings will encounter the existing fill materials, buried topsoil, native compact to very dense granular materials, stiff clayey silt, stiff peat and weathered bedrock. Excavations for pile caps or shallow foundations will encounter the existing structure and may also encounter remnants of temporary structures used during the original construction. Excavations for shallow foundations and caissons designed to bear on/or be socketed in the bedrock will extend below the inferred groundwater level elevation of 277 metres. Temporary open cut slopes within the fill materials above groundwater levels should be maintained no steeper than 1 horizontal to 1 vertical.

Some groundwater seepage from the lowermost granular fills and native granular materials should be expected. Groundwater inflow from the bedrock should be anticipated due to the degree of fracturing. A 1.8 metre by 1.8 metre by 0.9 metre deep test pit was excavated for Geocres Report No. 40P8-76 to evaluate the quality of the bedrock. This report noted that groundwater was present in the test pit upon excavation but that it was dewatered by pumping at a rate of 400 Imperial gallons per hour (30 litres per minute). The rate of groundwater inflow into bedrock excavations is considered to be highly variable as it is dependent on the nature of fracturing which will not be constant across this site. For construction of pile caps and shallow foundations, it is considered that adequate groundwater control could be achieved by pumping from properly constructed and filtered sumps in the base of the excavations but outside of the actual footing limits. Temporary or permanent liners would be required to prevent caving of the granular soils and if caissons are constructed. Groundwater control, such as



pumping from properly constructed and filtered sumps, may be required based on timing of construction and prevailing weather conditions. Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away from the excavations at all times. The appropriate NSSP should be included in the Contract Documents. All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations For Construction Projects. The fill and native organic materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey silt and properly dewatered cohesionless materials would be classified as Type 2 soils.

### **6.8.2 Temporary Protection Systems**

Temporary roadway protection systems will be required where space is restricted and will not permit open cuts, to support the sides of the excavation and permit the use of vertical cuts. It is understood that a protection system is to be used during the construction of the median pier foundations. These systems are to be designed by the Contractor to Performance Level 2 as specified by OPSS 539. The limits of the systems are to be determined by the contractor.

Temporary support systems 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. Since the depth to bedrock is shallow (2.6 to 3.1 metres), use of a soldier pile and lagging wall with the soldier piles installed in pre-augered (drilled) holes and socketed into bedrock is preferred. Support to the systems 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.

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter provided that the centre to centre pile spacing is greater than three times the pile socket diameter.

The unfactored triangular earth pressure distribution ( $p$  in  $\text{kN/m}^2$ ; increasing with depth), can be calculated as follows:

$$p = K_a (\gamma H + q)$$

where  $H$  = the height of the excavation at any point in metres

$K_a$  = active coefficient of earth pressure

$\gamma$  = soil unit weight

$q$  = surcharge for traffic and other loading



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For granular fill and native materials, the unfactored rectangular earth pressure distribution ( $p$  in  $\text{kN/m}^2$ ; constant with depth), can be calculated as follows:

$$p = 0.65 K_a (\gamma H + q)$$

where  $H$  = the total height of the excavation  
 $K_a$  = active coefficient of earth pressure  
 $\gamma$  = soil unit weight  
 $q$  = surcharge for traffic and other loading

The support systems may be designed using the following parameters:

Soil Type	Coefficient Of Earth Pressure			Angle of Internal Friction (degrees)	Unit Weight ( $\text{kN/m}^3$ )
	Active, $K_a$	At Rest, $K_o$	Passive, $K_p$		
Granular Fill	0.33	0.50	3.0	30	19.0
Sand and Gravel	0.27	0.43	3.7	35	21.0
Clayey Silt	0.38	0.55	2.7	27	19.0
Sand	0.33	0.50	3.0	30	20.0
Silty Sand	0.32	0.48	3.1	31	20.0

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 Team Leader, Mr. Storer J. Boone, Ph.D., 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.**

**ORIGINAL SIGNED**

Dirka U. Prout, P.Eng.  
Project Engineer

**ORIGINAL SIGNED**

Storer J. Boone, Ph.D., P.Eng.  
Associate

**ORIGINAL SIGNED**

Fintan J. Heffernan, P.Eng.  
MTO Designated Contact

DUP/SJB/FJH/cr

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n:\active\2010\1132 - geotechnical\1132-0000\10-1132-0056 delcan - gwp 4-00-00 - hwy 401\ph 2000 - foundations\reports\r03 speedsville rd up\1011320056-2000-r03 feb 13 13 (final)  
fdns part a&b speedsville rd underpass.docx



TABLE I

**COMPARISON OF FOUNDATION ALTERNATIVES REPLACEMENT STRUCTURE**

Speedsville Road Underpass, Site 33-145  
 Highway 401 Improvements  
 GWP 4-00-00

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>ESTIMATED COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
Spread footings for median pier supported on competent bedrock	<ul style="list-style-type: none"> <li>• Feasible for median pier only, not suitable for abutments</li> <li>• Preferred technical alternative</li> </ul>	<ul style="list-style-type: none"> <li>• Least expensive option</li> <li>• Founding surface can be readily inspected and treated as required prior to footing construction</li> <li>• Ease of construction</li> <li>• Less expensive than deep foundation options</li> <li>• Piers of existing structure are supported on spread footings with satisfactory performance</li> </ul>	<ul style="list-style-type: none"> <li>• Not compatible with integral abutments</li> <li>• Longer construction time and larger work area required compared to caissons</li> <li>• Rock excavation may be difficult if stronger seams encountered</li> </ul>	<ul style="list-style-type: none"> <li>• \$35,000 for median pier</li> <li>• Higher cost will be incurred if remediation is required</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low risk</li> <li>• Extensive remediation and/or deeper excavations required if bedrock quality at founding elevation is unsuitable</li> </ul>
End bearing steel H-pile foundations for abutments driven to refusal into bedrock	<ul style="list-style-type: none"> <li>• Feasible for abutments, not suitable for median pier</li> <li>• Preferred technical alternative</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Negligible settlement</li> <li>• Only solution compatible with integral abutments</li> <li>• Abutments of existing structure supported on H-piles with satisfactory performance</li> <li>• Most cost effective deep foundation option</li> </ul>	<ul style="list-style-type: none"> <li>• More expensive than shallow foundations</li> <li>• Bedrock at founding depth cannot be inspected</li> <li>• Possible pile tip damage if piles are not adequately protected</li> <li>• More noise and vibration compared to caissons</li> </ul>	<ul style="list-style-type: none"> <li>• Estimated cost \$270 per metre length of pile</li> </ul>	<ul style="list-style-type: none"> <li>• Low risk</li> <li>• Pile tip elevations may vary due to variation in rock quality</li> </ul>

**COMPARISON OF FOUNDATION ALTERNATIVES REPLACEMENT STRUCTURE**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>ESTIMATED COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
Single drilled shafts (caissons) socketed into bedrock	<ul style="list-style-type: none"> <li>• Feasible for abutments and median pier</li> </ul>	<ul style="list-style-type: none"> <li>• Single caisson has higher load capacity than single pile.</li> <li>• Lower equipment mobilization costs compared to driven piles</li> <li>• Less noise and vibration compared to driven piles</li> <li>• If size permits, bedrock at foundation level can be inspected</li> <li>• Faster construction and less work space required compared to shallow foundations</li> <li>• Compared to driven piles, less potential for caissons to be impeded by cobbles in existing embankment fill</li> </ul>	<ul style="list-style-type: none"> <li>• May not be cost-effective for support of median pier considering shallow depth to bedrock (less than 3 metres)</li> </ul>	<ul style="list-style-type: none"> <li>• \$1,000 per metre length of caisson (high costs expected since temporary or permanent liners required to control potential ground losses in granular fill and native soils and rock coring required)</li> </ul>	<ul style="list-style-type: none"> <li>• Low risk</li> <li>• Pile tip elevations may vary due to spatial variation in rock quality</li> </ul>
End bearing concrete filled steel tube piles for abutment driven to refusal on bedrock	<ul style="list-style-type: none"> <li>• Feasible for conventional or semi-integral abutments only</li> <li>• Not suitable for median pier</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Negligible settlement</li> </ul>	<ul style="list-style-type: none"> <li>• More costly than shallow footings</li> <li>• Not compatible with integral abutments</li> <li>• More noise and vibration than caissons</li> <li>• Bedrock at founding level cannot be inspected</li> <li>• Possible pile tip damage if piles are not adequately protected</li> </ul>	<ul style="list-style-type: none"> <li>• Estimated cost \$275 per metre length of pile</li> </ul>	<ul style="list-style-type: none"> <li>• Low to moderate risk</li> <li>• Pile tip elevations may vary due to variation in rock quality</li> </ul>

- NOTES:
1. Costs are very preliminary estimates based on 2012 construction costs and are intended to provide a comparison between alternatives rather than actual construction costs.
  2. Table to be read in conjunction with accompanying report.

**RECORD OF BOREHOLE No 301**

1 OF 2

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808493.5 ; E 235957.5

ORIGINATED BY MA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE, HQ CASING

COMPILED BY WDF

DATUM GEODETIC

DATE April 26, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE		W <sub>p</sub>	W	W <sub>L</sub>		
285.14	PAVEMENT SURFACE						20	40	60	80	100				GR SA SI CL
0.00	ASPHALT														
0.15	FILL, crushed sand and gravel, trace silt														
0.40	Brown														
	FILL, sand and gravel, trace silt, trace clay, with cobbles		1	SS	28										
	Compact		2	SS	16										
	Brown		3	SS	26										
282.24															
2.90	FILL, silty sand, some gravel		4	SS	9										
281.48	Loose														
3.66	Brown														
	FILL, sand and gravel, trace silt		5	SS	15										
	Compact														
	Brown														
280.72															
4.42	FILL, silty sand, some gravel, trace to some clay, with cobbles		6	SS	17										
	Compact to very dense		7	SS	100/50mm										
	Brown														
279.35															
5.79	FILL, silty sand, some gravel, trace topsoil		8	SS	69										
	Compact to very dense		9	SS	17										
	Brown														
277.67															
7.47	FILL, sand and gravel, some silt		10	SS	16										
	Compact														
	Brown														
276.91															
8.23	FILL, silty sand, trace gravel, trace topsoil		11	SS	13										
276.54	Compact														
8.60	Brown														
8.84	TOPSOIL, sandy, some silt														
275.87	Compact		12	SS	10										
9.27	Black														
9.39	SAND, fine, some silt, trace clay, trace topsoil		13	SS	100/0mm										
275.23	Compact														
9.91	Grey		14	SS	50/0mm										
	SAND, fine, trace silt														
	Compact														
	Grey														
	PEAT, fibrous														
	Stiff		15	NQ RC											
	Black														
	Tan brown to grey DOLOMITIC LIMESTONE TO DOLOSTONE. Generally massive, fine crystalline with common vuggy, calcite-filled voids. Traces of pyrite, common stylolites. Core pitted and highly fractured to elev. 271.7m, with irregular, rough surfaces with calcite coating/infilling. Strength R2-R3.		16	NQ RC											
270.17															

Continued Next Page

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

LDN\_MTO\_06 10-1132-0056-2000.GPJ LDN\_MTO.GDT 11/02/13



**RECORD OF BOREHOLE No 302**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808576.8 ; E 235939.6

ORIGINATED BY MA

DIST HWY 401





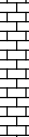
BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE, HQ CASING

COMPILED BY WDF

DATUM GEODETIC

DATE April 27, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			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																			
285.56	PAVEMENT SURFACE						20	40	60	80	100											GR	SA	SI	CL		
0.00	ASPHALT																										
0.21	FILL, crushed sand and gravel, trace silt																										
0.43	Brown																										
	FILL, sand and gravel, trace silt, trace clay, with cobbles Dense Brown		1	SS	42																						
			2	SS	39																						
283.43																											
2.13	FILL, sand, fine to medium, some gravel, trace silt, trace clay Compact to dense Brown		3	SS	24																						
			4	SS	20																						
			5	SS	30																						
			6	SS	27																						
280.71	FILL, silty sand, some gravel Compact Brown			7	SS	30																					
4.85																											
280.38	Brown																										
5.18	FILL, sand and gravel, trace silt Compact to dense Brown		8	SS	27																						
279.62																											
5.94	FILL, sandy silt, some gravel, some clay Compact Brown		9	SS	34																						
278.85																											
6.71	SAND AND GRAVEL, some silt, with cobbles Dense to very dense Brown		10	SS	100/ 200mm																						
277.64																											
7.92	Interbedded, thinly laminated to massive light to dark grey DOLOMITIC LIMESTONE TO DOLOSTONE Strength R4-R5, with common to rare millimetre-scale, horizontally interbedded dark grey shale, grading into tan brown, fossiliferous dolomitic limestone Strength R3-R5. Slightly weathered below elev. 277.1m. Rubble noted between elev. 276.4m and 276.0m. Local calcite infilled voids below elev. 274.3m.																										
			11	NQ RC																							
			12	NQ RC																							
			13	NQ RC																							
273.46	END OF BOREHOLE																										
12.10	Borehole dry during drilling on April 27, 2012.																										

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

**RECORD OF BOREHOLE No 303**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808572.8 ; E 235934.3

ORIGINATED BY MA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE, HQ CASING

COMPILED BY WDF

DATUM GEODETIC

DATE April 30, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			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					○					
285.58	PAVEMENT SURFACE						20	40	60	80	100	10	20	30	GR	SA	SI	CL
0.00	ASPHALT																	
0.18	FILL, crushed sand and gravel, trace silt																	
0.43	Brown																	
	FILL, sand and gravel, trace silt, trace clay, with cobbles		1	SS	33													
	Loose to dense																	
	Brown																	
282.68																		
2.90	FILL, sand, fine to medium, some silt, trace gravel, trace clayey silt seams		4	SS	4													
	Loose to compact																	
	Brown																	
			5	SS	25													
			6	SS	21													
280.40																		
5.18	FILL, sand and gravel, some silt, trace gravel, trace clay		7	SS	27													
	Compact																	
	Brown																	
279.64																		
5.94	FILL, silty sand, some gravel, trace to some clay		8	SS	37													
	Dense																	
			9	SS	33													
278.11																		
7.47	SAND AND GRAVEL, with topsoil		10	SS	28													
	Compact																	
	Brown																	
277.35																		
8.23	Tan brown to buff white, very thinly laminated, fine crystalline																	
	DOLOMITIC LIMESTONE TO DOLOSTONE with minor fossils		11	NQ														
	Strength R4-R5 to elev. 276.4m																	
	grading into tan brown fossiliferous dolomitic limestone Strength R3.																	
	Narrow interbedded interval of muddy dolomitic limestone between elev. 276.0m and 274.3m containing dark grey, millimetrescale, interbedded shale. Rubble noted between elev. 276.4m and 276.0m. Vuggy calcite fillings below elev. 274.2m.		12	NQ														
			13	NQ														
273.45																		
12.13	END OF BOREHOLE																	
	Borehole dry during drilling on April 30, 2012.																	

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

**RECORD OF BOREHOLE No 304**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808493.3 ; E 235952.0

ORIGINATED BY MA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE, HQ CASING

COMPILED BY WDF

DATUM GEODETIC

DATE May 1, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)					
								○ UNCONFINED		+ FIELD VANE												
								● QUICK TRIAXIAL		× LAB VANE												
285.19	PAVEMENT SURFACE						20	40	60	80	100											
0.00	ASPHALT																					
0.18	FILL, crushed sand and gravel, trace silt																					
0.40	Brown																					
	FILL, sand and gravel, trace silt, with cobbles		1	SS	25								○									
	Compact Brown																					
			2	SS	22								○									
			3	SS	15								○									
			4	SS	11								○									
281.53																						
3.66	FILL, silty sand, fine, trace clay		5	SS	16								○									
	Compact Brown																					
280.77																						
4.42	FILL, sandy silt, some gravel, trace clay		6	SS	19								○									
	Loose to compact Brown																					
			7	SS	7								○									
279.25																						
5.94	FILL, sand, fine, trace gravel, trace silt		8	SS	33								○									
278.88	Dense Brown																					
6.31	FILL, sand and gravel, trace to some silt, trace clay												○									
	Dense Brown																					
			9	SS	43								○									
276.96																						
8.23	TOPSOIL, sandy		10	SS	11									○								
	Compact Black																					
276.20																						
8.99	CLAYEY SILT, some topsoil																					
275.80	Stiff Grey		11	SS	9									○								
9.39	PEAT, fibrous																					
9.66	Stiff Black																					
	Blue-grey DOLOMITIC LIMESTONE TO DOLOSTONE grading to fine crystalline, fossiliferous, brown dolomitic limestone. Possible blue-grey anhydrite nodules between elev. 275.3m and 274.8m. Stylolites common. Common fracturing throughout with calcite-filled and smooth stained surfaces. Strength R4. Pitted and vuggy with localized large voids with calcite surfaces below elev. 274.8m. Rubble noted near elev. 271.7m.		12	NQ RC																		
			13	NQ RC																		
			14	NQ RC																		
271.69																						
13.50	END OF BOREHOLE																					
	Borehole dry during drilling on May 1, 2012.																					

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

LDN\_MTO\_06 10-1132-0056-2000.GPJ LDN\_MTO.GDT 11/02/13

**RECORD OF BOREHOLE No 305**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808477.8 ; E 235954.4

ORIGINATED BY MA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY WDF

DATUM GEODETIC

DATE May 1, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE								
						● QUICK TRIAXIAL	×	LAB VANE										
284.60	PAVEMENT SURFACE							20	40	60	80	100						
0.00	ASPHALT																	
0.15	FILL, crushed sand and gravel, trace silt																	
0.37	Brown																	
283.90	FILL, sand and gravel, trace silt, with cobbles		1	SS	19		284											
0.70	Brown																	
282.95	FILL, silty sand and gravel		2	SS	17		283										30 36 25 9	
1.65	Compact Brown																	
282.16	FILL, silty sand, fine to medium, some gravel, trace clay		3	SS	18		282											
2.44	Compact Brown																	
	FILL, silty sand, some gravel		4	SS	20		281											
	Compact Brown																	
			5	SS	16		280											
279.88	FILL, sand, fine to medium, some silt, trace to some gravel, trace to some clay		6	SS	23		279										12 61 16 11	
4.72	Compact to very dense Brown																	
			7	SS	58		278											
278.66	FILL, sand and gravel, trace to some silt		8	SS	79		277											
5.94	Compact to very dense Brown																	
			9	SS	26		276											
276.74	TOPSOIL, sandy, some gravel, some silt		10	SS	25													
7.86	Compact Black																	
276.37	PEAT, fibrous		11	SS	9													
8.23	Stiff Black																	
275.46	PROBABLY BEDROCK		12	SS	-													
9.21	END OF BOREHOLE																	
	Auger refusal (Probably on bedrock)																	
	Groundwater encountered at about elev. 277.1m during drilling on May 1, 2012.																	



**RECORD OF BOREHOLE No 306**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000  
W.P. 4-00-00 LOCATION N 4808595.5 ; E 235936.3 ORIGINATED BY MA  
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF  
DATUM GEODETIC DATE May 2, 2012 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
								20 40 60 80 100									
285.11	PAVEMENT SURFACE																
0.00	ASPHALT						285										
0.15	FILL, crushed sand and gravel, trace silt																
0.37	Brown																
284.38	FILL, sand and gravel, trace silt, with cobbles		1	SS	27												
0.73	Brown																
	FILL, sand, fine to medium, interbedded with silty sand, trace to some gravel		2	SS	25												
	Compact to dense																
	Brown																
			3	SS	34												
			4	SS	28												
			5	SS	30												
280.96	FILL, clayey silt, trace sand and gravel																
4.15	Very stiff																
4.42	Brown		6	SS	49												
	FILL, sandy silt, some gravel, trace to some clay																
	Compact to dense																
	Brown		7	SS	16												
			8	SS	19												
278.89	FILL, silty sand, trace topsoil																
6.22	Compact																
278.40	Brown																
6.71	TOPSOIL, silty																
277.89	Loose		9	SS	12												
7.22	Black																
277.58	SAND, fine, some silt, trace gravel		10	SS	100/0 mm												
7.53	Compact																
	Brown																
	END OF BOREHOLE																
	Auger refusal (Probably on bedrock)																
	Groundwater encountered at about elev. 277.9m during drilling on May 2, 2012.																

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

**RECORD OF BOREHOLE No 307**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808536.9 ; E 235958.5

ORIGINATED BY BT

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE, HQ CASING

COMPILED BY LMK

DATUM GEODETIC

DATE June 3, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT	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									
279.49	PAVEMENT SURFACE						20	40	60	80	100						
0.00	ASPHALT																
0.21	FILL, crushed sand and gravel, trace silt																
0.46	Brown																
	FILL, sand and gravel, trace silt, with cobbles		1	SS	33												
278.12	Dense Brown																
1.37	FILL, sand, fine to medium, trace to some gravel, trace silt		2	SS	61												
277.63	Very dense Brown																
1.86	FILL, sandy silt, trace to some gravel		3	SS	27												
1.98	Dense Brown																
	SAND, fine, some silt, some clay, trace gravel		4	NQ RC													
276.35	Compact Brown																
3.14	Tan brown to light grey, thinly laminated DOLOMITIC LIMESTONE TO DOLOSTONE with millimetre scale shaley interbeds decreasing with depth between elev. 275.7m and 274.2m. Rubbly zone between elev. 275.7m and 275.3m. Pale chalky white calcite filled voids near elev. 275.3m. Massive with no visible structure. Strength R5 to R6 above elev. 274.2m, R3 below.		5	NQ RC													
			6	NQ RC													
			7	NQ RC													
			8	NQ RC													
270.10	END OF BOREHOLE																
9.39	Groundwater encountered at about elev. 276.8m during drilling on June 3, 2012.																

**RECORD OF BOREHOLE No 308**

1 OF 1

**METRIC**

PROJECT 10-1132-0056-2000

W.P. 4-00-00

LOCATION N 4808536.3 ; E 235937.4

ORIGINATED BY BT

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE, HQ CASING

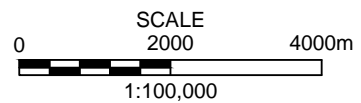
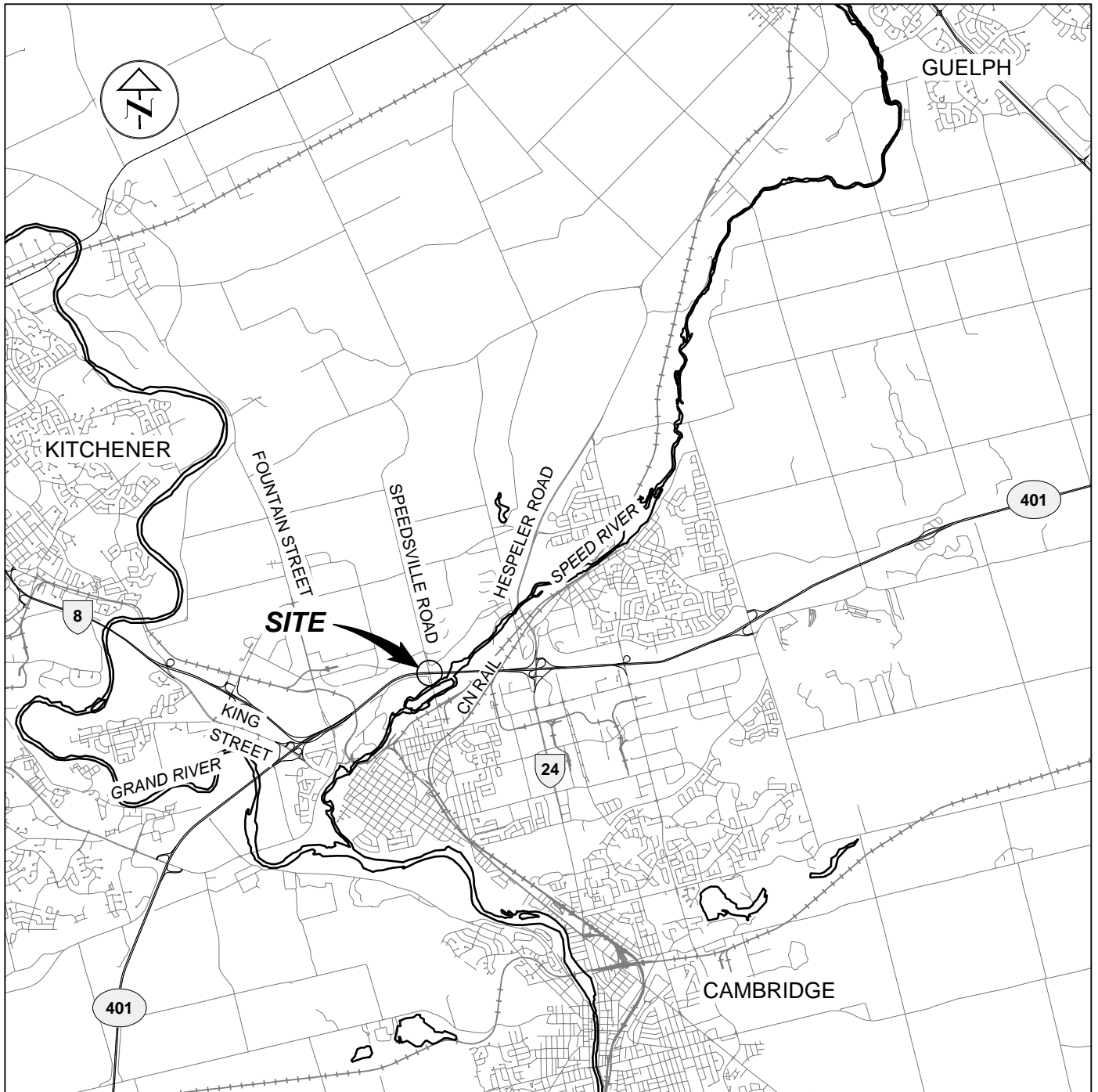
COMPILED BY LMK

DATUM GEODETIC

DATE June 3, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE														
								● QUICK TRIAXIAL      × LAB VANE														
279.42	PAVEMENT SURFACE							20	40	60	80	100						GR SA SI CL				
0.00	ASPHALT																					
0.23	FILL, crushed sand and gravel, trace silt						279															
0.46	Brown																					
	FILL, sand and gravel, trace silt, with cobbles		1	SS	34									○								
278.05	Dense Brown						278															
1.37	FILL, sand, fine to medium, trace silt, trace gravel		2	SS	18										○							
277.53	Compact Brown													○								
1.89																						
276.80	SILTY SAND, fine, trace gravel, trace clay		3	SS	101/175mm		277							○			8 46 32 14					
2.62	Very dense Brown																					
	Fine grained, dark grey, shaley DOLOMITIC LIMESTONE TO DOLOSTONE at elev. 275.3m, grading into tan brown to blue-grey, fossiliferous, fine crystalline to sucrosic dolomitic limestone to limestone. Common irregular and discontinuous, millimetre-scale, green shaley interbeds and healed horizontal fractures to elev. 274.6m. Common stylolites and local calcite-filled vugs and voids below elev. 274.6m Strength R3 to R4.		4	NQ RC			276	70	62	7												
			5	NQ RC			275															
							274	91	89	13												
			6	NQ RC			273	T.C.R. (%) 74	S.C.R. (%) 67	R.Q.D. (%) 7												
			7	NQ RC			272															
							271	96	96	26												
			8	NQ RC			270	86	86	51												
269.67	END OF BOREHOLE																					
9.75	Borehole dry during drilling on June 3, 2012.																					




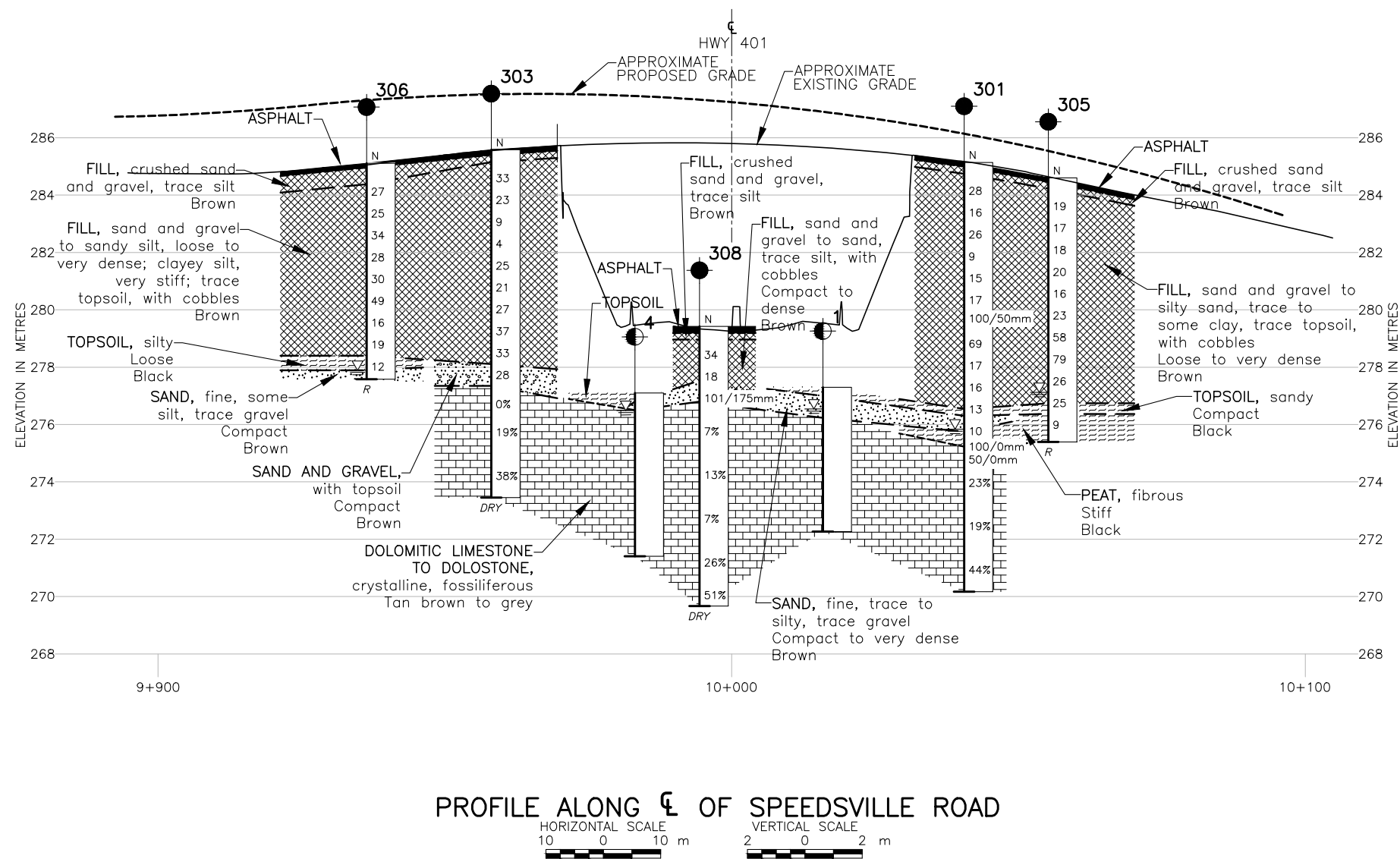
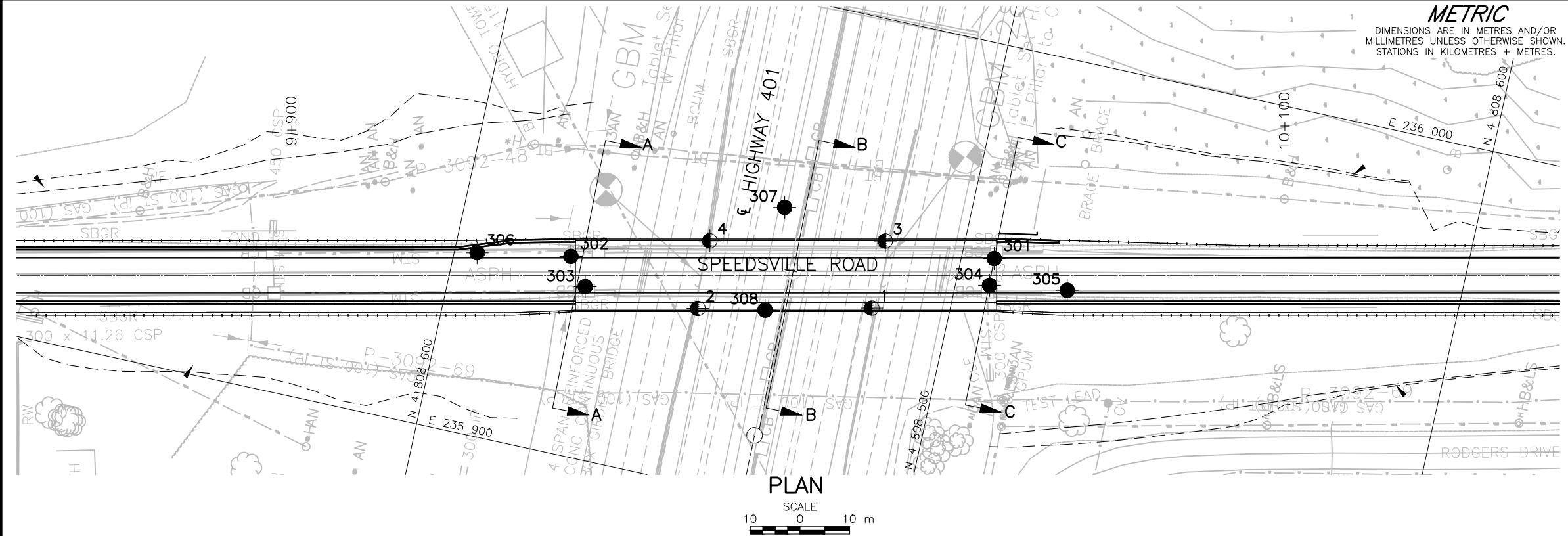
## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

## NOTE

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

PROJECT		SPEEDSVILLE ROAD UNDERPASS, SITE 33-145 HIGHWAY 401 IMPROVEMENTS GWP 4-00-00					
TITLE							
KEY PLAN							
 <b>Golder Associates</b> LONDON, ONTARIO		PROJECT No.	10-1132-0056	FILE No.	1011320056-2000-F03001		
				SCALE	AS SHOWN	REV.	0
		CADD	AMG/LMK	Feb. 11/13		FIGURE 1	
		CHECK					



CONT No.  
WP No. 4-00-00

SPEEDVILLE ROAD UNDERPASS  
HIGHWAY 401 IMPROVEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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

KEY PLAN  
SCALE IN KILOMETRES 0 1 2

LEGEND			
	Borehole - Current Investigation		
	Borehole (Geocres 40P8-76)		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
100%	Rock Quality Designation (RQD)		
	WL encountered during drilling		
DRY	Borehole dry during drilling		
R	Auger or split spoon refusal on inferred bedrock		
No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
301	285.14	4 808 493.5	235 957.5
302	285.56	4 808 576.8	235 939.6
303	285.58	4 808 572.8	235 934.3
304	285.19	4 808 493.3	235 952.0
305	284.60	4 808 477.8	235 954.4
306	285.11	4 808 595.5	235 936.3
307	279.49	4 808 536.9	235 958.5
308	279.42	4 808 536.3	235 937.4
Geocres 40P8-76			
1	277.29	4 808 550.2	235 948.7
2	277.86	4 808 515.4	235 942.4
3	277.52	4 808 549.6	235 934.9
4	277.12	4 808 515.6	235 956.2

**NOTES**

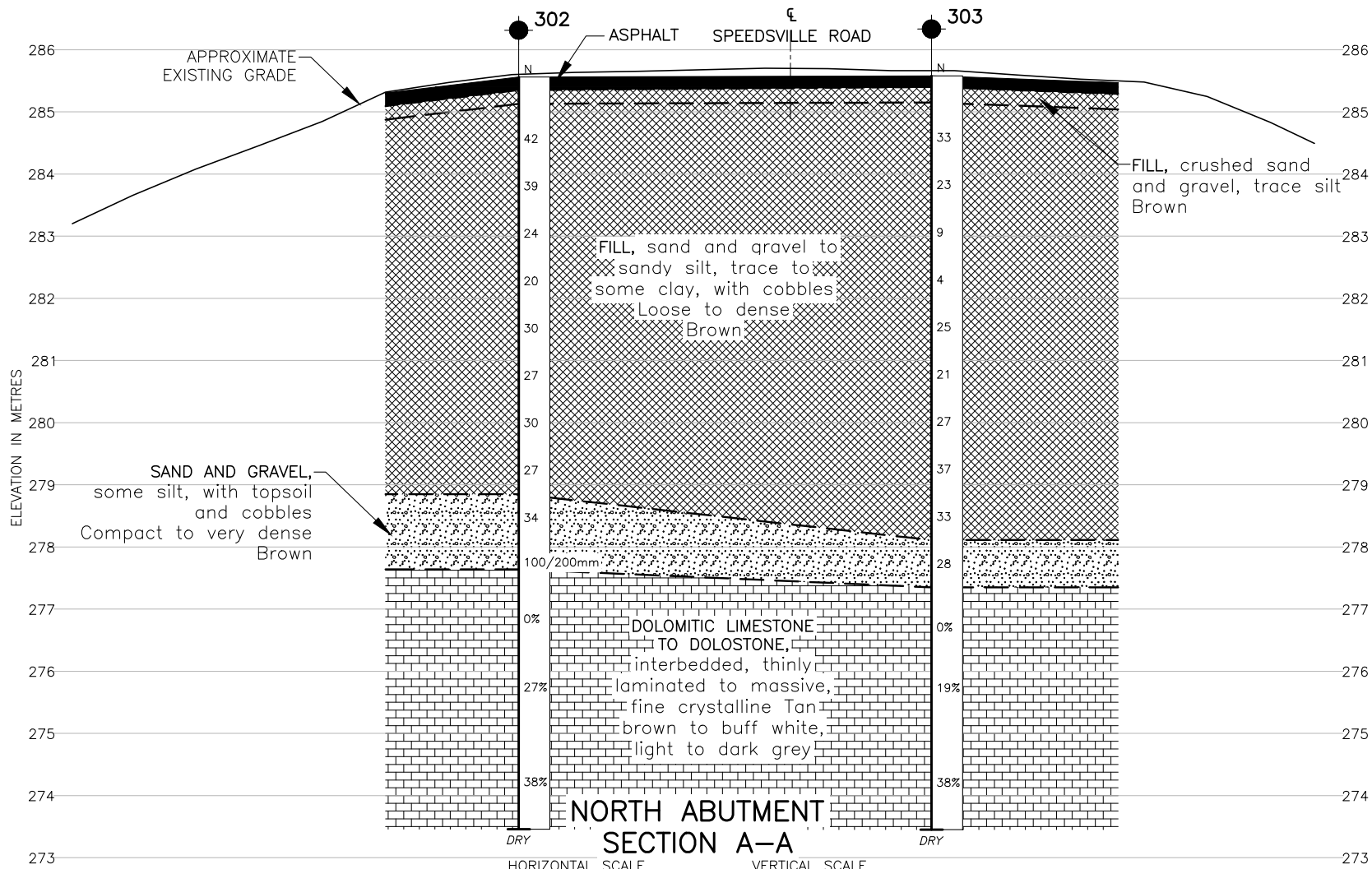
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

**REFERENCE**

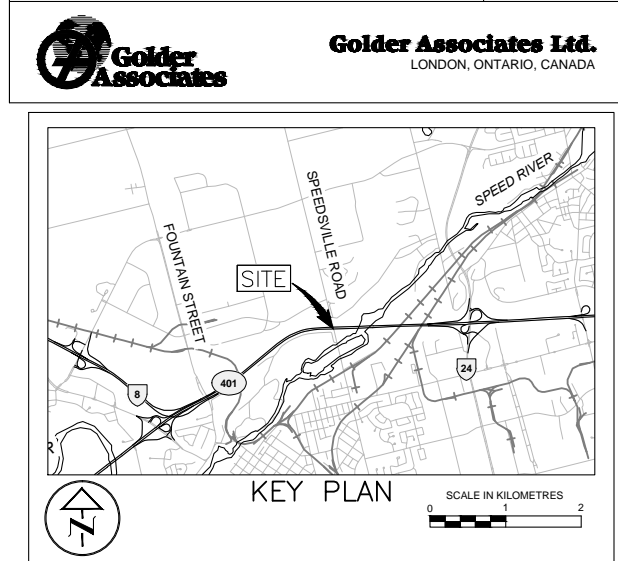
Base plans provided in digital format by Delcan.

NO.	DATE	BY	REVISION
Geocres No. 40P8-204			
HWY.	401	PROJECT NO.	10-1132-0056
SUBM'D.	TP	CHKD.	DUP
DATE:	Sept. 17/12	SITE:	33-145
DRAWN:	LMK	CHKD.	FJH
APPD.	SJB	DWG.	1

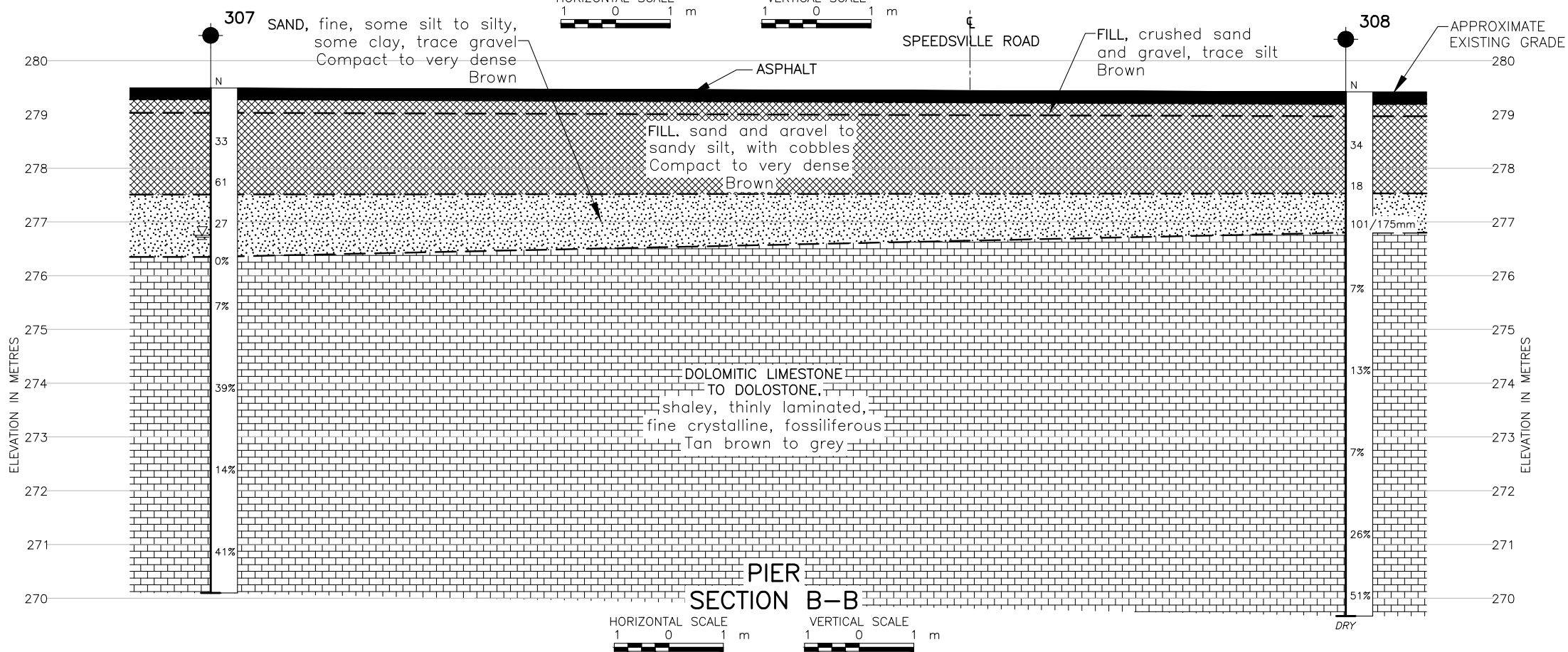


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.	
WP No.	4-00-00
SPEEDSVILLE ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS	SHEET
SOIL STRATA	



LEGEND			
	Borehole - Current Investigation		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
100%	Rock Quality Designation (RQD)		
	WL encountered during drilling		
DRY	Borehole dry during drilling		
No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
302	285.56	4 808 576.8	235 939.6
303	285.58	4 808 572.8	235 934.3
307	279.49	4 808 536.9	235 958.5
308	279.42	4 808 536.3	235 937.4



**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.

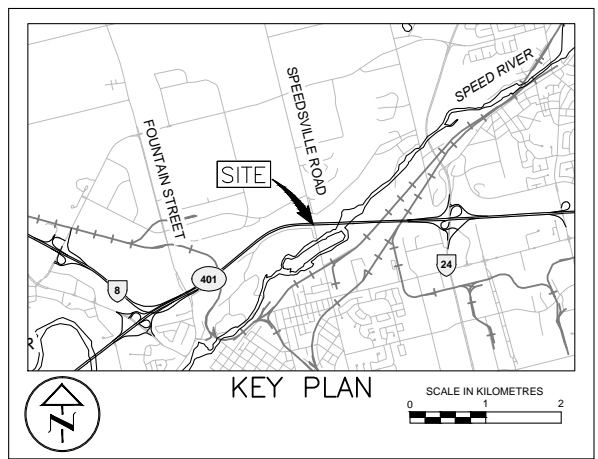
NO.	DATE	BY	REVISION
Geocres No.	40P8-204		
HWY.	401	PROJECT NO.	10-1132-0056
SUBM'D.	TP	CHKD.	DUP
DRAWN:	LMK	CHKD.	FJH
DATE:	Sept. 17/12	APPD.	SJB
DIST.		DATE:	Sept. 17/12
SITE:	33-145	DWG.	2



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

CONT No. WP No. 4-00-00	
SPEEDSVILLE ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS	SHEET
SOIL STRATA	



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



LEGEND			
	Borehole – Current Investigation		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL encountered during drilling		
DRY	Borehole dry during drilling		
No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
301	285.14	4 808 493.5	235 957.5
304	285.19	4 808 493.3	235 952.0

**NOTES**

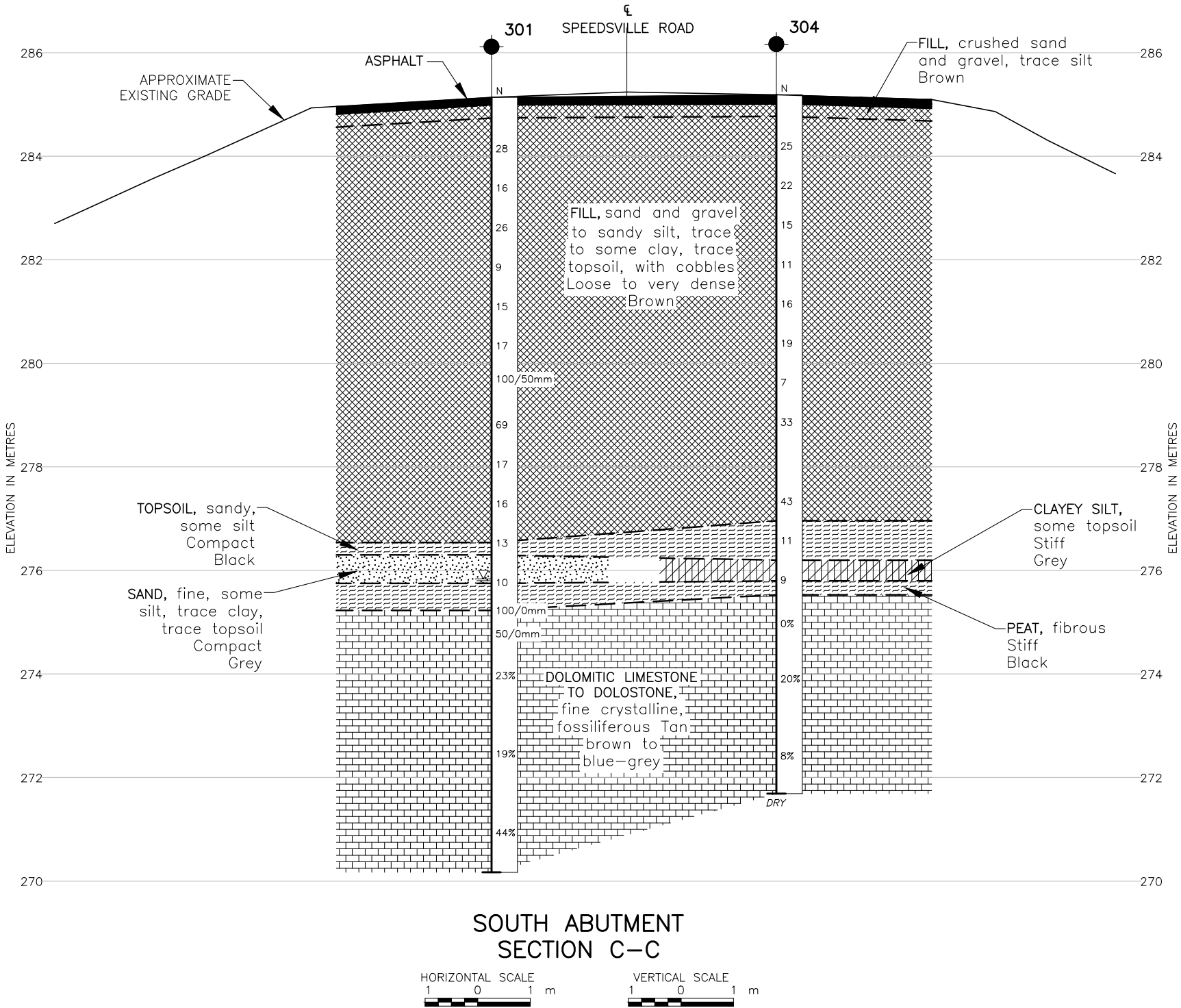
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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. 40P8-204			
HWY.	401	PROJECT NO.	10-1132-0056
SUBM'D.	TP	CHKD.	DUP
DATE:	Sept. 17/12	SITE:	33-145
DRAWN:	LMK	CHKD.	FJH
APPD.	SJB	DWG.	3

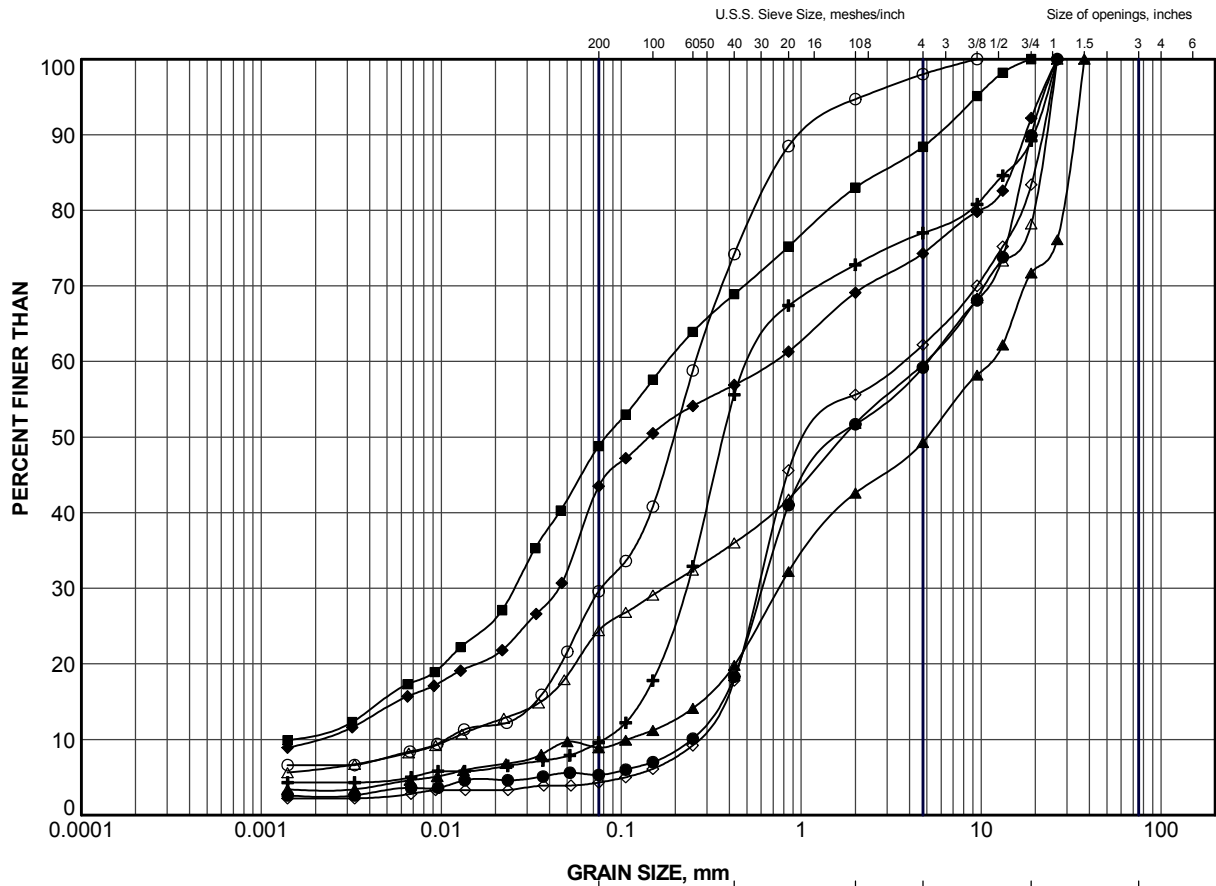




# **APPENDIX A**

## **Laboratory Test Data - Soils**





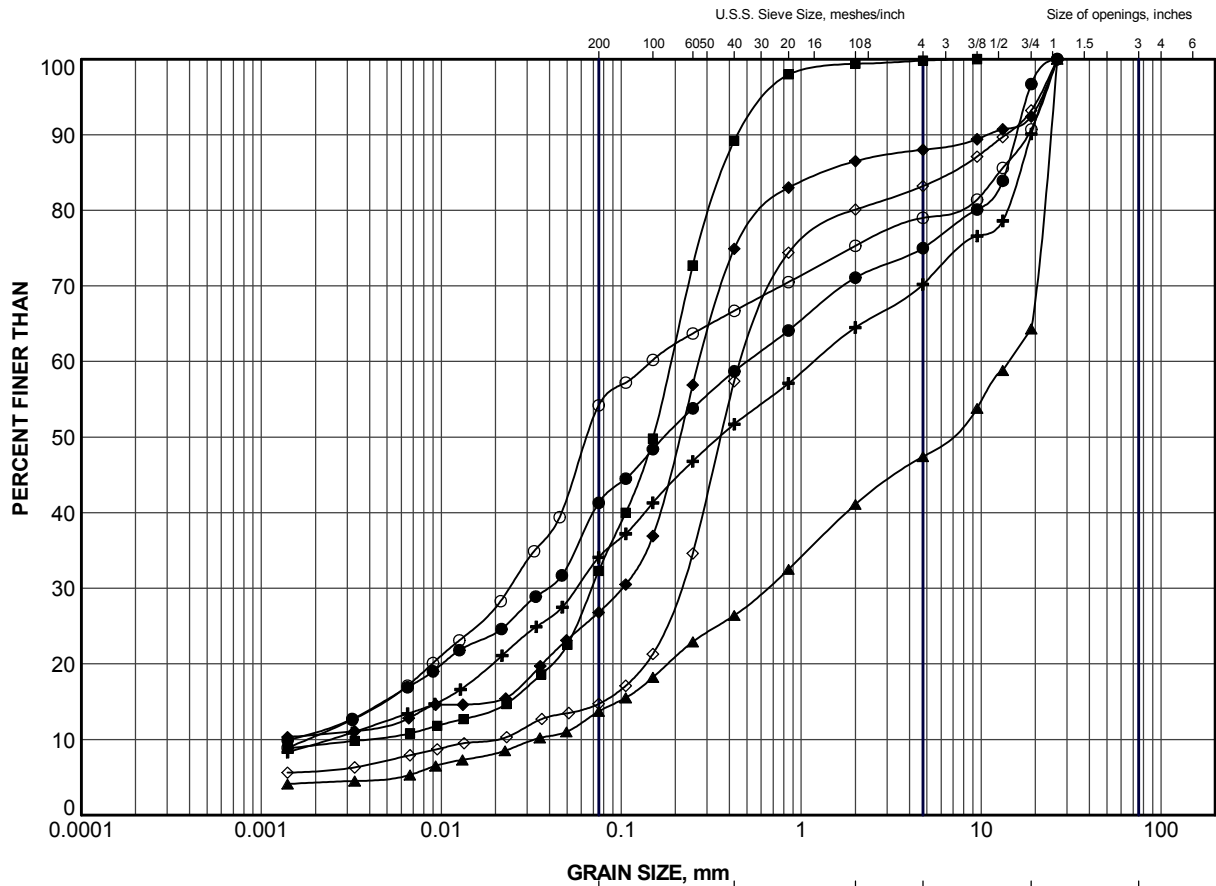
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	2	283.4
■	301	6	280.3
▲	302	1	284.6
+	302	5	281.5
◆	302	8	279.2
◇	303	3	283.1
○	303	4	282.3
△	303	7	280.0

PROJECT				SPEEDSVILLE ROAD UNDERPASS, SITE 33-145 HIGHWAY 401 IMPROVEMENTS GWP 4-00-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.		10-1132-0056		FILE No.		1011320056-2000-F030A1	
DRAWN		LMK		Feb. 11/13		SCALE N/A REV.	
CHECK						FIGURE A-1	





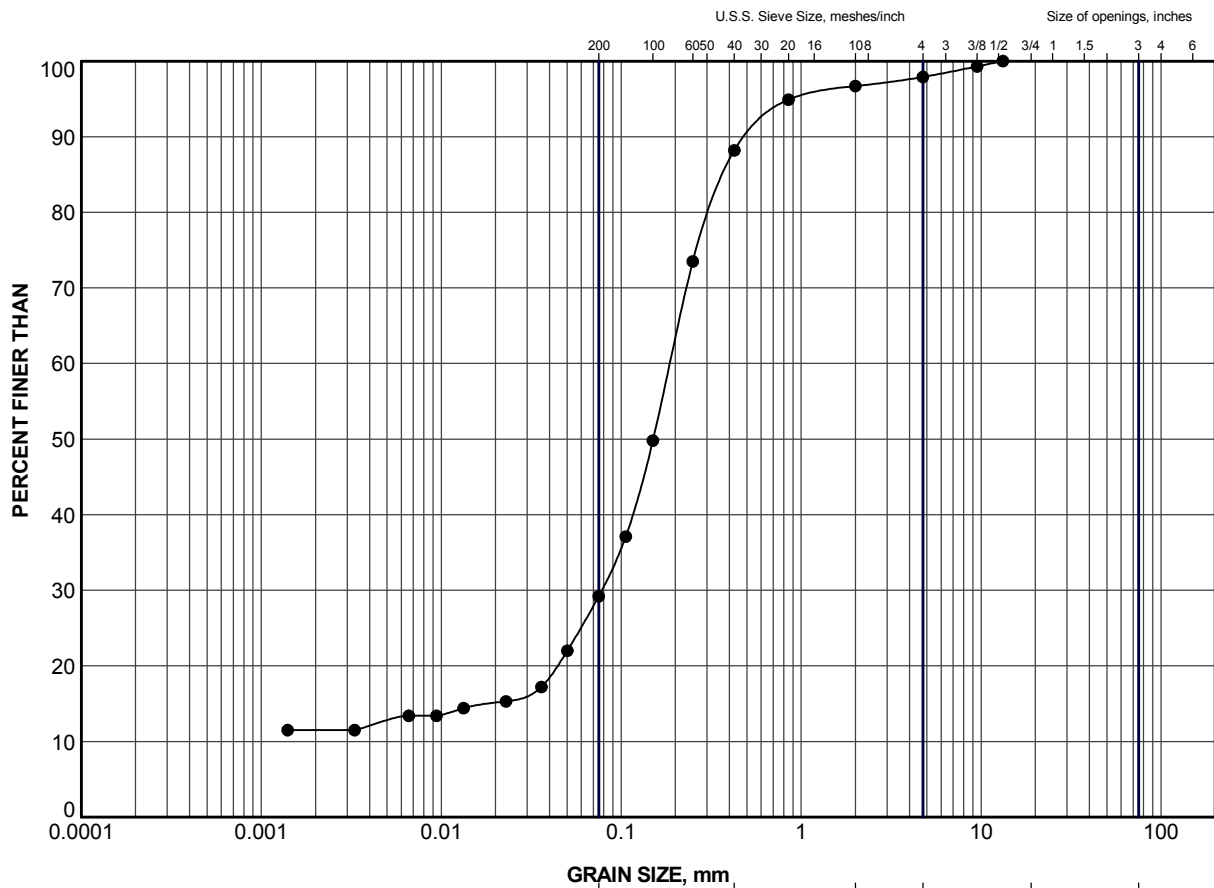
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	303	9	278.5
■	304	5	281.2
▲	304	9	277.3
+	305	2	282.9
◆	305	7	279.0
◇	306	3	282.6
○	306	6	280.3

PROJECT				SPEEDSVILLE ROAD UNDERPASS, SITE 33-145 HIGHWAY 401 IMPROVEMENTS GWP 4-00-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.		10-1132-0056		FILE No.		1011320056-2000-F030A2	
DRAWN		LMK		Feb. 11/13		SCALE N/A REV.	
CHECK						FIGURE A-2	





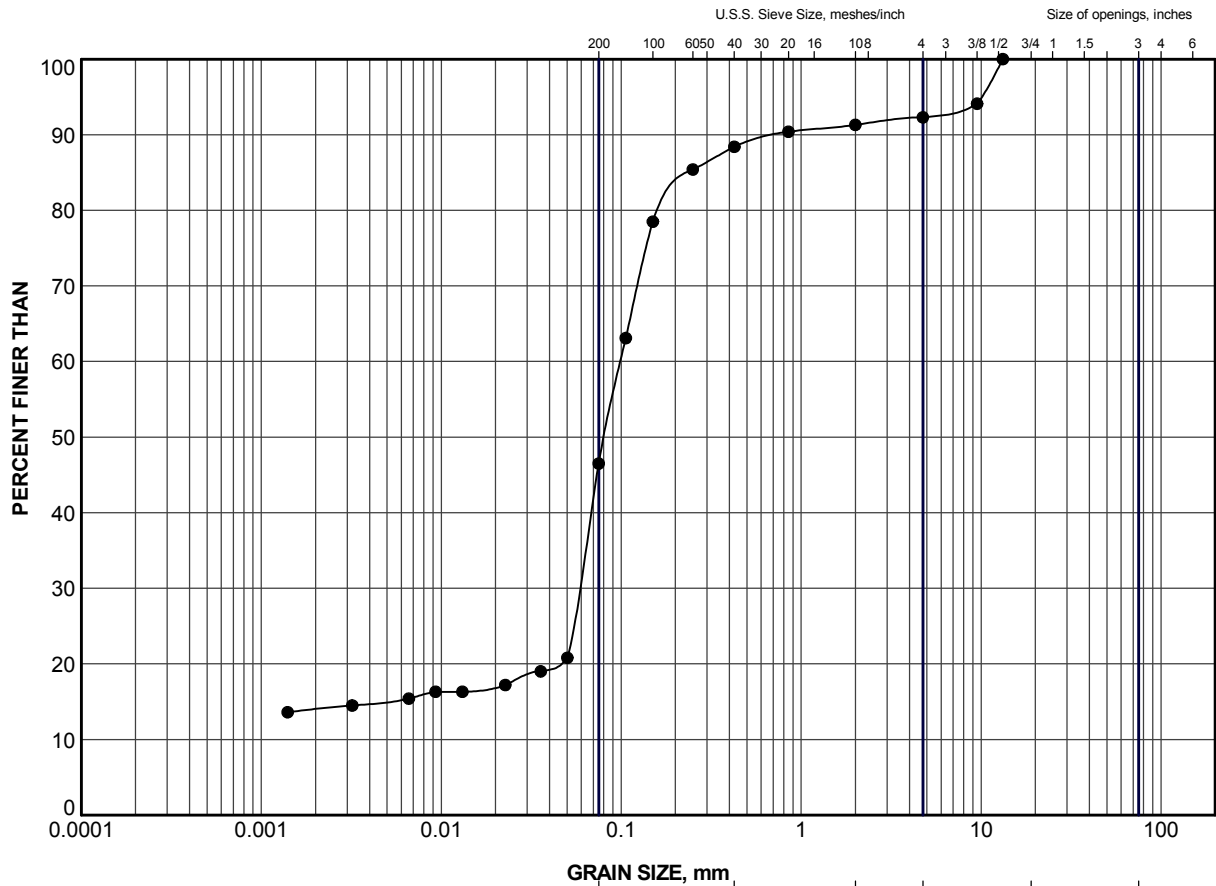
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	307	3	277.0

PROJECT				SPEEDSVILLE ROAD UNDERPASS, SITE 33-145 HIGHWAY 401 IMPROVEMENTS GWP 4-00-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND			
PROJECT No.		10-1132-0056		FILE No.		1011320056-2000-F030A3	
DRAWN		LMK		Feb. 11/13		SCALE N/A REV.	
CHECK						FIGURE A-3	





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

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	308	3	277.0

PROJECT				SPEEDSVILLE ROAD UNDERPASS, SITE 33-145 HIGHWAY 401 IMPROVEMENTS GWP 4-00-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY SAND			
PROJECT No.		10-1132-0056		FILE No.		1011320056-2000-F030A4	
DRAWN		LMK		Feb. 11/13		SCALE N/A REV.	
CHECK						FIGURE A-4	



LDN\_MTO\_GSD\_GLDR\_LDN.GDT



# **APPENDIX B**

## **Laboratory Test Data - Rock**

TABLE BI

**SUMMARY OF UNCONFINED COMPRESSION TESTING**

Speedsville Road Underpass, Site 33-145  
Highway 401 Improvements  
GWP 4-00-00

<b>BOREHOLE</b>	<b>SAMPLE NUMBER (m)</b>	<b>DEPTH (m)</b>	<b>SAMPLE HEIGHT (cm)</b>	<b>SAMPLE DIAMETER (cm)</b>	<b>WATER CONTENT (%)</b>	<b>UNIT WEIGHT (kN/m<sup>3</sup>)</b>	<b>VOID RATIO</b>	<b>UNCONFINED COMPRESSIVE STRENGTH (MPa)</b>
302	13	10.92 - 11.05	10.58	4.46	0.05	26.50	0.00	111.0
307	6	5.74 - 5.92	10.32	4.71	0.04	27.29	0.01	63.8

- NOTES:
1. Detailed test reports are shown on Figures B1A to B2B.
  2. Table to be read in conjunction with accompanying report.

Prepared By: NG  
Checked By: DUP

**UNCONFINED COMPRESSION TEST (UC)****FIGURE B1A****ASTM D 7012-07****SAMPLE IDENTIFICATION**

PROJECT NUMBER	10-1132-0056	SAMPLE NUMBER	-
BOREHOLE NUMBER	302	SAMPLE DEPTH, m	10.92 - 11.05

**TEST CONDITIONS**

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.37

**SPECIMEN INFORMATION**

SAMPLE HEIGHT, cm	10.58	WATER CONTENT, (specimen) %	0.05
SAMPLE DIAMETER, cm	4.46	UNIT WEIGHT, kN/m <sup>3</sup>	26.50
SAMPLE AREA, cm <sup>2</sup>	15.62	DRY UNIT WT., kN/m <sup>3</sup>	26.49
SAMPLE VOLUME, cm <sup>3</sup>	165.29	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	446.90	VOID RATIO	0.00
DRY WEIGHT, g	446.68		

**VISUAL INSPECTION****FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	111.0
----------------------	---	-------------------------	-------

REMARKS:

DATE:

10/10/2012

Checked By:

**Golder Associates**

# UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE B1B



BEFORE COMPRESSION



AFTER COMPRESSION

Date 10/12/2012  
Project 10-1132-0056

**Golder Associates**

Drawn Frank  
Chkd.



**UNCONFINED COMPRESSION TEST (UC)****FIGURE B2A****ASTM D 7012-07****SAMPLE IDENTIFICATION**

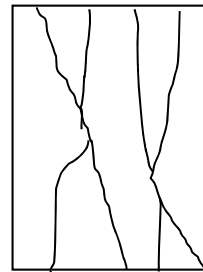
PROJECT NUMBER	10-1132-0056	SAMPLE NUMBER	-
BOREHOLE NUMBER	307	SAMPLE DEPTH, m	5.74-5.92

**TEST CONDITIONS**

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.19

**SPECIMEN INFORMATION**

SAMPLE HEIGHT, cm	10.32	WATER CONTENT, (specimen) %	0.04
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m <sup>3</sup>	27.29
SAMPLE AREA, cm <sup>2</sup>	17.44	DRY UNIT WT., kN/m <sup>3</sup>	27.27
SAMPLE VOLUME, cm <sup>3</sup>	180.03	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	501.10	VOID RATIO	0.01
DRY WEIGHT, g	500.90		

**VISUAL INSPECTION****FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	63.8
----------------------	---	-------------------------	------

REMARKS:

DATE:

10/10/2012

Checked By:

**Golder Associates**

# UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE B2B



BEFORE COMPRESSION



AFTER COMPRESSION

Date 10/12/2012  
Project 10-1132-0056

**Golder Associates**

Drawn Frank  
Chkd.



# **APPENDIX C**

## **Photographs of Rock Core**





## APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 1: BH301 Elevation 274.72 metres to 270.17 metres.



Photograph 2: BH302 Elevation 277.18 metres to 273.46 metres.





## APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 3: BH303 Elevation 276.92 metres to 273.45 metres.



Photograph 4: BH304 Elevation 275.28 metres to 271.69 metres.





## APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 5: BH307 Elevation 276.35 metres to 273.15 metres.



Photograph 6: BH307 Elevation 273.15 metres to 270.10 metres.





## APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 7: BH308 Elevation 276.52 metres to 271.95 metres.



Photograph 8: BH308 Elevation 271.95 metres to 269.67 metres.





# **APPENDIX D**

## **Records of Previous Boreholes (Geocres Report No. 40P8-76)**



# BOREHOLE LOG

Checked By CFF

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Ground Surface			0' 0"					
Fine to medium sand, some organic matter	Brown		909.73	{ S }	1	X bag		moist
Sand, fine to medium, silty	mottled grey brown		— — —	{ S }	2	X bag		Note: losing water after wet.
some organic matter			806.6" 906.33 5' 0"					W.L. 2' 6" Sept. 16, 1958 Note: loose water after 2 feet
Limestone	Dk. grey							badly fissured, weathered & broken up
As above	Lt. grey		10' 0"					fissures & faults
As above	Pale grey brown		15' 0"					fissures & faults
			16' 6" 893.23					Recovery
			Hole terminated					3' 6" - 4' 6" 83.5 % 4' 6" - 7' 3" 59.0 % 7' 3" - 11' 6" 100.0 % 11' 6" - 13' 10" 100.0 % 13' 10" - 16' 6" 87.5 %

# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

Job Name Waterloo Twp No 6

Job No. 58 log

Borehole No. 2

Client Dept. of Highways of Ontario

Casing B.X.

Boring Date Sept 15, 1958

Datum D.H.O.

Compiled By G.Y.S.

Checked By E.F.F.

### SAMPLE CONDITION
























### SAMPLE TYPE

### ABBREVIATIONS

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
ground surface			0' 0"					
andy loam & considerable organic matter	Dk. brown		911.63		1	 bag		Notice moist
& above but less organic content	- " -				2	 bag		loosing water after 2 hrs. v. moist
rock fragments in matrix of	Yellowish brown		3' 0"		3	 bag		
ne sand, minor silt content			908.63					
limestone	Lt. grey	I -	5' 0"				W.L. 4' 6"	Sept 16, 1958
limestone	Dk. grey							badly fissured & fractured
								badly faulted
is above	medium grey							
			10' 0"					
								
is above	pale brown							badly fractured at 11'
	grey							pitted, badly fractured at 13' 3"
								Recovery
			15' 3"					3' 0" - 5' 1" 78.0%
			896.38					5' 1" - 6' 8" 100.0%
								6' 8" - 11' 1" 80.0%
								11' 1" - 15' 3" 100.0%
								
								
								
								

**e. m. peto associates ltd.**  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
**BOREHOLE LOG**

Job Name Waterloo Twp. No. 6

Job No. 58109

Borehole No. 3

Client Dept. of Highways of Ontario

Casing B.X.

Boring Date Sept. 13, 1958

Datum D.H.O.





Compiled By C.Y.S.

Checked By C.F.F.

**SAMPLE CONDITION**














**SAMPLE TYPE**

**ABBREVIATIONS**

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

- V.T. IN SITU VANE SHEAR TEST
- Q<sub>u</sub> UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth, Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Ground Surface			0' 0"					
sandy loam, considerable organic matter	Dk. brown		910.51		1	S.S.		Note: losing water after 2' moist
coarse sand & fine gravel, some organic matter	mixed brown				2	S.S.	W.L.	2.7" Sept 16, 1958 moist
limestone	Lt. grey		907.51					broken up at 3' 3" badly fractured
limestone	Dk. grey		5' 0"					
limestone	- - -							very badly fractured, evidence of deterioration by water action
limestone	Dk to medium grey							very badly fractured, 6' 2" to 6' 11" and 7' 2" to 7' 4"
limestone	medium grey		10' 0"					horizontal & vertical fissures
limestone	Lt. grey							pitted by water action scattered packets of pyrites
								Recovery:
								3' 0" - 5' 6" 100%
								5' 6" - 6' 2" 100%
			16' 0"					6' 2" - 7' 6" 100%
			844.00					7' 6" - 10' 6" 96.6%
			Hole terminated					10' 6" - 16' 0" 96.2%



SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
BOREHOLE LOG

Borehole No. 4

Boring Date Sept. 12, 1958

Checked By C. F. M.

## ABBREVIATIONS

### V. T. IN SITU VANE SHEAR TEST

Q/u UNCONFINED COMPRESSIVE STRENGTH.

W.L. WATER LEVEL IN CASING

W. T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
ground surface			0' 0"					
sandy loam mixed with organic matter	Dk. brown		909.18		1	X	6.5	Note: losing water after 2' moist
to medium sand mixed with rock fragments, minor silt cont., roots	brown, some yellowish brown		2' 3" 906.93		2	X	5.5	W.L. Sept. 16, 1958; 2' moist
limestone	Lt. grey		5' 0"					badly fissured
limestone	Lt.-Dk grey medium grey							badly fissured at 6' 2" and broken up; evidence of deterioration by water action
limestone	medium - Lt. grey		10' 0"					pitted by water, some vertical fissures
limestone	Lt. grey		15' 0"					pitted, badly fissured
			18' 8" 888.27					Recovery: 2' 3"-4' 5" 84.5% 4' 5"-5' 6" 100.0% 5' 6"-8' 5" 88.3% 8' 5"-8' 9" 87.5% 8' 9"-13' 9" 97.5% 13' 9"-18' 8" 100.0%
			Hole terminated					



# **APPENDIX E**

## **Site Photographs**



## APPENDIX E SITE PHOTOGRAPHS



Photograph 1: Speedsville Road Underpass, east elevation (photography courtesy of Ministry of Transportation, Ontario - Bridge Office).



Photograph 2: Speedsville Road Underpass, west elevation (photography courtesy of Ministry of Transportation, Ontario - Bridge Office).

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Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
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North America	+ 1 800 275 3281
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[solutions@golder.com](mailto:solutions@golder.com)  
[www.golder.com](http://www.golder.com)

**Golder Associates Ltd.**  
**309 Exeter Road, Unit #1**  
**London, Ontario, N6L 1C1**  
**Canada**  
**T: +1 (519) 652 0099**

