February 2011

FOUNDATION INVESTIGATION AND DESIGN REPORT

Temporary Bridge at Schneider Creek
Widening of Highway 7/8
From 1.9 km West of Fischer-Hallman Road Interchange
Easterly to 0.8 km East of Courtland Avenue Interchange
Kitchener
GWP 131-98-00
Ministry of Transportation, Ontario - West Region

Submitted to:
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REPORT

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Geocres No. 40P8-194

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FOUNDATION INVESTIGATION REPORT

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COURTLAND AVENUE INTERCHANGE, KITCHENER
GWP 131-98-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 131-98-00, the reconstruction and widening of Highway 7/8. This report presents the results of the foundation investigation conducted for the proposed temporary bridge which will be built over Schneider Creek immediately north of the Highway 7/8 Courtland Avenue Interchange. This temporary crossing is required for access during construction of the retained soil system (RSS) retaining walls west of this area and for work on the nearby Canadian National Railway Overhead Structure and the Highway 7/8 Courtland Avenue Overpass.

The purpose of the foundation investigation is to determine the subsurface conditions at the locations of the proposed works by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal, Golder Associates' proposal P81-3002 dated April 8, 2008, our letters dated July 21 and 22, 2008 and our revised scope of work letter dated April 13, 2010. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated July 4, 2008.

2.0 SITE DESCRIPTION

2.1 General

The project area of Highway 7/8 is located in the south-central area of Kitchener, Ontario. The project extends from 1.9 km west of Fischer-Hallman Road easterly to 0.8 km east of Courtland Avenue. The location of the project is shown on the Key Plan, Figure 1.

This section of Highway 7/8 is currently a four lane divided highway oriented generally east-west. Four overpass structures for Westmount Road, Homer Watson Boulevard, Ottawa Street South and Courtland Avenue East, one underpass structure for Fischer-Hallman Road and an overhead structure for Canadian National Rail (CNR) tracks are situated within the project limits.

The temporary bridge over Schneider Creek will be constructed immediately north of the existing Courtland Avenue Overpass. The existing Courtland Avenue Overpass consists of twin structures, each with four spans, crossing Highway 7/8. The overpass also crosses Schneider Creek which flows southerly into an open channel on the west side of Courtland Avenue. The creek was diverted into an open channel when the existing structures were erected in the late 1960s. The design channel invert was at elevation 311.12 metres. The current Highway 7/8 pavement surface elevation at the overpass structure varies between approximately elevation 322.6 metres at the west abutment to about elevation 320.5 metres at the east abutment. West of Courtland Avenue, Highway 7/8 is on an approximately 9 metre high embankment.

Courtland Avenue is a four lane roadway divided by a concrete median that crosses beneath Highway 7/8 in a northwest-southeast orientation. The pavement surface of Courtland Avenue is at approximately elevation 315 metres. Adjacent land use is typically urban residential to the northeast of the interchange. A golf course is present to the northwest and industrial/commercial properties are present to the south.

2.2 Site Geology

This project lies within the physiographic region of southwestern Ontario known as the Waterloo Hills[1]. The soils generally consist of sandy hills; some are ridges of sandy till while others are kames or kame moraines, with outwash sands deposited in the valleys. Adjoining the sandy hills is the Grand River spillway system comprised of alluvial terraces of sand and gravel.

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

Based on the Ministry of Northern Development and Mines Map 2508 entitled "Quaternary Geology, Cambridge Area", the site is in an area of primarily ice contact sand deposited in the Pleistocene era. Within the ice contact sand are pockets of Maryhill Till (clayey silt till) and Port Stanley Till (silt to sandy silt till).

The Geologic Survey of Canada Map 1263A entitled "Geology, Toronto-Windsor Area, Ontario" indicates that the subcropping bedrock in the area of the site is dolomite and mudstone of the Salina formation of Upper Silurian age. Based on the Ministry of Natural Resources Map P.1985 entitled "Bedrock Topography Series, Cambridge Area, Southern Ontario", the elevation of the bedrock surface at the site ranges between 265 and 270 metres or some 44 to 50 metres below the ground surface on Courtland Avenue.

3.0 INVESTIGATION PROCEDURES

The foundation investigation was carried out on June 9, 2010 at which time borehole 414 was drilled to a depth of 21.8 metres on the east side of Schneider Creek.

This information was supplemented with borehole 413 previously drilled on the west side of the creek. This borehole was advanced for the design of the widening for the Courtland Avenue Overpass (Geocres No. 40P8-176). The locations of boreholes 413 and 414 are shown on the Record of Borehole sheets and on Drawing 1, attached.

The table below summarizes the borehole locations, ground surface elevations at the borehole locations and the borehole depths:

Borehole	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
413 (40P8-176)	4 810 555.8	226 201.8	313.98	31.76
414	4 810 569.0	226 225.7	315.43	21.79

The drilling for borehole 414 was carried out using a truck mounted CME 75 power auger which was supplied and operated by a specialist drilling contractor. Samples of the overburden were generally obtained at 0.75 to 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures. The samplers used in the investigations limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger size particles, including cobbles and boulders, are known to be present in the glacial till deposits as discussed in the text of this report.

The groundwater conditions were observed throughout the drilling operations and are presented in the Record of Borehole sheet. The borehole was backfilled in accordance with current Ministry of Transportation, Ontario (MTO) procedures and Ontario Regulation 372/07.

The field work was monitored on a full-time basis by experienced members of our engineering staff who located the borehole in the field, monitored the drilling, sampling and in situ testing operations, logged the borehole and surveyed the borehole location and elevation. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A. The Record of Borehole 413 (40P8-176) is presented in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ and laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling resistance and represent transitions between soil types rather than exact planes of geological change. Subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the approximate location of the temporary bridge at Schneider Creek generally encountered, in sequence, surficial fill and topsoil, clayey silt interlayered with sandy silt and silt, clayey silt till, silty clay and clayey silt interlayered with silt.

The borehole locations are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized below. A stratigraphic profile is presented on Drawing 2.

4.1.1 Fill

Layers of variable fill materials were encountered from the ground surface in boreholes 413 (40P8-176) and 414. The fill comprised predominantly granular materials ranging from sandy silt to silty sand to sand. Topsoil fill was also encountered at the surface of borehole 413 (40P8-176) and from elevation 315.4 metres in borehole 414. The topsoil fill layers were 150 and 75 millimetres thick in boreholes 413 (40P8-176) and 414, respectively. The overall thickness of the fill layers were 2.0 and 2.3 metres in boreholes 413 (40P8-176) and 414, respectively.

The granular fill was generally compact with standard penetration test N values of 12 to 32 blows per 0.3 metres and water contents of 11 and 13 per cent.

4.1.2 Clayey Silt

Layers of upper clayey silt were encountered beneath the fill in boreholes 413 (40P8-176) and 414 from elevations 312.0 to 313.1 metres. The upper clayey silt was interlayered with sandy silt, silt and, less frequently, sand. An additional layer of clayey silt was encountered at elevation 284.7 metres in borehole 413 (40P8-176). The thicknesses of the upper clayey silt layers were 0.6 to 8.2 metres. The lower clayey silt in borehole 413 (40P8-176) was 1.5 metres thick.

The firm to hard upper clayey silt had N values of 7 to 35 blows per 0.3 metres and water contents of 13 to 19 per cent. The hard lower clayey silt had an N value greater than 100 blows per 0.3 metres. The upper clayey silt was of low plasticity based on plastic limits, liquid limits and plasticity indices ranging from 13 to 16, 19 to 24 and 6 to 8 per cent, respectively. The Atterberg limits results for tests performed on samples of clayey silt are shown on Figure A-5.

The results of the grain size testing conducted on selected clayey silt samples obtained during the standard penetration testing are presented on Figure A-1.

4.1.3 Sandy Silt

A layer of sandy silt was encountered in the clayey silt in borehole 413 (40P8-176) from elevation 307.7 metres. The thickness of the sandy silt layer was 2.0 metres.

The N values measured in the compact to dense sandy silt ranged from 11 to 36 blows per 0.3 metres.

4.1.4 Silt

In boreholes 413 (40P8-176) and 414, layers of silt were found in the clayey silt at elevation 305.8 metres and elevation 312.5 metres, respectively. Additional layers of silt were found in borehole 413 at depth at elevation 285.6 and 283.2 metres. The 1.2 metre thick layer of upper silt in borehole 413 (40P8-176) was very dense with an N value of 61 blows per 0.3 metres and a water content of 20 per cent. The 0.6 metre thick layer of upper silt in borehole 414 was compact with an N value of 11 blows per 0.3 metres. Where fully penetrated, the lower silt in borehole 413 (40P8-176) was 0.9 metres thick. Borehole 413 (40P8-176) was terminated in the lower silt after exploring it for 1.0 metres. The lower silt had N values greater than 100 blows per 0.3 metres and a water content of 20 per cent.

The results of grain size testing conducted on samples of the silt are shown on Figure A-2.

4.1.5 Sand

A layer of dense sand was encountered below the upper clayey silt in borehole 414 at elevation 303.7 metres. The fine sand layer was about 0.8 metres in thickness and had an N value of 33 blows per 0.3 metres.

4.1.6 Clayey Silt Till

Clayey silt till was encountered beneath the upper clayey silt in borehole 413 (40P8-176) and below the sand in borehole 414. The clayey silt till was encountered from elevation 301.5 metres in borehole 413 (40P8-176) and elevation 302.9 metres in borehole 414. The thicknesses of the clayey silt till layers were 9.1 and 8.2 metres in boreholes 413 (40P8-176) and 414, respectively.

The very stiff to hard clayey silt till had N values of 23 to over 100 blows per 0.3 metres with water contents of 11 to 24 per cent. The clayey silt till is of low plasticity based on Atterberg limits determinations. The plastic limits, liquid limits and plasticity indices for clayey silt till samples were 12 to 13, 17 to 20 and 5 to 7 per cent, respectively. The results of the Atterberg limits tests are shown on Figure A-5.

The results of the grain size testing conducted on clayey silt till samples obtained during standard penetration testing are presented on Figure A-3. Evidence of cobbles was encountered in borehole 414. The presence of both cobbles and boulders should be anticipated in the clayey silt till due to the depositional history of this material.

4.1.7 Silty Clay

Hard silty clay was found beneath the clayey silt till in boreholes 413 (40P8-176) and 414 from elevations 292.3 and 294.7 metres, respectively. Clayey silt layers were found in the silty clay in borehole 413 (40P8-176). The thickness of the silty clay layer was 6.7 metres in borehole 413 (40P8-176). Borehole 414 was terminated in the silty clay after exploring the layer for about 1.1 metres.

The silty clay had N values of 42 to 80 blows per 0.3 metres with a water content of 26 per cent. The silty clay is of intermediate plasticity based on an Atterberg limits determination with a plastic limit of 22 per cent, liquid limit of 47 per cent and plasticity index of 25 per cent. The results of the Atterberg limits determination are presented on Figure A-5. A grain size distribution curve for the silty clay is presented on Figure A-4.

4.2 Groundwater Conditions

The groundwater conditions in the boreholes were monitored throughout the fieldwork. The observed groundwater conditions are noted on the Record of Borehole sheets and on Drawing 1 and are summarized in the following text and table.

Summary of Encountered Groundwater Levels

Borehole	Ground Surface Elevation	Encountered Groundwater Level	
		Depth	Elevation
	(m)	(m)	(m)
413 (40P8-176)	313.98	2.0	312.0
414	315.43	3.0	312.4

The groundwater levels in boreholes 413 (40P8-176) and 414 were encountered between elevations 312.0 and 312.4 metres or at depths of 2 to 3 metres. The above-noted groundwater levels are not necessarily considered to be representative of the long-term, stabilized groundwater conditions as the readings were taken for a short duration only.

The encountered groundwater levels in boreholes 413 (40P8-176) and 414 are consistent with the measured and encountered groundwater levels obtained for the geotechnical investigation for the widening of the adjacent Courtland Overpass structure (Geocres No. 40P8-176). Groundwater was encountered between elevations 311.5 and 315.0 metres in boreholes on the north side of the overpass structure. Monitoring data compiled since the fall of 2008 from the shallow standpipe and deep piezometer installed in borehole 401 (40P8-176) indicates that the long term groundwater level in the upper clayey silt is near elevation 313 metres and fluctuates between 312 and 313 metres in the deeper clayey silt till and silty clay.

The water level in Schneider Creek was measured at approximately 312.7 metres during the October 1964 investigation (Geocres No. 40P08-039) and at elevation 311.4 metres on June 10, 2010.

Based on the measured and encountered groundwater levels in boreholes 401 (40P8-176), 413 (40P8-176) and 414, the inferred groundwater level for the site is at elevation 312.5 metres. The groundwater levels are expected to fluctuate due to climatic and seasonal variations.

5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Aardvark Drilling Ltd., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Matthew Rhody and Mr. Daniel Babcock, P.Eng. under the direction of Mr. David J. Mitchell.

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

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PART B

FOUNDATION DESIGN REPORT

TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
FROM 1.9 KM WEST OF FISCHER-HALLMAN ROAD INTERCHANGE
EASTERLY TO 0.8 KM EAST OF COURTLAND AVENUE INTERCHANGE
KITCHENER
GWP 131-98-00
MINISTRY OF TRANSPORTATION, ONTARIO - 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 proposed temporary bridge over Schneider Creek in the Highway 7/8 - Courtland Avenue Interchange area 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.

A temporary crossing is required to provide construction access for the embankment retaining walls between the Canadian National Railway (CNR) Overhead structure and the Courtland Avenue Interchange and work at the CNR Overhead and Courtland Avenue Overpass structures. The design of the temporary bridge will be carried out by the contractor. As a result, only minimal information was available for the preparation of this report. However, it is anticipated that the bridge will consist of a prefabricated modular structure designed and constructed in accordance with Ontario Provincial Standard Specification (OPSS) 918. As an alternative to the bridge option, the currently proposed design consists of temporary culverts for the crossing of Schneider Creek.

6.2 Bridge Foundations

The soil conditions encountered in the boreholes put down during the investigation typically consist of surficial fills overlying firm to hard clayey silt to approximately elevation 302 metres. The clayey silt is interlayered with compact to very dense sandy silt, silt and sand. The clayey silt is underlain by very stiff to hard clayey silt till which is in turn underlain by hard silty clay from about elevation 292 metres west of Schneider Creek and elevation 295 metres east of the creek. The silty clay is underlain by a lower deposit of clayey silt interlayered with silt. The groundwater level was inferred to be at elevation 312.5 metres. The water level in Schneider Creek was measured at approximately elevation 313 metres in October 1964 and 311 metres in June 2010.

Based on the results of the boreholes and the anticipated type of bridge, it is expected that shallow crib foundations will be used for the temporary bridge structure. Alternatively, deep foundations could be considered.

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

6.2.1 Shallow Foundations

Crib foundations for a pre-fabricated modular temporary bridge structure can be erected directly on the sandy silt to sand fill near elevation 314 metres. Assuming a foundation width of 3 metres, a factored geotechnical resistance of 250 kilopascals at Ultimate Limit States (ULS) and a geotechnical resistance of 200 kilopascals at Serviceability Limit States (SLS) can be used for design of these foundations. The SLS value assumes a maximum settlement of 50 millimetres. The geotechnical resistances are based on the edge of the footing being at least one metre behind the top of the channel slope. Shallow foundations are the preferred foundation alternative from a foundation engineering perspective.

6.2.2 Deep Foundations

The abutments for the proposed temporary bridge could be designed using driven HP 310 x 110 steel H-piles or 324 millimetre outside diameter (O.D.), open steel tube piles with a nominal 9.5 millimetre thick wall thickness.

Geotechnical Axial Resistance – Driven Steel H-Piles

For design, the factored axial geotechnical resistances at Ultimate Limit States (ULS) for HP 310 x 110 piles driven into the hard clayey silt till to or below the elevations are shown in the following table. The SLS values assume 50 millimetres of settlement. It was assumed that the cut-off elevations will be similar to the existing ground surface elevation.

Location	Cut-Off Elevation	Proposed Tip Elevation	Pile Length	Founding Strata	Geotechnical Resistances	
					Factored ULS	SLS
	(m)	(m)	(m)		(kN)	(kN)
West abutment	314.0	297.0	17.0	Hard clayey silt till	950	625
East abutment	315.0	297.0	18.0	Hard clayey silt till	950	625

The steel H-piles should be installed and monitored in accordance with Ontario Provincial Standard Drawing (OPSD) 3000.150 and OPSS 903. The piles are to be equipped with reinforced flanges as shown in OPSD 3000.100

Geotechnical Axial Resistance – Driven Steel Tube Piles

Open steel tube piles 324 millimetres in outside diameter with a 9.5 millimetre wall thickness driven open ended may be used for support of the temporary crossing structure. The factored ULS resistance and SLS geotechnical resistance stated in the table below are available for tube piles driven to or below the elevations shown. The SLS values assume 50 millimetres of settlement. It was assumed that the cut-off elevations will be similar to the existing ground surface elevation.

Location	Cut-Off Elevation	Proposed Tip Elevation	Pile Length	Founding Strata	Geotechnical Resistances	
					Factored ULS	SLS
	(m)	(m)	(m)		(kN)	(kN)
West abutment	314.0	298.5	15.5	Hard clayey silt till	600	400
East abutment	315.0	300.5	14.5	Hard clayey silt till	600	400

The steel tube piles should be installed and monitored in accordance with OPSD 3001.150 and OPSS 903. The piles are to be equipped with Type 1 driving shoes as shown in OPSD 3001.100.

Construction Considerations

Cobbles were encountered in the clayey silt till layers in borehole 414. Cobbles and boulders are likely present in the till soils and may impact pile driving operations. A non-standard special provision (NSSP) should be added to the contract documents to alert the contactor to the presence of cobbles and boulders within the till soils.

Downdrag Load (Negative Skin Friction)

Limited fill placement is anticipated for construction of the temporary crossing over Schneider Creek. Therefore, negligible negative skin friction is expected to develop on the piles installed for this structure.

Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. 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_s = coefficient of horizontal subgrade reaction (MPa/m) $= n_h\,(z/d)$ for cohesionless soils

$= \dfrac{67S_u}{d}$ for cohesive soils

d = pile width or diameter (m)

n_h, k_{si} = constant of horizontal subgrade reaction (MPa/m)

z = depth below ground surface grade (m)

Soil Type	Elevation (m)		n_h	S_u
	From	To	(MPa/m)	(MPa)
Compact to dense fill (sandy silt to silty sand)	Surface	312	4 – 6	-
Firm to hard clayey silt	312	302	-	0.05 – 0.23
Compact to dense sandy silt (west abutment only)	308	306	2 – 8	-
Very dense silt (west abutment only)	306	305	10 – 12	
Dense sand (east abutment only)	304	303	6 – 8	-
Very stiff to hard clayey silt till	304	292	-	0.15 – 0.45

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

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

For the purposes of design, the lateral geotechnical resistance for an HP 310 x 110 pile can be taken as 160 kilonewtons at factored ULS and 65 kilonewtons at SLS based on assessed values quoted in Table C6.4 of the CHBDC. For a 324 millimetre diameter, 9.5 millimetre wall tube pile, the corresponding resistances are 175 kN at factored ULS and 65 kN at SLS based on Brom's Method. The SLS values are based on 10 millimetres of deflection.

Monitoring of Existing Structure

The process of installing the new piles will produce ground vibrations in the surrounding soils. It is anticipated that structures more than one pile length away from the areas where the new piles are constructed are not likely to be affected[2]. Loose, clean, saturated and uniform granular soils were considered to be the most problematic with respect to pile driving vibration. Such soils were not encountered in the boreholes. Since a small number of piles will be installed, vibration and settlement monitoring in conjunction with the pile driving operations is not considered warranted unless the temporary bridge will be installed within one pile length of the existing overpass structure and a structural analysis indicates that this bridge is in a condition that it is susceptible to vibrations.

6.3 Temporary Flow Passage System

It is currently proposed to use a temporary flow passage system consisting of a culvert installed north of the Courland Avenue Overpass structure to allow access to the north slope of Highway 7/8 and the Canadian National Railway (CNR) Overhead structure. This temporary structure is to be approximately 10 metres long. The temporary passage systems/water body crossing is to be designed, supplied, installed, maintained and removed by the Contractor. The design should conform to the requirements of OPSS 182 and OPSD 221.010 or 221.040, as applicable.

The borehole data suggests that the channel bed will likely consist of firm clayey silt or compact silt with clayey silt layers. Therefore a channel liner consisting of rounded stone should be placed prior to placement of the fill. Fill placed in the channels would consist of rip-rap or gabion stone in accordance with OPSS 182. Disturbed areas of the bank and channel invert should be provided with temporary scour and erosion protection until construction is complete, and the channel restored. These requirements should be included in a Non-Standard Special Provision for temporary water body crossings.

6.4 Excavations and Temporary Cut Slopes

Excavations for shallow foundations or pile caps construction will extend into the existing granular fill materials. These excavations are not expected to encounter the groundwater level which has been inferred to be at elevation 312.5 metres. However, localized seepage may be encountered within the clayey silt. Groundwater control, such as pumping from properly constructed and filtered sumps, may be required based on the timing of construction and the 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.

[2] Woods, Richard D. : Dynamic Effects of Pile Installations on Adjacent Structures, National Cooperative Highway Research Program Synthesis of Highway Practice 253. National Academy Press, Washington, D.C., 1997

Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical. 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 granular fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey materials and properly dewatered cohesionless materials would be classified as Type 2 soils.

If the excavations for the west pile cap encroach into the toe of the slope of the existing or newly widened approach embankment of the Courtland Avenue overpass structure, temporary protection systems may be required to support and maintain the stability of the embankment. The temporary support system could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds or driven steel sheet piling. Support to the system could be in the form of struts and walers or rakers and anchors. The temporary excavation support system should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system.

6.5 Foundation Preparation For Crib Foundations

Prior to placement of the timber cribs, the ground surface must be levelled. It is recommended that a levelling pad, constructed of compacted Granular A with a minimum thickness of 1 metre, be placed to provide a level surface. The footprint of the levelling pad must extend beyond the extents of the timber cribbing a minimum distance of 1 metre plus the thickness of the levelling pad. In areas where a continuous level area is not possible, benches may be constructed as necessary. However, the timber cribs must be set back well away from the edges of benches.

7.0 MISCELLANEOUS

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

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Temporary Bridge at Schneider Creek
Widening of Highway 7/8
GWP 131-98-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Shallow crib foundation	• Feasible • Preferred technical option	• Lowest cost • Rapid construction • Easily removed	• Lower geotechnical resistance in the firm to stiff soils immediately underlying the existing fill	• $25,000 per abutment	• Bearing materials with relatively low strengths near surface. Can be replaced with compacted Granular A pad
End bearing steel H-pile foundations driven into hard clayey silt till	• Feasible	• High bearing resistance • Negligible settlement • Less vibration related damage compared to steel tube piles • Higher capacity than steel tube piles when driven to end bearing	• Susceptible to deflection if cobble nest or boulders encountered within till materials • Must be left in place	• $270 per lineal metre	• Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils
End bearing or friction, open steel tube piles driven into hard clayey silt till	• Feasible	• High bearing resistance • Negligible settlement	• Higher vibration related damage potential compared to H-pile • Must be left in place	• $275 per lineal metre if concrete filled; $245 per lineal metre if unfilled	• Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils

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

Prepared By: DUP
Checked By: PRB

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 414

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00 | LOCATION N 4810569.0 ;E 226225.7 | ORIGINATED BY MR
DIST | HWY 7/8 | BOREHOLE TYPE POWER AUGER / HOLLOW STEM | COMPILED BY WDF
DATUM GEODETIC | DATE June 9, 2010 | CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
315.43	GROUND SURFACE						
	FILL, sand, some silt, trace gravel						
0.15	FILL, topsoil, silty Dark brown						
	FILL, sandy silt, some clay, some topsoil, trace to some gravel Compact to dense Brown	1	SS	20			
		2	SS	32			
313.14 2.29	CLAYEY SILT, trace sand Stiff Brown	3	SS	11			
312.53 2.90	SILT, with clayey silt layers Compact Grey	4	SS	11	∇		
311.92 3.51	CLAYEY SILT, trace to some sand, with silt seams Stiff to hard Grey	5	SS	9			
		6	SS	20			
		7	SS	18			
		8	SS	19			0 2 69 29
		9	SS	16			
		10	SS	31			
		11	SS	23			0 0 76 24
		12	SS	16			
303.70 11.73	SAND, fine, some silt Dense Grey						
302.93 12.50	CLAYEY SILT TILL, trace to some sand, trace gravel, with cobbles Hard Grey	13	SS	33			
		14	SS	52			2 13 61 24

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
UNIT WEIGHT γ kN/m^3
ELEVATION SCALE: 315 – 301

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 414

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00 | LOCATION N 4810569.0 ;E 226225.7 | ORIGINATED BY MR

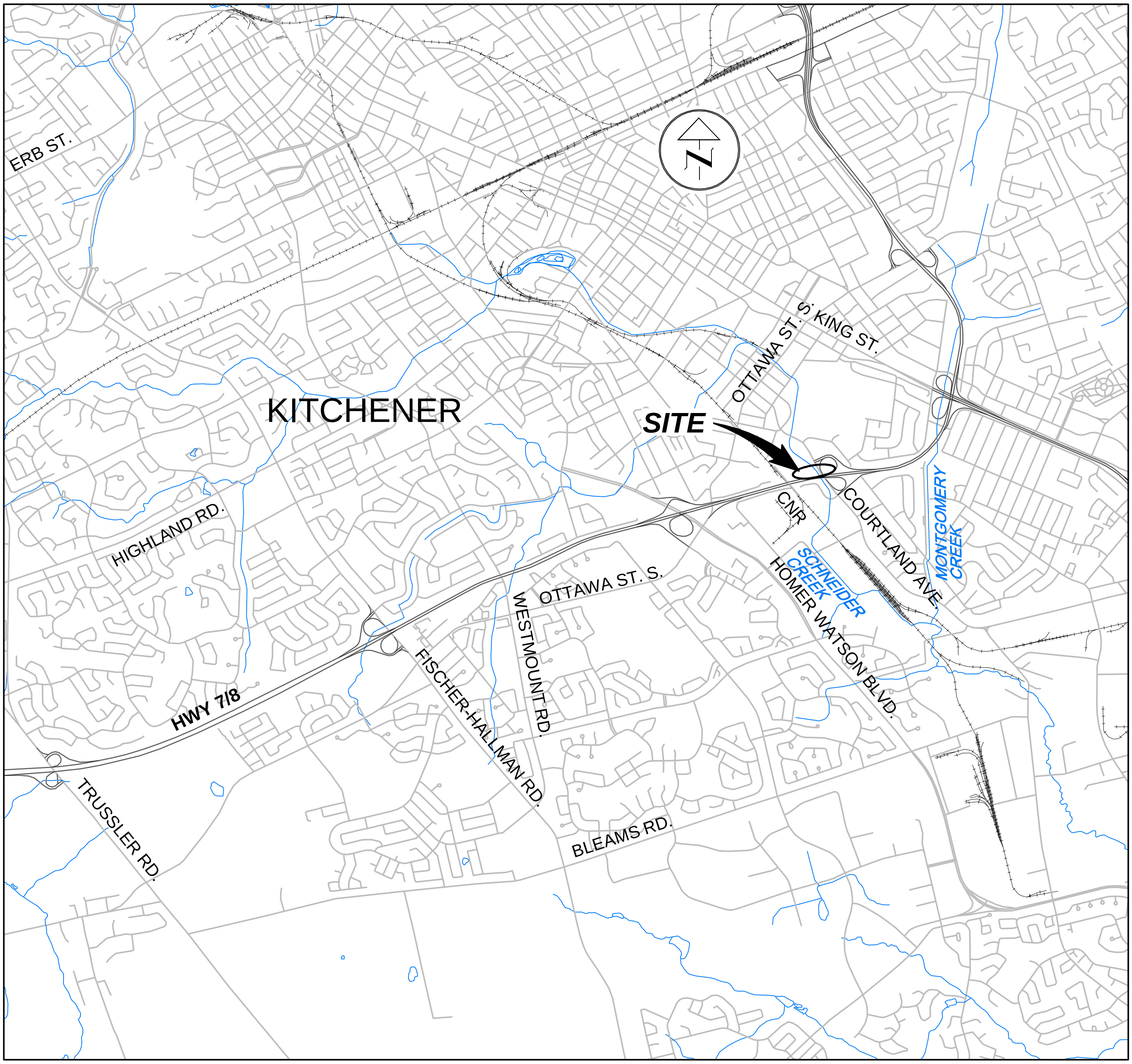
DIST ____ HWY 7/8 | BOREHOLE TYPE POWER AUGER / HOLLOW STEM | COMPILED BY WDF

DATUM GEODETIC | DATE June 9, 2010 | CHECKED BY ____

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (w_P – w – w_L)	GR	SA	SI	CL
	CLAYEY SILT TILL, trace to some sand, trace gravel, with cobbles Hard Grey	15	SS	62	300					
					299					
		16	SS	31	298					
		17	SS	100/ 225mm	297	plotted	1	17	59	23
					296					
		18	SS	53	295					
294.70 20.73	SILTY CLAY, trace sand Hard Grey	19	SS	42	294					
293.64 21.79	END OF BOREHOLE Groundwater encountered at about elev. 312.4m during drilling on June 9, 2010.									

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

$+^3, \times^3$: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE



REFERENCE

DRAWING BASED ON CANMAP STREETFILES V2005.4.

NOTE

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

PROJECT TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-084-1	FILE No. 0811320841-F14001
		SCALE AS SHOWN REV.
CADD WDF	Nov. 08/10	
CHECK		**FIGURE 1**

Drawing file: 0811320841-F14001.DWG Feb 18, 2011 - 9:12am

METRIC

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

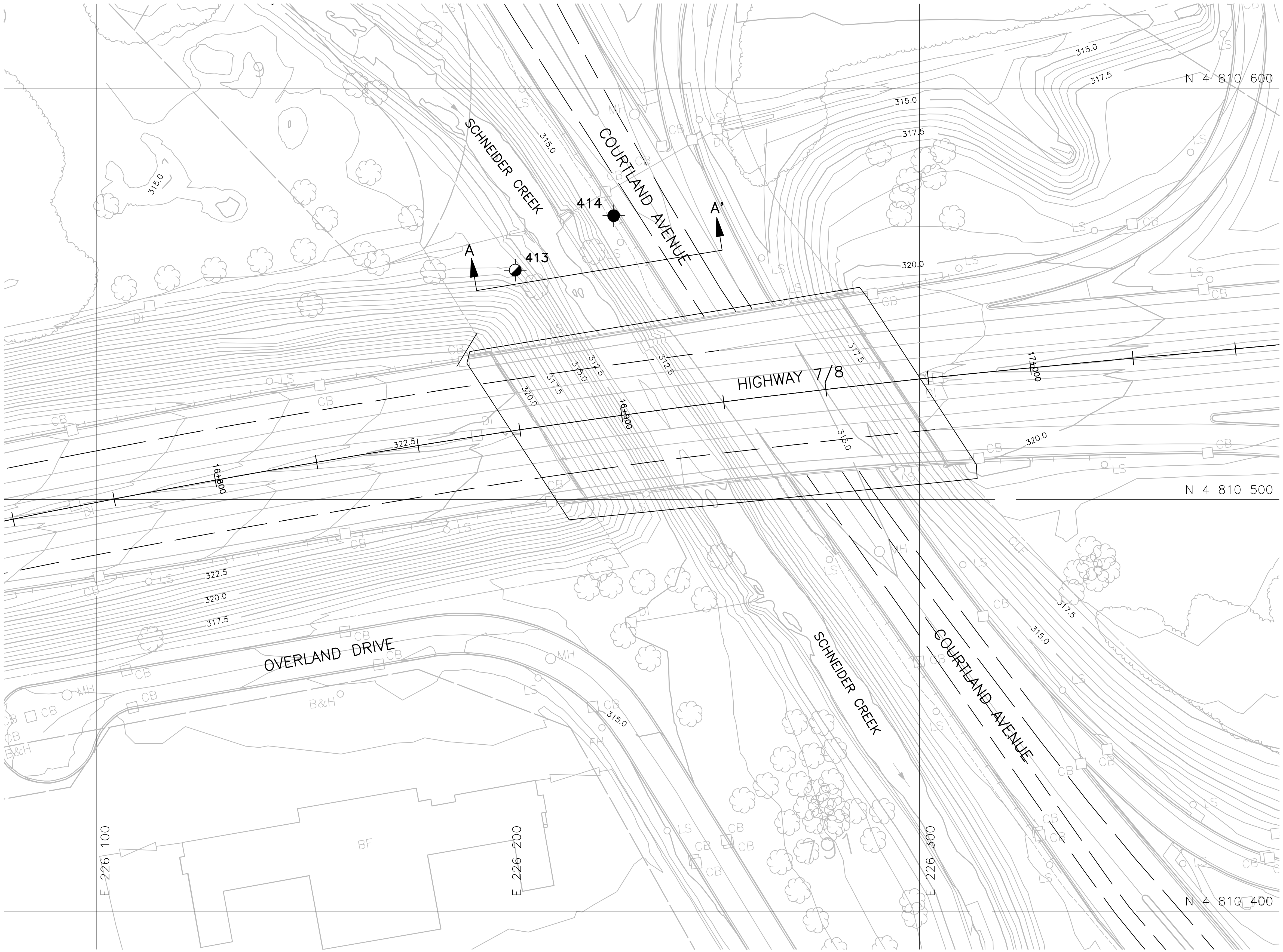
CONT No.
WP No. 131–98–00

SHEET

TEMPORARY BRIDGE AT SCHNEIDER CREEK

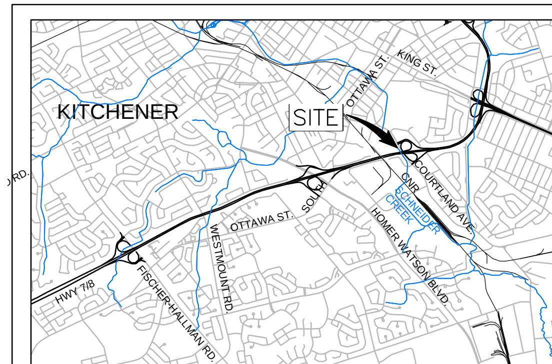
HIGHWAY 7/8 WIDENING
BOREHOLE LOCATIONS

Golder Associates
Golder Associates Ltd.
LONDON, ONTARIO, CANADA



PLAN

SCALE
10 0 10 m



KEY PLAN

SCALE IN KILOMETRES
0 1 2

LEGEND

Borehole – Current Investigation

Borehole – (Geocres No. 40P8–176)

No.	ELEVATION	CO–ORDINATES (MTM ZONE 10) NORTHING	EASTING
414	315.43	4 810 569.0	226 225.7
(Geocres No. 40P8–176)			
413	313.98	4 810 555.8	226 201.8

NOTES

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

REFERENCE

Base plans provided in digital format by Dillon Consulting.

NO.	DATE	BY	REVISION

Geocres No. 40P8–194

HWY. 7/8		PROJECT NO. 08–1132–084–1	DIST.
SUBM'D. ML	CHKD.	DATE: Feb. 18/11	SITE: 33–224
DRAWN: LMK/AMG	CHKD.	APPD.	DWG. 1

PLOT DATE: February 18, 2011
FILENAME: N:\active\2008\1132 – Geotechnical\1132–000–0\08–1132–084–1 DILLON – GWP 131–98–00 FDNS – HWY 7–8\Drafting\AutoCAD files\0811320841–D14001.dwg

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

CONT No.
WP No. 131–98–00

TEMPORARY BRIDGE AT SCHNEIDER CREEK
HIGHWAY 7/8 WIDENING
SOIL STRATA

SHEET

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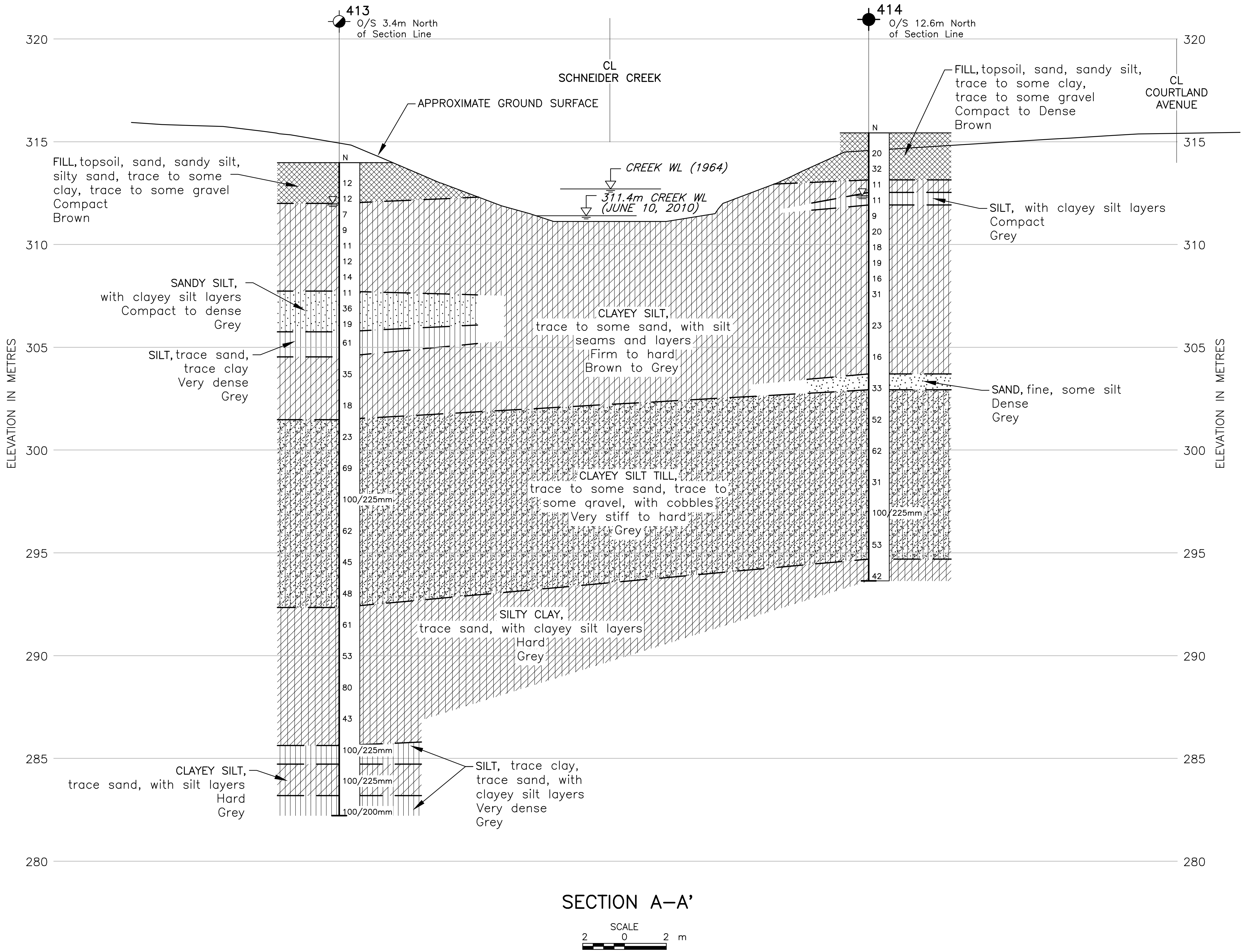
Golder Associates Ltd.
LONDON, ONTARIO, CANADA



SECTION A–A'
SCALE
2 0 2 m

LEGEND

- Borehole – Current Investigation
- Borehole – (Geocres No. 40P8–176)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- DRY Borehole dry during drilling
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES (MTM ZONE 10) NORTHING	EASTING
414	315.43	4 810 569.0	226 225.7
(Geocres No. 40P8–176)			
413	313.98	4 810 555.8	226 201.8

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 Dillon Consulting.

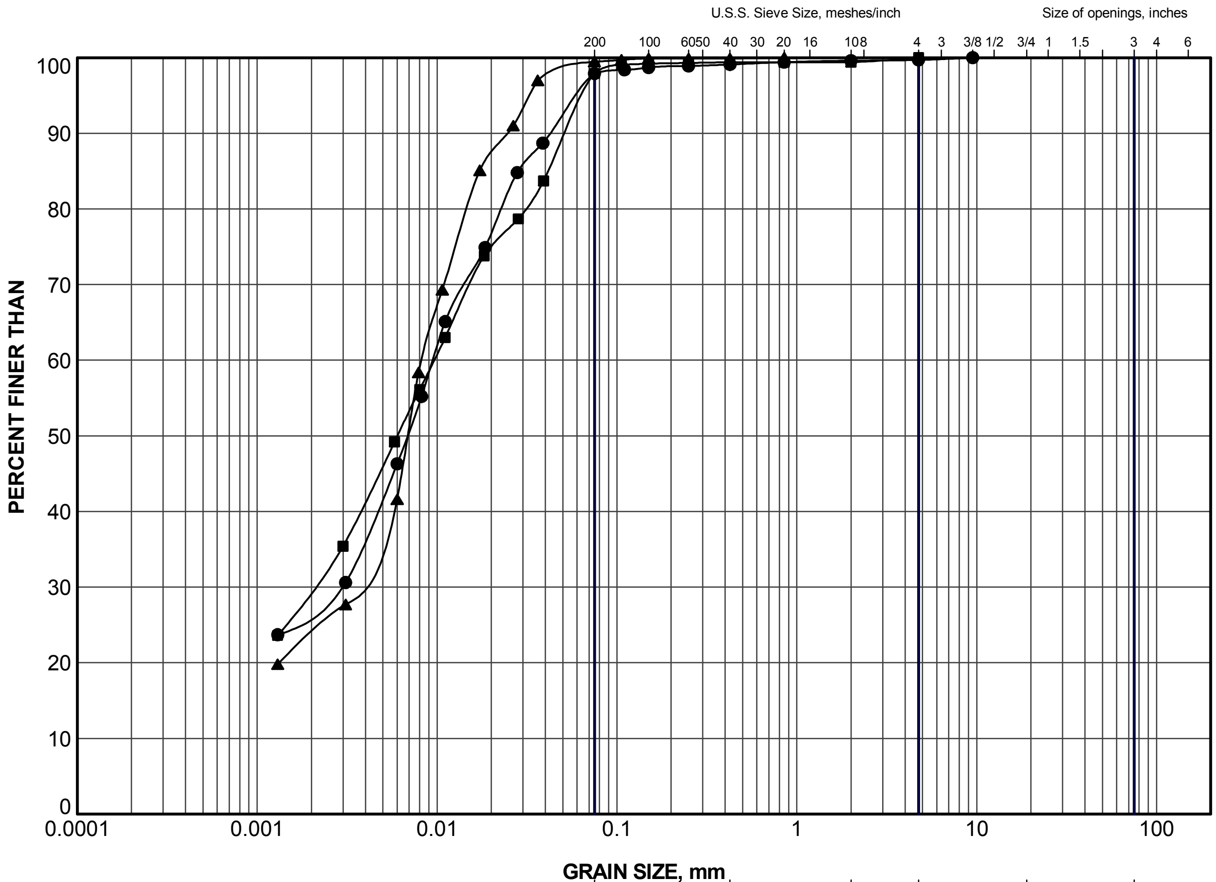
NO.	DATE	BY	REVISION

Geocres No. 40P8–194

HWY. 7/8		PROJECT NO. 08–1132–084–1	DIST.
SUBM'D. ML	CHKD.	DATE: Feb. 18/11	SITE: 33–224
DRAWN: LMK/AMG	CHKD.	APPD.	DWG. 2

APPENDIX A

Laboratory Test Data



<table>
<tr><td rowspan="2">CLAY AND SILT</td><td>fine</td><td>medium</td><td>coarse</td><td>fine</td><td>coarse</td><td rowspan="2">Cobble Size</td></tr>
<tr><td colspan="3">SAND SIZE</td><td colspan="2">GRAVEL SIZE</td></tr>
</table>

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	413	6	309.2
■	414	8	309.1
▲	414	11	306.1

PROJECT
TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

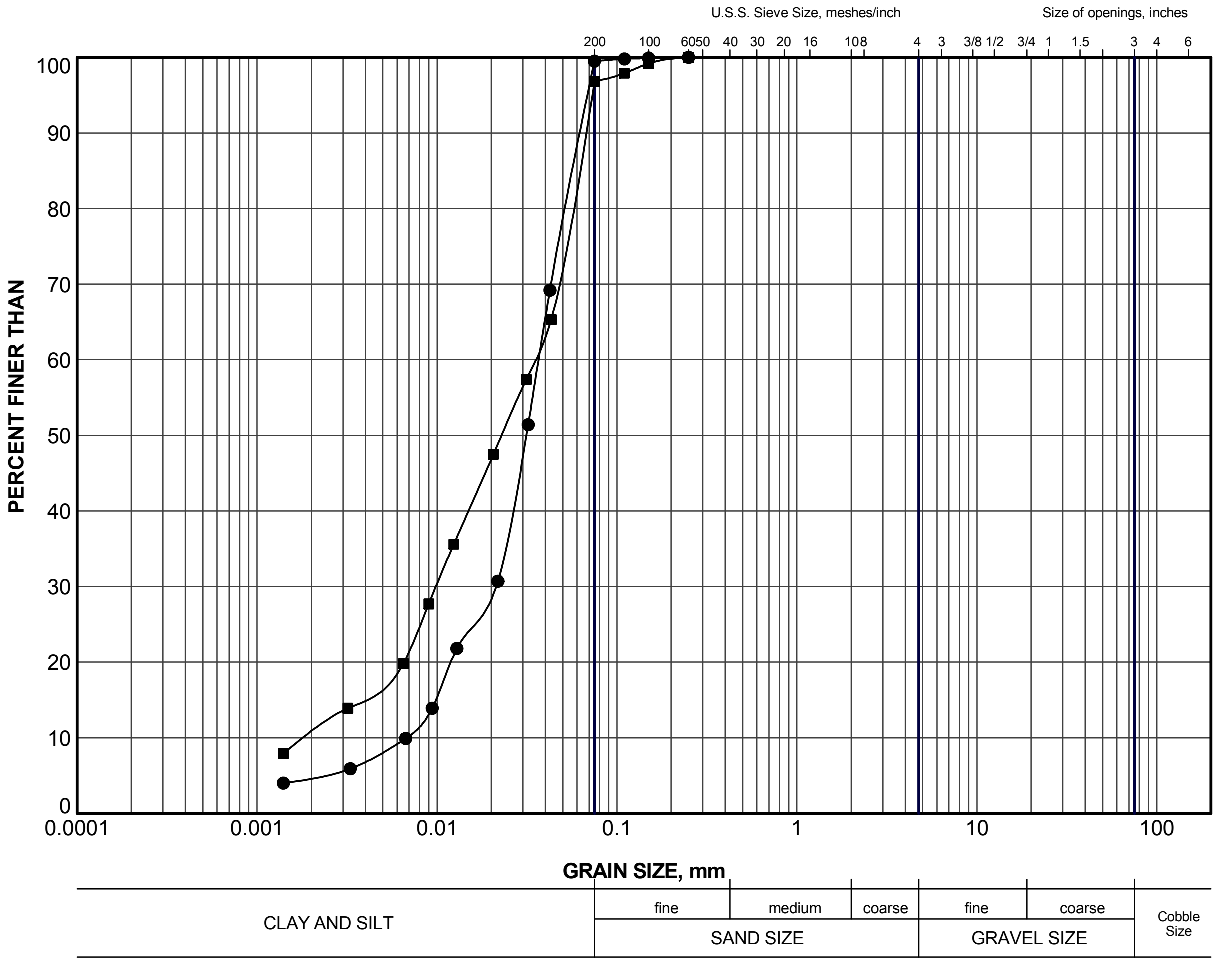
GRAIN SIZE DISTRIBUTION

CLAYEY SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.		08-1132-084-1	FILE No.	0811320841-F140A1	
			SCALE	N/A	REV.
DRAWN	LMK	Nov. 08/10	**FIGURE A-1**		
CHECK					

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	413	11	305.2
■	413	24	285.4

PROJECT
TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

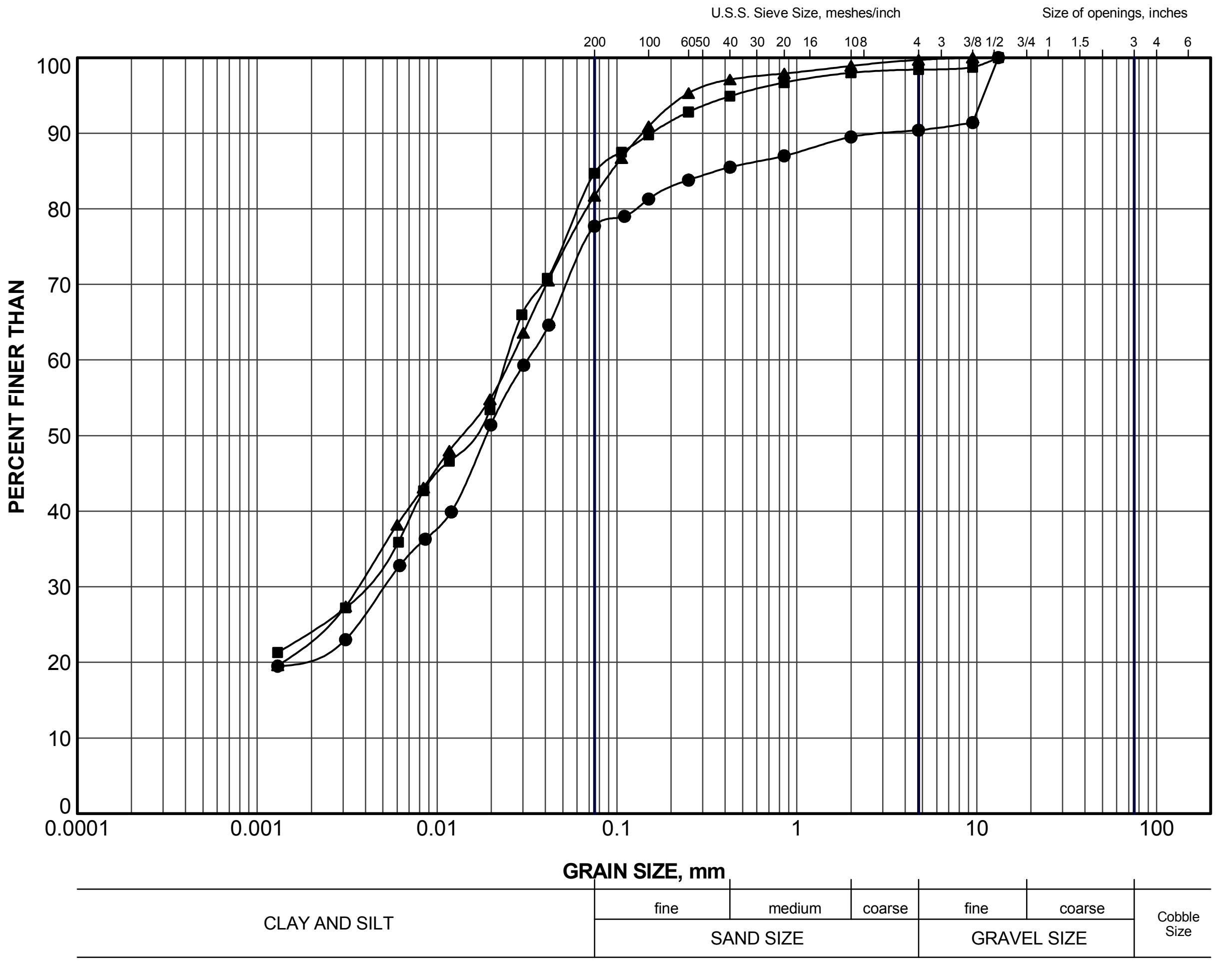
GRAIN SIZE DISTRIBUTION

SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-084-1		FILE No.	0811320841-F14A02
			SCALE N/A	REV.
DRAWN	LMK	Nov. 08/10	**FIGURE A-2**	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	413	16	297.6
■	414	14	301.5
▲	414	17	297.0

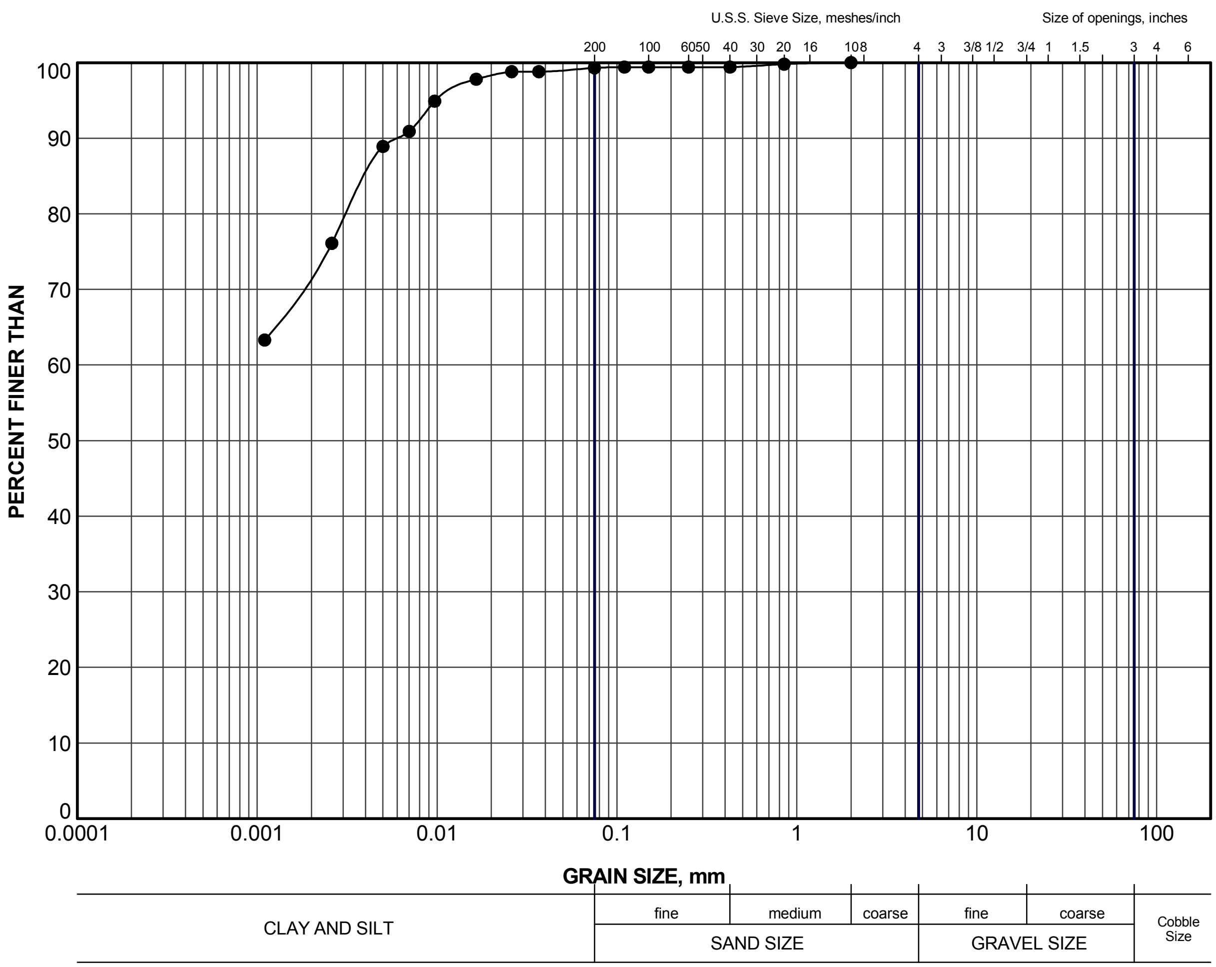
PROJECT
TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-084-1		FILE No.	0811320841-F140A3
			SCALE N/A	REV.
DRAWN	LMK	Nov. 08/10	FIGURE A-3	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	413	20	291.5

PROJECT

TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

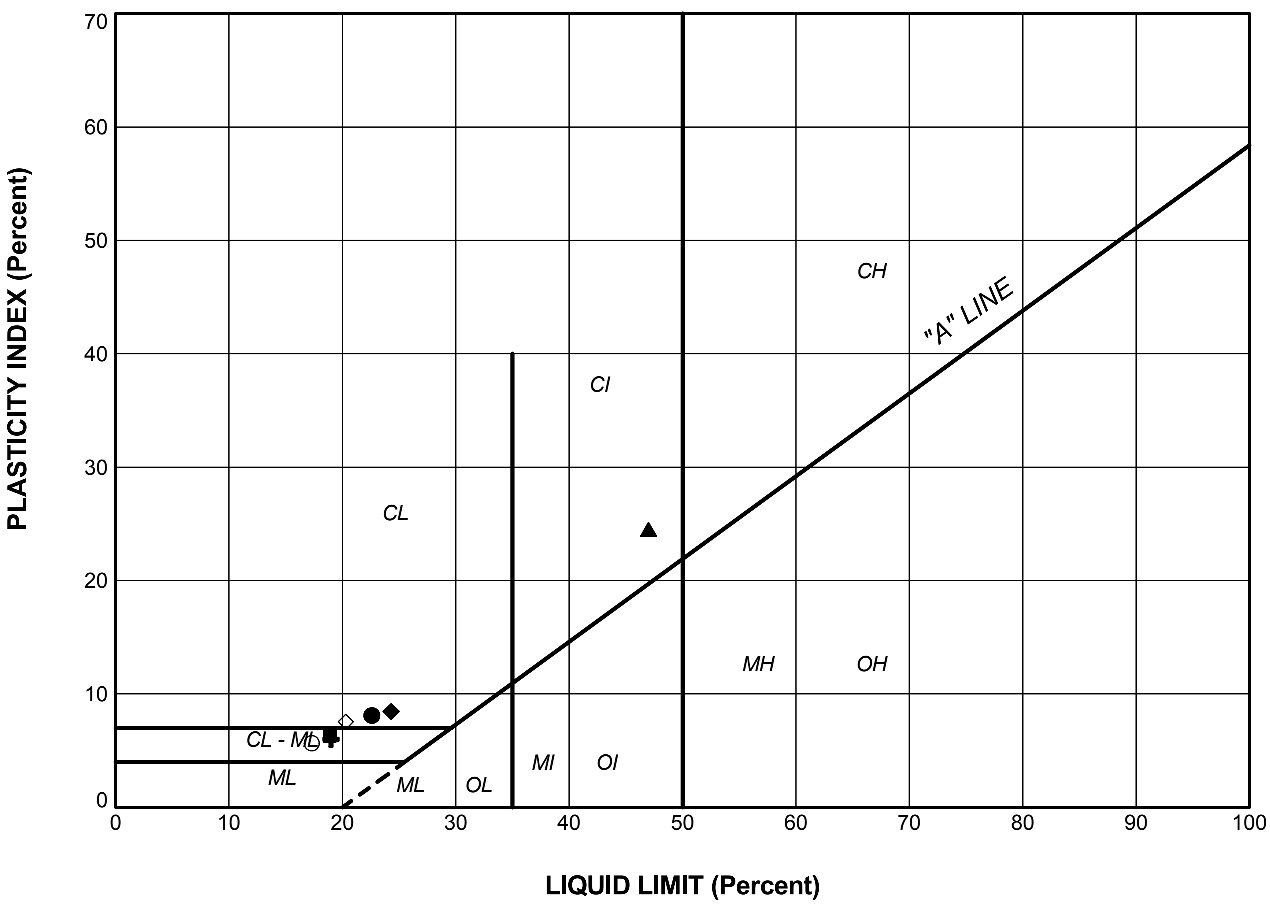
GRAIN SIZE DISTRIBUTION

SILTY CLAY

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-084-1		FILE No.	0811320841-F140A4
			SCALE N/A	REV.
DRAWN	LMK	Nov. 08/10	FIGURE A-4	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



SOIL TYPE	**PLASTICITY**
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
SILTY CLAY					
▲	413	20	47.0	22.5	24.5
CLAYEY SILT					
●	413	6	22.6	14.5	8.1
✚	414	8	19.0	13.0	6.0
◆	414	11	24.3	15.9	8.4
CLAYEY SILT TILL					
■	413	16	18.9	12.6	6.3
◇	414	14	20.3	12.8	7.5
○	414	17	17.3	11.7	5.6

PROJECT
TEMPORARY BRIDGE AT SCHNEIDER CREEK
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	08-1132-084-1		FILE No.	0811320841-F14A05
			SCALE N/A	REV.
DRAWN	LMK	Nov. 08/10	**FIGURE A-5**	
CHECK				

MTO_PI GLDR_LDN.GDT

APPENDIX B

Record of Borehole from GWP 131-98-00 - Highway 7/8 Courtland Avenue Overpass (Site No. 33-224) (Geocres No. 40P8-176)

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 413

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4810555.8 ;E 226201.8

ORIGINATED BY DB

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / ROTARY DRILLING

COMPILED BY LMK

DATUM GEODETIC

DATE March 25, 2009 - March 26, 2009

CHECKED BY

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
313.98	GROUND SURFACE
0.00	FILL, topsoil, silty Brown
0.15	FILL, sandy silt, trace clay with fine sand layers, trace gravel Compact Brown
312.61 1.37	FILL, silty sand, some gravel, with silt layers Compact Brown
312.00 1.98	CLAYEY SILT, trace sand, with silt layers Firm to stiff Grey
307.73 6.25	SANDY SILT, with clayey silt layers Compact to dense Grey
305.75 8.23	SILT, trace sand, trace clay Very dense Grey
304.53 9.45	CLAYEY SILT, with silt seams and layers Very stiff to hard Grey
301.48 12.50	CLAYEY SILT TILL, trace sand, trace to some gravel Very stiff to hard Grey

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	12	
2	SS	12	
3	SS	7	
4	SS	9	
5	SS	11	
6	SS	12	0 2 72 26
7	SS	14	
8	SS	11	
9	SS	36	
10	SS	19	
11	SS	61	0 1 94 5
12	SS	35	
13	SS	18	
14	SS	23	
15	SS	69	

GROUND WATER CONDITIONS: ∇ (at approx. elev. 312)

ELEVATION SCALE: 313, 312, 311, 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 413

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810555.8 ;E 226201.8
ORIGINATED BY DB
DIST HWY 7/8
BOREHOLE TYPE POWER AUGER / ROTARY DRILLING
COMPILED BY LMK
DATUM GEODETIC
DATE March 25, 2009 - March 26, 2009
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT TILL, trace sand, trace to some gravel Very stiff to hard Grey				298		
		16	SS	100/ 225mm			10 12 58 20
					297		
		17	SS	62	296		
					295		
		18	SS	45	294		
		19	SS	48	293		
292.34 21.64	SILTY CLAY, trace sand, with clayey silt layers Hard Grey				292		
		20	SS	61	291	47	0 1 28 71
		21	SS	53	290		
					289		
		22	SS	80	288		
		23	SS	43	287		
					286		
285.63 28.35	SILT, trace clay, trace sand, with clayey silt layers Very dense Grey	24	SS	100/ 225mm	285		0 3 87 10
284.72 29.26	CLAYEY SILT, trace sand, with silt layers Hard Grey				284		

Continued Next Page

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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 413

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810555.8 ;E 226201.8
ORIGINATED BY DB
DIST
HWY 7/8
BOREHOLE TYPE POWER AUGER / ROTARY DRILLING
COMPILED BY LMK
DATUM GEODETIC
DATE March 25, 2009 - March 26, 2009
CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace sand, with silt layers Hard Grey		25	SS	100/ 225mm						
283.19 30.79	SILT, trace clay, trace sand Very dense Grey						283				
			26	SS	100/ 200mm						
282.22 31.76	END OF BOREHOLE Groundwater encountered at about elev. 312.0m during drilling on March 25, 2009.										

LDN_MTO_06 08-1132-084-1.GPJ LDN_MTO.GDT 18/02/11

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

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