April 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

North Saugeen River Culvert Rehabilitation
Site No. 8-342/C, Highway 6, Williamsford
Contract 4 Structure Replacements and Rehabilitation
GWP 3042-11-00
Ministry of Transportation, Ontario - West Region

Submitted to:

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REPORT

Report Number: 12-1132-0163-4000-R02

Geocres No.: 41A-240

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FOUNDATION INVESTIGATION REPORT

NORTH SAUGEEN RIVER CULVERT REHABILITATION
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CONTRACT 4 STRUCTURE REPLACEMENTS AND REHABILITATION
GWP 3042-11-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 3042-11-00. The project involves the detailed design of the replacement or rehabilitation of several structures along multiple highways in southern Ontario. This report addresses the proposed rehabilitation of the twin culverts at the North Saugeen River in Williamsford (Site 8-342/C) at Stations 17+532 and 17+550 on Highway 6 in the Geographic Township of Holland in Grey County. The various reference documents indicate the structure as a bridge. However, the structure will be referenced as a culvert for the purpose of this report.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed twin culverts rehabilitation by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P2-1132-0163 dated February 25, 2013. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated March 26, 2013.

Stantec Consulting Ltd. (Stantec) provided Golder Associates with preliminary drawings for the proposed rehabilitation works in digital format.

2.0 SITE DESCRIPTION

The subject culverts are located at about Stations 17+532 (south) and 17+550 (north) in the Community of Williamsford on Highway 6, approximately 90 metres north of Chatsworth Road 24 in the Geographic Township of Holland in Grey County, Ontario. The Community of Chatsworth is approximately 9 kilometres north of the site. The approximate location of the site is shown on the Key Plan, Figure 1.

This section of Highway 6 is currently a two lane, undivided highway with paved shoulders. It is generally oriented north-south in the vicinity of the site and has an asphalt riding surface at about elevation 322 metres. The North Saugeen River (formerly Hamilton Creek) flows through the culverts from east to west beneath Highway 6. The approximate stream bed elevations at the north and south outlets are 318.6 and 318.0 metres, respectively. The banks of the North Saugeen River and the embankments near the culverts are grass covered. The Mill Pond is located east of the spillways, which is fed by the North Saugeen River. In March 2016, this section of Highway 6 was closed after heavy rains caused flooding and inundation of several nearby buildings.

The existing culverts were constructed in 1932 and were subsequently removed and extended to the west in 1987. The original structures are comprised of the concrete non-rigid frame box culverts (NRFB) now about 4.76 metres long and the extensions (to the west) consist of concrete rigid frame box culverts (RFB) approximately 10 metres long. The culverts have a skew of 89.5 degrees to the centreline of Highway 6. Retaining walls ‘A’, ‘B’ and ‘C’ are located north, between and south of the culverts, respectively, on the west side of Highway 6. The retaining walls have a top of footing elevation of 317.25 metres.

A dam owned by Ontario Ministry of Infrastructure is present immediately upstream (east) of the culverts. It is understood that this dam was previously used as part of the former operations of the Williamsford Mill southwest of the site. Although the mill is now defunct, the dam is still operational. The dam consists of a pair of concrete weirs at the ends of spillways leading to the inlet of each culvert.

Structure	Dimensions (m)			Obvert Elevation (m)		Construction
	Width[3]	Height[3]	Length	Lt[1]	Rt[1]	
North Culvert	5.50	2.70	14.74	321.29	321.66	Concrete NRFB/RFB
South Culvert	5.08	2.70	Note 2	321.43	Note 2	Concrete NRFB/RFB

NOTE: 1. When facing the direction of increasing chainage, Lt and Rt are defined as Left and Right of centreline, respectively.

2. Information not provided.

3. Width and height dimension provided pertain to outlet.

Land use in the area of the site is mixed residential and commercial, with some small businesses and residents adjacent to the site. Site photographs are provided in Appendix B.

2.1 Site Geology

The site is located within the Horseshoe Moraines physiographic region. This region is characterized by irregular, stony knobs and ridges which are composed mostly of till with some sand and gravel deposits, sand and gravel terraces and swampy valley floors.[1] The overburden in the area of the site generally consists of glaciofluvial outwash deposits mainly consisting of gravel and sand.[2]

The geological mapping indicates that the underlying bedrock consists of sandstone, shale, dolostone and siltstone of the Guelph Formation of Lower Silurian age.[3] The bedrock surface at the site is at about elevation 311 metres based on information provided in Geocres Report No. 41A-157, with the overburden thickness being about 11 metres (see Appendix C).

3.0 INVESTIGATION PROCEDURES

The initial phase of the field work for the investigation was carried out on May 31 and June 1, 2016, during which time three boreholes, BH-201E, BH-202 and BH-203, were drilled using an all-terrain vehicle mounted CME 55 drilling rig supplied and operated by a specialist drilling contractor. Subsequently, BH-204B was drilled on July 6, 2016 using a Geoprobe 7822DT. A smaller drill rig was mobilized to the site to access the BH-204B location. The locations of the boreholes are shown on the Borehole Location Plan, Drawing 1.

Several attempts were made to drill a borehole between the two culverts, then immediately south of the south culvert. At each location, reinforced concrete approach slabs were encountered which the drilling equipment was unable to penetrate. The resulting successful borehole, BH-201E, was drilled about 0.5 metres south of south approach slab. Borehole BH-203 was later drilled between the two culverts after specialized coring equipment was mobilized to the site. In the northeast quadrant of the site, north of the north culvert, the first attempt of borehole BH-204 resulted in refusal at a depth of about 0.5 metres on cobbles. A second attempt was successful about 0.5 metres further north.

Samples of the overburden were typically obtained at depth intervals of 0.75 metres using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The recorded SPT N values are noted on the Record of Borehole sheets and shown on Drawing 1. 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 after an initial 150 millimetres of penetration. The results of the SPT testing as presented on the Record of Borehole sheets, Drawing 1 and in Section 5.0 of this report are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation 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 particle sizes, including cobbles and boulders, are known to be present in the soils as discussed in the text of this report.

[1] Chapman, L.J., and Putnam, D.F., 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p.

[2] Ontario Geological Survey 2000. Quaternary Geology, seamless coverage of the Province of Ontario; Ontario Geological Survey, Data Set 14---Revised.

[3] Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in borehole 202 as indicated on the corresponding Record of Borehole sheet. 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 an experienced member of our staff who 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. 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 results of the 2016 investigation carried out by Golder was supplemented with information from Geocres Report No. 41A-157 titled "Foundation Investigation Report for Hamilton Creek Crossing, Hwy. #6; Town of Williamsford, W.P. 123-83-01; Site 8-159-342, District #5 (Owen Sound)". Relevant information from Geocres Report No. 41A-157 is presented in Appendix C.

The as-drilled borehole locations and ground surface elevations are shown on the Record of Borehole sheets and on Drawing 1. The table below summarizes the coordinates, ground surface elevations and depths of the boreholes.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
201E	4 916 338	195 494	322.03	8.08
202	4 916 339	195 500	322.01	8.08
203	4 916 353	195 491	321.90	8.08
204B	4 916 374	195 500	322.02	8.08

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

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

The boreholes drilled at the site generally encountered the existing pavement structure or surficial topsoil overlying fill materials and native granular soils. The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, 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 in the following sections.

4.2 Soil Conditions

Asphaltic concrete pavement (asphalt) was encountered at the pavement surface in BH-201E, BH-202 and BH-203. The asphalt ranged in thickness from about 180 to 210 millimetres at the borehole locations. Sand and crushed gravel material, interpreted to be granular base materials based on visual and textural examination, was encountered beneath the asphalt in BH-201E and BH-202. The granular base was about 340 millimetres thick in both boreholes. Concrete was encountered beneath the asphalt in BH-203 and was 250 millimetres thick. The concrete contained reinforcing steel. Concrete approach slabs are inferred to be present between the culverts.

Topsoil was encountered in BH-204B at the ground surface and was about 90 millimetres thick. Variable amounts of topsoil, organics and wood pieces were also encountered in some of the fill layers in all of the boreholes and in the silty sand and gravel layer in borehole BH-203. 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.

Variable layers of fill were encountered beneath the pavement structure in BH-201E and BH-202 at elevation 321.5 metres, beneath the concrete in BH-203 at elevation 321.5 metres and beneath the topsoil in BH-204B at elevation 321.9 metres. The thicknesses of the fill ranged from about 2.5 to 3.9 metres. The fill was generally granular in nature and ranged in gradation from sandy silt to sand and gravel. Variable amounts of topsoil were encountered in the fill layers in BH-201E, BH-202 and BH-204B. Wood pieces and organics were also encountered in the fill in BH-201E. Cobbles were inferred based on drilling resistance and should be expected in the fill layers. Standard penetration test N values ranged from 5 to 48 blows per 0.3 metres and water contents of the fill materials ranged from about 6 to 30 per cent. Grain sizes analyses carried out on samples of fill are presented on Figure A-1 in Appendix A.

Granular deposits were encountered beneath the fill in all of the boreholes. The granular deposits consisted of sand, silty sand and gravel, as well as sand and gravel. A 0.8 metre thick layer of very loose sand was encountered beneath the fill at elevation 317.6 metres in BH-201E. A standard penetration test N value of 2 blows per 0.3 metres was recorded in the sand. Loose to very dense sand and gravel was encountered beneath the sand in BH-201E and the fill in BH-202 to BH-204B at elevations ranging from 316.9 to 319.0 metres. All of the boreholes were terminated in the sand and gravel after exploring it for about 2.8 to 5.2 metres. The sand and gravel layer in borehole BH-203 was interlayered with a 0.8 metre thick layer of silty sand and gravel at elevation 317.5 metres. N values in the sand and gravel ranged from ranged from 8 to 103 blows per 0.3 metres of penetration. Water contents for the granular deposits ranged from 8 to 15 per cent. Grain sizes analyses carried out on samples of the sand and gravel are presented on Figure A-2 in Appendix A.

The borehole logs, laboratory testing, rock core descriptions and stratigraphic profile from Geocres Report No. 41A-157 are provided in Appendix C. The subsurface conditions encountered in the native materials during the current investigation are consistent with those shown on the borehole logs from Geocres Report No. 41A-157. Cobbles and boulders were present in the granular deposits during the current investigation and the presence of frequent boulders was noted on the borehole logs from Geocres Report No. 41A-157. Based on the information provided on the borehole logs, dolostone bedrock was encountered at about elevation 311 metres or at about 11 metres depth.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation piezometer was installed in BH-202. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in BH-201E, BH-202 and BH-204B during drilling. Due to the method of drilling used to advance BH-203 (wash bore/mud rotary), the groundwater level could not be determined. Boreholes BH-201E, BH-202 and BH-204B encountered groundwater at depths of about 4.4 metres or at about elevation 317.6 metres. A summary of the encountered and measured groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)	
			June 1, 2016	July 6, 2016
201E	322.03	317.6	-	-
202	322.01	317.6	317.39	317.59
203	321.90	-*	-	-
204B	322.02	317.6	-	-

*Water level could not be determined due to drilling method

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. On June 1, 2016, the water level in the piezometer installed in borehole BH-202 was about 4.6 metres below ground surface or at about elevation 317.4 metres and, on July 6, 2016, was about 4.4 metres below ground surface or at about elevation 317.6 metres. The upstream water level was measured at elevation 320.88 and 321.13 metres on June 1 and July 16, 2016, and the downstream water level was measured at elevation 319.11 and 319.16 metres on June 1 and July 16, 2016, respectively.

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

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Lantech Drilling Inc. and Strata Drilling Group, both Ontario Ministry of Environment and Climate Change licensed well contractors. The field operations were supervised by Mr. Daniel Hyland, E.I.T. under the direction of the Field Investigation Manager, Mr. Brett Thorner, P.Eng. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Michael Arthur. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Mr. Daniel Hyland, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. Michael E. Beadle, P.Eng., an Associate with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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PART B

FOUNDATION DESIGN REPORT

NORTH SAUGEEN RIVER CULVERT REHABILITATION
SITE NO. 8-342/C, HIGHWAY 6, WILLIAMSFORD
CONTRACT 4 STRUCTURE REPLACEMENTS AND REHABILITATION
GWP 3042-11-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION

6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the design of the proposed twin culvert rehabilitation at the North Saugeen River Bridge in Williamsford (Site 8-342/C) at Stations 17+532 and 17+550 on Highway 6 in the Geographic Township of Holland in Grey County, Ontario.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed foundations. As such, where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing culverts are 14.74 metre long concrete structures with 2.7 metre high openings. The north cell has a 5.50 metre span and the south cell has a 5.08 metre span. The north cell has an approximate obvert elevation of 321.29 metres and the south cell has an approximate obvert elevation of 321.43 metres at the outlet. Invert elevations for the north and south cell at the outlet are 318.57 and 318.70 metres, respectively. The original (east) structures are non-rigid frame boxes (NRFB) about 4.8 metres long constructed in 1932. A portion of the original structures were subsequently removed in 1987 and 10.05 metre extensions were added on the west side. The extensions are rigid frame box (RFB) structures. Existing retaining walls ‘A’, ‘B’ and ‘C’, shown on Drawing 1, have an underside of footing elevation of 316.55 metres and are up to about 3.3 metres high measured from the stream bed elevation to the road surface.

It is understood that the proposed rehabilitation will consist of waterproofing the top slab and repairing cracks and other defects in the concrete from inside of the culverts. Repair of Retaining Wall ‘B’, which spans between the two cells, is also being considered. This report documents the analyses for this retaining wall and outlines rehabilitation alternatives for discussion by the TPM Consultant and MTO. Based on the provided information, it is understood that Option 3 (described in Section 6.2 below) will be implemented, together with replacement of the sidewalk adjacent to Retaining Wall ‘B’.

6.1 Sidewalk Settlement and Movement of Retaining Wall ‘B’

The Ontario Bridge Management System (OBMS) inspection report dated June 2006 noted an outward leaning of Retaining Wall ‘B’ by about 15 millimetres. OBMS inspections dated August 2008 and 2010 noted an outward leaning of approximately 20 millimetres at the north end and 15 millimetres at the south end. Settlement of the adjacent sidewalk has also been recorded since 2002. The 2006, 2008 and 2010 OBMS inspection reports noted 25 millimetres of settlement of the sidewalk adjacent to Retaining Wall ‘B’. The settlement and outward leaning of about 25 millimetres was also observed at the time the field work was carried out for this report. The settlement of the sidewalk is likely attributable to the outward rotation of the retaining wall. Based on the borehole drilled adjacent to the settled sidewalk, our other nearby boreholes and our analysis of the retaining wall, it is considered likely that the retaining wall was not originally designed to accommodate elevated water table conditions such as those caused by flooding conditions. Based on our analysis, the factor of safety for resisting overturning when the groundwater level is at or just below Highway 6 pavement surface is about 1.1 and 1.3 for the north and south end, respectively. If the groundwater level in the analysis is lowered by about one metre (to about elevation 321 metres), the factor of safety is about 1.8. Generally, a factor of safety of at least 2.0 is desirable when designing for overturning resistance. The Factor of Safety with respect to sliding reduces to approximately 1.1 when the groundwater level is near the top of the wall with the downstream water level near elevation 319.1 metres. It is likely that the retaining wall performed adequately until such time that elevated groundwater levels occurred during a flooding event, at which point movement likely would have first occurred. Additional possible causes for the rotation are listed below.

- Poor quality backfill behind the wall. Variable amounts of silt, topsoil, organics and wood pieces were encountered in the boreholes. These materials may have a lower angle of friction than contemplated during design which would result in an increase in active earth pressures.
- Inhibited drainage. Although the backfill is generally free draining, silty zones were noted. Also, there is some concern about the drain/weep hole detailing as discussed in Section 6.2. A significant buildup of hydraulic pressures, even in the short-term, behind the wall decreases the factor of safety against overturning and sliding.
- Scour. During the flood conditions, scour of the soil below the retaining wall footings can take place.

6.2 Rehabilitation Alternatives

Potential methods of rehabilitating Retaining Wall 'B' are:

1) Constructing a buttress on the downstream side of the wall.
2) Connecting the retaining wall to the adjacent culverts with dowels.
3) Improving drainage of the backfill by constructing additional wall drains.
4) Do Nothing – monitor wall
5) Anchoring the retaining wall with tie-backs.
6) Partial or full replacement of the wall.

Alternative 1 – Buttress

A buttress can be formed by either increasing the thickness of the lower portion of Retaining Wall 'B' or by adding a structure at the mid-point of the wall on the spit of land between the culverts. With either option, the thickness of the additional structure must be greater at the base. The base elevation should be sufficiently deep to avoid scour. It should then taper to a height such that the resisting moments are increased to provide a Factor of Safety against overturning of 2.0 or greater under flooding conditions. Shear transfer at the interface may be provided by dowels drilled into the existing wall. Coring the existing walls is suggested to confirm that the existing concrete can support the additional loads.

Alternative 2 – Connect to Adjacent Culverts

Notches could be drilled into the walls of the adjacent culverts and retaining structure, dowels inserted and the holes filled with concrete. A structural engineering assessment should be made to confirm whether this solution is viable and whether the existing wall can accommodate the resulting stresses.

Alternative 3 – Install Additional Wall Drains

Additional weep holes or wall drains could be added at or above the level of the existing wall drains by coring through the wall. This could be accomplished by coring a hole larger than the drain size to permit installation of casing or drive point. Alternatively, if a drive point is used, it should have a slot size selected to be filler compactible with the backfill. Clear stone wrapped in a non-woven geotextile can be inserted into the rear of the hole to form a drain in accordance with Ontario Provincial Standard Drawing (OPSD) 3190.100. The hole can then be backfilled with non-shrink grout after insertion of a 75 millimetre diameter casing or drive point. The drain should be inclined to promote drainage away from the backfill. Grouting operations should be carefully monitored to avoid blockage of the slots of the drive points or any drainage material. Use of drive points is preferred.

It should be noted that during review of Contract No. 87-47 Drawing Sheet 18 – Retaining Walls, dated April 1985, the details show the drain pipes being 50 millimetres in diameter and sloping down into the backfill instead of horizontal or sloping away from the backfill. If the wall drains were constructed in this manner, the lower portion of the backfill could remain saturated much longer than if the wall drains were properly detailed. Also, no geotextile wrap or filter was indicated for the 20 millimetre diameter clear stone placed for the drain. Given the presence of sandy silt layers within the backfill, the weep holes may be clogged. These two conditions would contribute to increased hydrostatic pressure on the retaining wall.

Alternative 4 – Do Nothing

The wall has moved outwards some 0.2 to 0.5 per cent of the wall height since it was constructed in 1987. This magnitude of movement is considered to be relatively minor. Provided the MTO is prepared to accept the risk of future displacement as a result of heavy precipitation events and flooding, no rehabilitation work could be done at this time. If this option is selected, it would be prudent to establish survey points at the top of the wall and various locations along the face of Retaining Wall ‘B’ to monitor the rate of displacement with time. Monitoring should be done quarterly and weekly for up to one month following heavy precipitation events which increase the water level to or above the high water level.

Alternative 5 – Anchor Wall with Tie-Backs

The wall could be anchored with tie-backs. However drilling for the tie-backs will be difficult due to the presence of cobbles and dense granular zones within the backfill. Further there is likely insufficient distance between the opposing walls to extend the tie-back beyond the active zone of Retaining Wall ‘B’. The wall would have to be anchored with tensioned tie-rods using face plates at each wall. This is considered the second most costly alternative. It also carries the risk that drilling may be unsuccessful due to obstructions and that the actual site geometry may not be appropriate for development of the required resistance.

Alternative 6 – Full or Partial Replacement of Wall

The inspection reports indicate that the components of culvert Site 8-342/C, including Retaining Wall ‘B’ are in generally good condition. Therefore, partial or full replacement of Retaining Wall ‘B’ is not considered to be economically feasible or warranted at this time.

Comparison of Alternatives

Review of the 2006, 2008 and 2010 OBMS inspection reports revealed that no additional settlement occurred in the sidewalk area adjacent to Retaining Wall ‘B’ during the period 2006 to 2010. Similarly, additional rotation of the wall did not appear to occur during this period. Based on the observations made during our 2016 field investigation, the wall has rotated outward some 5 millimetres in the last six years. Our analyses have indicated that Retaining Wall ‘B’ is stable during normal water levels but achieves marginal stability for overturning and sliding during flood conditions if the weirs of the Mill Pond Dam are overtopped and the water level increases within the backfill to within 1 metre of the pavement surface. The water level and marginally stable condition may remain for some time after the flood waters subside.

Alternatives 5 and 6 are costly and will require interruption of traffic and in-water work to implement. Given that Retaining Wall ‘B’ is in relatively good condition and rotation of the wall is considered minor, these two options will not be discussed further. Alternatives 1 to 4 are considered to be the most practical and economically feasible for consideration for Detailed Design. Alternative 3 can be implemented on its own or in combination with Options 1 and 2. A comparison of these four alternatives is presented in Table I. Costs are relative and are in comparison to the lowest cost alternative, Option 4. The risk rating is relative to the risk associated with Option 4 and considers the potential for continued rotation of Retaining Wall ‘B’. Based on the provided information, it is understood that Option 3 will be implemented, along with replacement of the sidewalk adjacent to Retaining Wall ‘B’.

Table 1: Comparison of Rehabilitation Alternatives for Retaining Wall 'B'

Option	Advantages	Disadvantages	Relative Cost	Risks/ Consequences
1 – Buttress	*Ensures stability at all water level conditions behind the wall	* Cofferdam required for in-water work. * Foundations must extend below depth of scour	Medium to High	*Low/Footing may be undermined due to scour if not adequately protected
2 – Connect to adjacent culverts (Preferred technical solution)	*Ensures stability at all water level conditions behind the wall * Work can proceed using a platform depending on proposed dowel locations	*If dowels are improperly designed, rotation may still occur during/after heavy precipitation events. *Bending of Retaining Wall ‘B’ to be considered.	Medium	*Low/Shear failure of dowels
3 – Add extra wall drains	* Second most economical solution * Can be implemented in conjunction with Options 1 and 2	* Coring may be difficult depending on reinforcement layout *Weep holes will only be functional once the downstream water level drops below the level of the weep hole	Low to Medium	*Moderate to High/Potential for continued rotation but less than Option 4 due to improved drainage

Option	Advantages	Disadvantages	Relative Cost	Risks/ Consequences
		*Depending on downstream water level, cofferdam may be required for in water work		
4 – Do Nothing	*Most economical solution	* Even if rotation of the wall is monitored, the situation may worsen with each subsequent heavy precipitation event	Low	*High/Potential for continued rotation

6.3 Excavations and Groundwater Control

For Option 1, excavations would extend through the existing stream bed and into the native granular deposits. Seepage volumes are expected to be such that groundwater control may not be achieved solely by the use of sumps due to the high permeability of the soils therefore proactive dewatering would be required. In addition, construction of a cofferdam would be required. The existing culvert flows would need to be diverted/piped during construction and a Permit to Take Water would likely be required. Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times.

Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical above the water level. Localized sloughing and ground movements should be expected below the water level. All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The native granular deposits would be classified a Type 2 soil above the groundwater level and Type 4 soils below the groundwater level.

Cobbles and boulders should be expected in the granular materials.

6.4 Other Observations

The 2006, 2008 and 2010 OBMS inspection reports noted that an area at the outlet of the south culvert has been scoured to a depth of approximately 400 millimetres. The depth did not increase between 2006 and 2010. However, no remarks were made on the approximate size of the scour area or if it was increasing in size with time but probably did during the flood of March 2016. This scour area should be repaired as it may impact the stability of the foundations for the south culvert and potentially Retaining Walls ‘B’ and ‘C’. Erosion protection at the outlets should be added in accordance with OPSD 810.010.

A very wide crack was noted in the north wall of the spillway at the north culvert. This crack and major spalling of the concrete walls was also noted in the 2006, 2008 and 2010 OBMS inspection reports. These cracks should be repaired.

7.0 MISCELLANEOUS

This section of the report was prepared by Mr. Daniel W. Hyland, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. Michael E. Beadle, P.Eng., and Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, who conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
D. U. PROUT
Apr 10/17
PROVINCE OF ONTARIO

Dirka U. Prout, P.Eng.
Project Engineer

Michael E. Beadle, P.Eng.
Associate

REGISTERED PROFESSIONAL ENGINEER
F.J. HEFFERNAN
Apr 10/17
PROVINCE OF ONTARIO

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

DH/DUP/MEB/FJH/CR

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r02 8-342 saugeen river\1211320163-4000-r02 apr 10 17 (final) part a&b fdns rehab clvrt 8-342.docx

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
FoS	factor of safety

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
υ	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

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	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_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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

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
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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.) drive open 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) Non-Cohesive (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.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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London, Ontario

RECORD OF BOREHOLE No 201E

METRIC

PROJECT 12-1132-0163
W.P. 3042-11-00
LOCATION N 4916338.2 , E 195493.8
ORIGINATED BY DH
DIST HWY 6
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE May 31, 2016
CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.03	PAVEMENT SURFACE						
0.00	ASPHALT						
0.21	FILL, sand and gravel, some silt, crushed Brown						
321.48 / 0.55	FILL, sand and gravel, some silt, with cobbles Dense Brown	1	SS	46			
320.66 / 1.37	FILL, silty sand and gravel Dense Brown	2	SS	33			
319.90 / 2.13	FILL, sand and gravel, some silt, trace clay, trace topsoil, with organics, wood pieces and cobbles Compact Brown	3	SS	21		~11	43 42 13 2
		4	SS	16			
		5	SS	10			
317.61 / 4.42	SAND, fine to coarse, some silt, some gravel Very loose Brown	6	SS	2	∇ (approx. elev. 317.6)		
316.85 / 5.18	SAND AND GRAVEL, trace to some silt, with cobbles and boulders Dense Brown and red	7	SS	34		~10	42 44 (14)
		8	SS	31			
		9	SS	45			
313.95 / 8.08	END OF BOREHOLE Groundwater encountered at about elev. 317.6m during drilling on May 31, 2016.						

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
ELEVATION SCALE: 322, 321, 320, 319, 318, 317, 316, 315, 314

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 202

METRIC

PROJECT 12-1132-0163
W.P. 3042-11-00
LOCATION N 4916339.2 , E 195500.0
ORIGINATED BY DH
DIST HWY 6
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE May 31, 2016
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.01	PAVEMENT SURFACE					322		
0.00	ASPHALT				Concrete			
0.18	FILL, sand and gravel, trace silt, crushed Brown							
321.49 / 0.52	FILL, sand and gravel, trace silt, with cobbles Brown							
0.76		1	SS	48	Granular bentonite	321		
320.64 / 1.37	FILL, silty sand and gravel, trace topsoil, with cobbles Dense Brown	2	SS	16			~8	53 39 (8)
319.88 / 2.13	FILL, sand and gravel, trace silt, with cobbles Compact Brown	3	SS	9		320	~30	
319.11 / 2.90	FILL, silty sand, some gravel, trace clay, trace topsoil, with cobbles Loose Brown	4	SS	11		319	~30	26 54 15 5
318.35 / 3.66	FILL, sand and gravel, some silt, with topsoil Compact Brown	5	SS	21	Cuttings	318		
	SAND AND GRAVEL, trace to some silt, with cobbles Compact to very dense Brown	6	SS	25		317		
		7	SS	65				
		8	SS	32	Filter sand	316	~8	70 23 (7)
					Piezometer	315		
313.93		9	SS	72		314		
8.08	END OF BOREHOLE							

Groundwater encountered at about elev. 317.6m during drilling on May 31, 2016.

Water level measured in piezometer at elev. 317.39m on June 1, 2016.

Water level measured in piezometer at elev. 317.59m on July 6, 2016.

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

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

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RECORD OF BOREHOLE No 203

METRIC

PROJECT 12-1132-0163

W.P. 3042-11-00 LOCATION N 4916352.6 , E 195491.0 ORIGINATED BY DH

DIST HWY 6 BOREHOLE TYPE WASH BORE / MUD ROTARY COMPILED BY LMK

DATUM GEODETIC DATE June 1, 2016 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
321.90	PAVEMENT SURFACE								
0.00	ASPHALT								
0.18	CONCRETE (reinforced)								
0.43	FILL, sand and gravel, some silt, with sandy silt pockets and cobbles; Compact to dense; Brown	1	SS	43	○	35	53	(12)	
		2	SS	19					
		3	SS	47					
		4	SS	29	○	52	37	(11)	
319.00 / 2.90	SAND AND GRAVEL, some silt, with cobbles; Compact; Brown	5	SS	13					
		6	SS	18					
317.48 / 4.42	SILTY SAND AND GRAVEL, with topsoil, organics and wood pieces; Compact; Brown	7	SS	13					
316.63 / 5.27	SAND AND GRAVEL, trace to some silt, with sandy silt pockets; Compact to very dense; Brown	8	SS	16					
		9	SS	28					
		10	SS	103	○	45	40	(15)	
313.82 / 8.08	END OF BOREHOLE. The groundwater level cold not be determined in this borehole due to the drilling method used.								

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (321, 320, 319, 318, 317, 316, 315, 314); 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

LDN_MTO_06 1211320163-4000.GPJ LDN_MTO.GDT 07/12/16

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London, Ontario

RECORD OF BOREHOLE No 204B

METRIC

PROJECT 12-1132-0163
W.P. 3042-11-00
LOCATION N 4916374.3 , E 195499.8
ORIGINATED BY DH
DIST
HWY 6
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE July 6, 2016
CHECKED BY

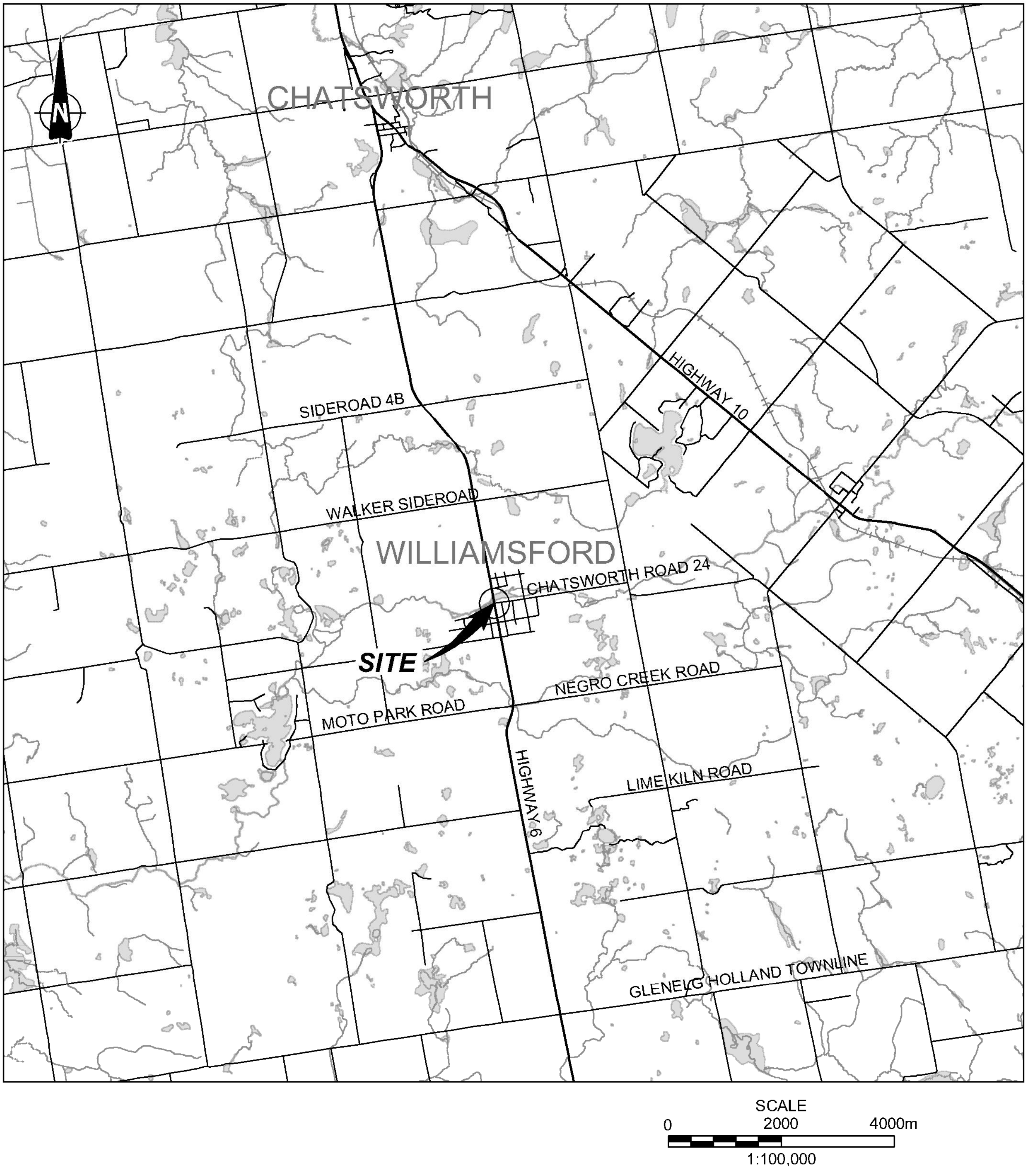
ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.02	GROUND SURFACE						
0.09	TOPSOIL, silty Brown						
321.41 / 0.61	FILL, sand and gravel, some silt with cobbles Brown						
	FILL, sandy silt, trace to some gravel, some topsoil, with cobbles Loose Brown	1	SS	8			
		2	SS	9			
319.89 / 2.13	FILL, silty sand and gravel, with cobbles and topsoil Loose to compact Brown	3	SS	5			
		4	SS	16		o	80 14 (6)
318.36 / 3.66	SAND AND GRAVEL, trace to some silt, with cobbles and boulders Loose to very dense Brown	5	SS	27			
					▽		
		6	SS	85			
		7	SS	8			
		8	SS	36		o	59 29 (12)
		9	SS	14			
313.94 / 8.08	END OF BOREHOLE Groundwater encountered at about elev. 317.6m during drilling on July 6, 2016.						

Elevation scale: 322, 321, 320, 319, 318, 317, 316, 315, 314

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

LDN_MTO_06 1211320163-4000.GPJ LDN_MTO.GDT 07/12/16



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

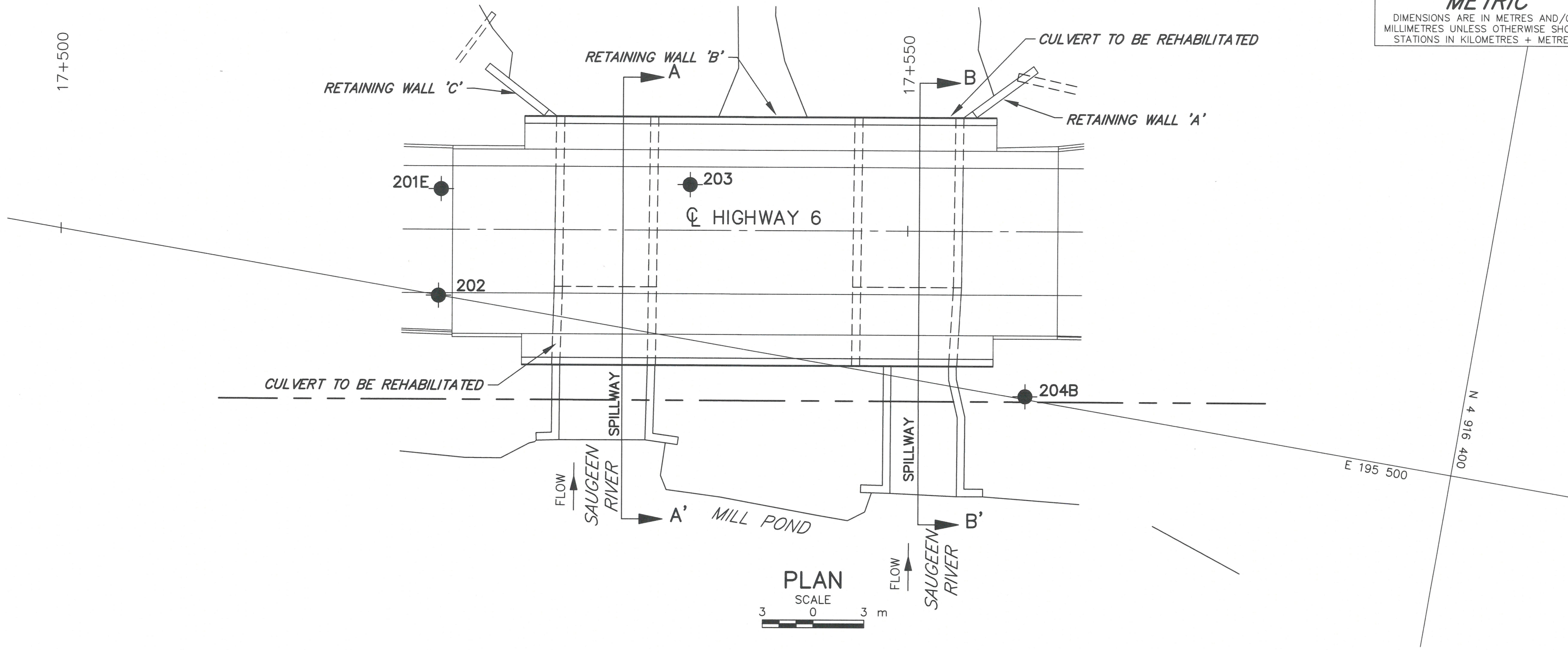
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT	NORTH SAUGEEN RIVER CULVERT (WILLIAMSFORD) SITE NO. 8-342/C HIGHWAY 6 GWP 3042-11-00
TITLE	**KEY PLAN**

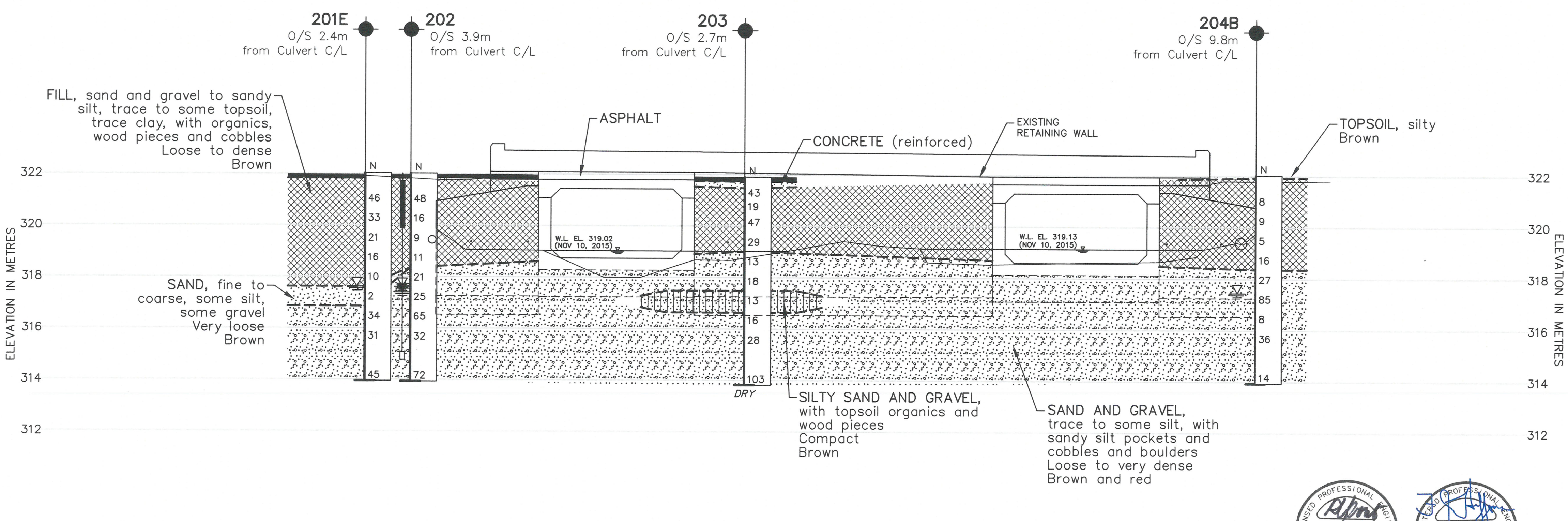
PROJECT No.	12-1132-0163	FILE No.	1211320163-4000-F02001	
		SCALE	AS SHOWN	REV. 0
CADD	LMK	Nov. 1/16	**FIGURE 1**	
CHECK				

Golder Associates

Drawing file: 1211320163-4000-F02001.dwg Dec 07, 2016 - 10:58am



PLAN
SCALE
3 0 3 m



PROFILE ALONG ℄ HIGHWAY 6
HORIZONTAL SCALE
2 0 2 m
VERTICAL SCALE
2 0 2 m

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

CONT No.
WP No. 3042–11–00

SHEET

NORTH SAUGEEN RIVER CULVERT (WILLIAMSFORD)
STRUCTURE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

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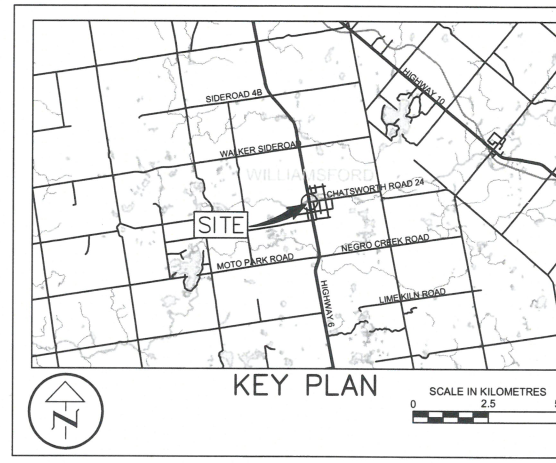
LONDON, ONTARIO, CANADA



KEY PLAN
SCALE IN KILOMETRES
0 2.5 5.0

LEGEND

- Borehole – Current Investigation
- Seal
- Standpipe
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on July 6, 2016
- WL encountered during drilling
- DRY — Water level not established

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 10) NORTHING	EASTING
201E	322.03	4 916 338.2	195 493.8
202	322.01	4 916 339.2	195 500.0
203	321.90	4 916 352.6	195 491.0
204B	322.02	4 916 374.3	195 499.8

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 by Stantec.

LICENSED PROFESSIONAL ENGINEER D. J. PROUT Apr 3/17 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN Apr 3/17 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 41A–240

HWY. 6		PROJECT NO. 12–1132–0163	DIST.
SUBM'D. DH	CHKD. DH	DATE: Mar. 6/17	SITE: 8–342/C
DRAWN: LMK	CHKD. DUP	APPD. FJH	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

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

CONT No.
WP No. 3042-11-00

NORTH SAUGEEN RIVER CULVERT (WILLIAMSFORD)
STRUCTURE REHABILITATION
SOIL STRATA

SHEET

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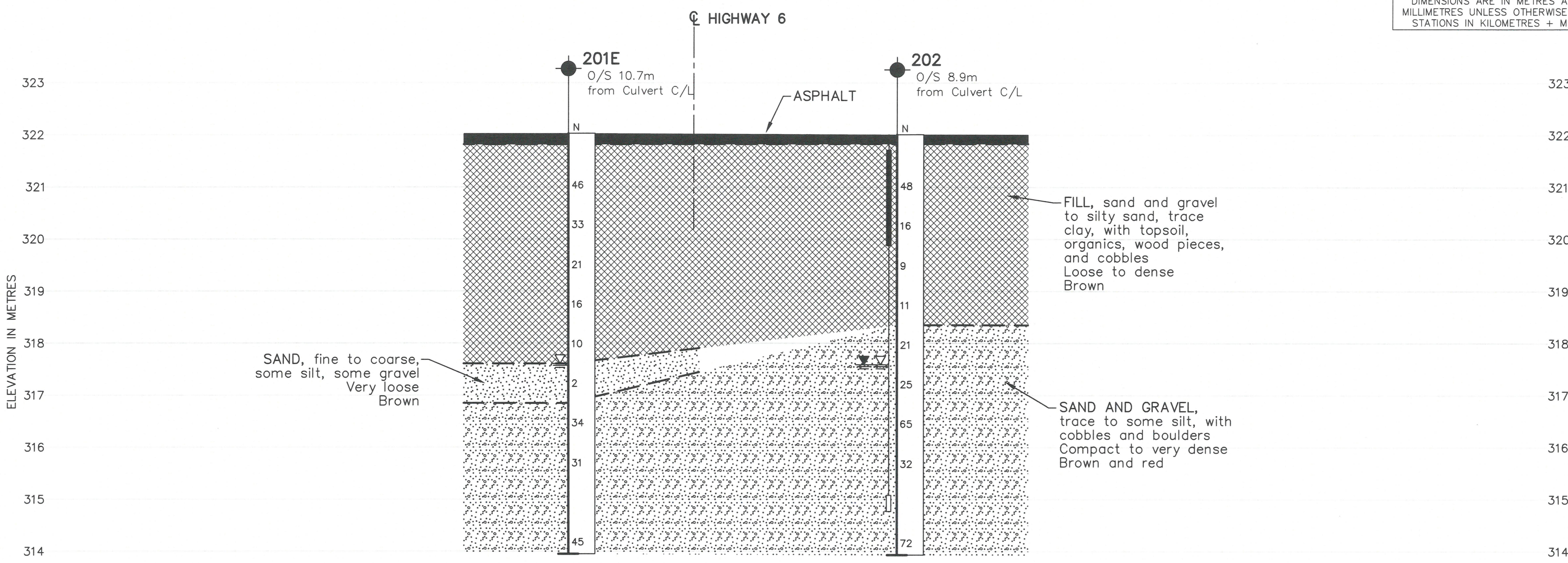
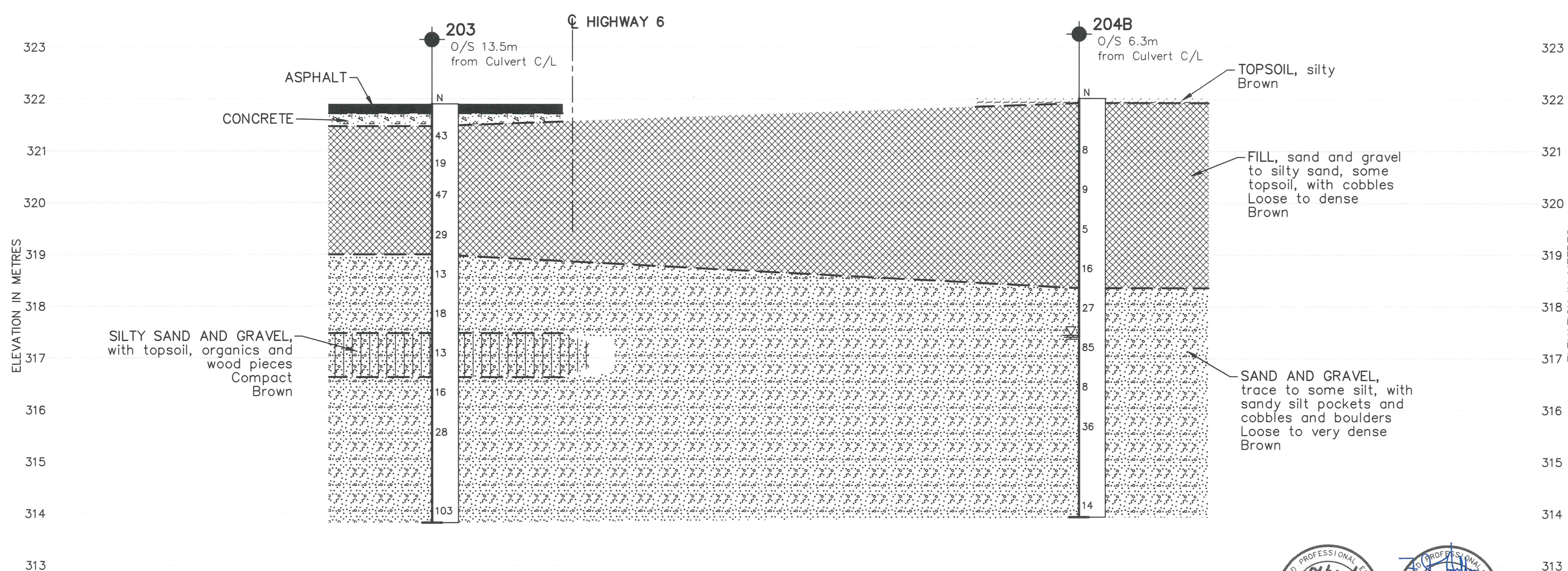


SECTION A

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



SECTION B

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

LEGEND

- Borehole – Current Investigation
- Seal
- Standpipe
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on July 6, 2016
- WL encountered during drilling
- *DRY* — Water level not established

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 10)	
		NORTHING	EASTING
201E	322.03	4 916 338.2	195 493.8
202	322.01	4 916 339.2	195 500.0
203	321.90	4 916 352.6	195 491.0
204B	322.02	4 916 374.3	195 499.8

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REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN Apr 3/17 PROVINCE OF ONTARIO

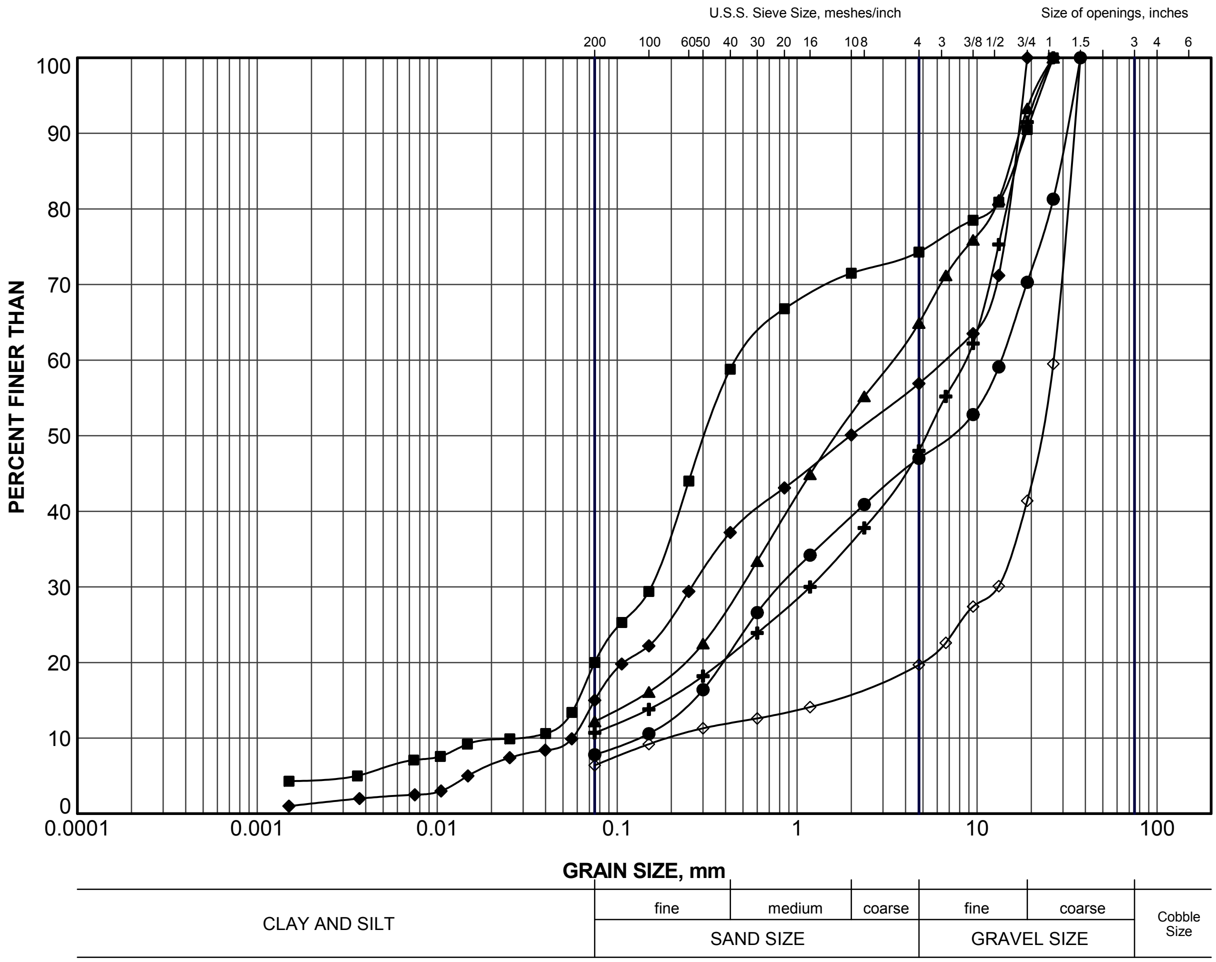
NO.	DATE	BY	REVISION

Geocres No. 41A-240			
HWY. 6		PROJECT NO. 12-1132-0163	DIST.
SUBM'D. DH	CHKD. DH	DATE: Mar. 6/17	SITE: 8-342/C
DRAWN: LMK	CHKD. DUP	APPD. FJH	DWG. 2

PLOT DATE: March 31, 2017
FILENAME: \\golder.gds\gal\London\active\2012\1132 - Geo\1132-0100\12-1132-0163 STANTEC-FDNS-MEGA CULVERTS-3011-E-0041\Drafting\AutoCAD files\1211320163-4000-D02001.dwg

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	2	320.3
■	202	4	318.7
▲	203	1	321.3
✚	203	4	319.4
◆	201E	3	319.5
◇	204B	4	318.7

PROJECT NORTH SAUGEEN RIVER CULVERT (WILLIAMSFORD)
SITE NO. 8-342/C
HIGHWAY 6
GWP 3042-11-00

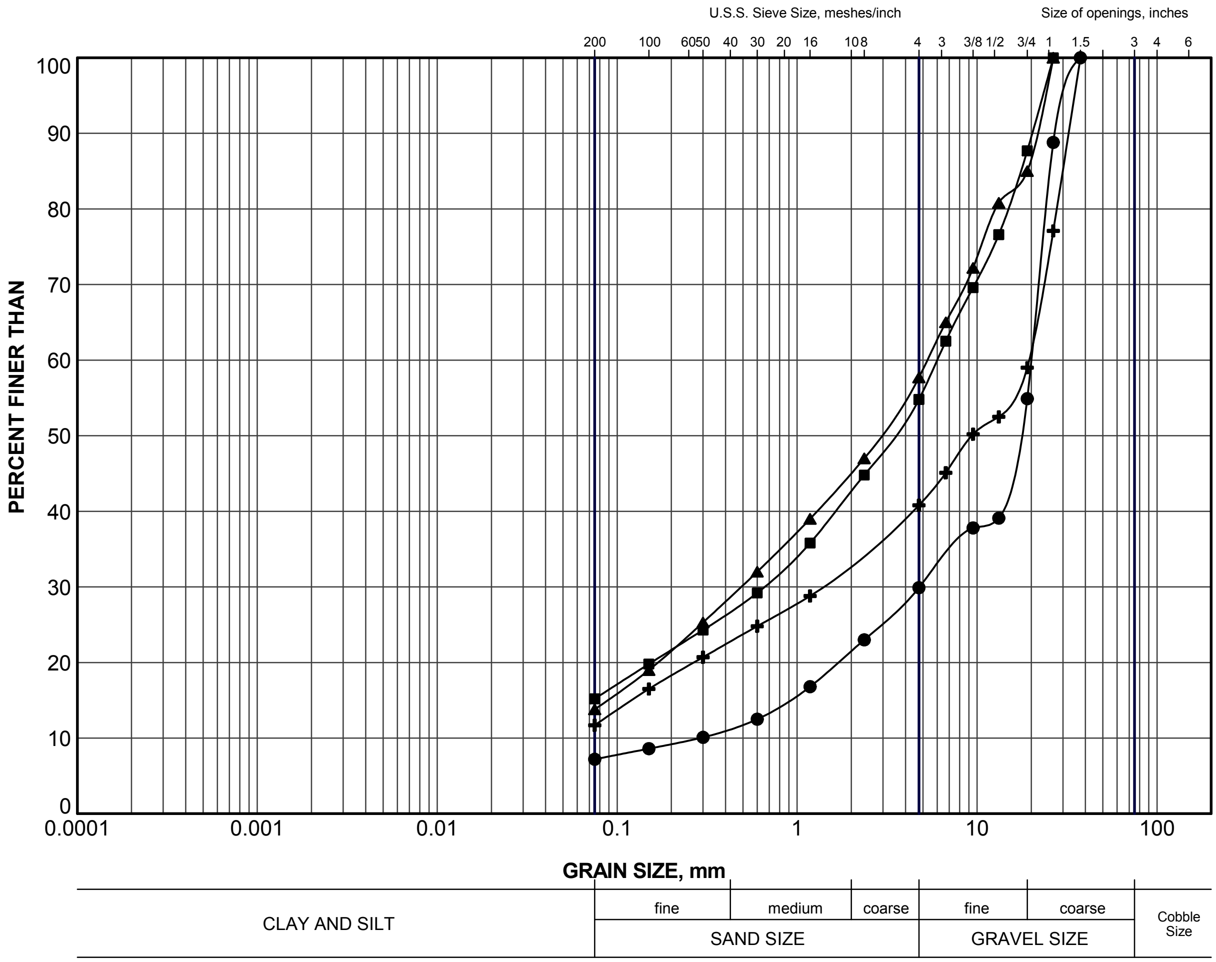
TITLE

GRAIN SIZE DISTRIBUTION
FILL

Golder Associates

PROJECT No.		12-1132-0163	FILE No. 1211320163-4000-F020A1	
			SCALE N/A	REV.
DRAWN	ZJB/LMK	Dec 7/16	FIGURE A-1	
CHECK				

LDN_MTO_GSD GLDR_LDN.GDT 14/11/16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	8	315.7
■	203	10	314.1
▲	201E	7	316.5
✚	204B	8	315.7

PROJECT NORTH SAUGEEN RIVER CULVERT (WILLIAMSFORD)
SITE NO. 8-342/C
HIGHWAY 6
GWP 3042-11-00

TITLE

GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL

Golder Associates

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F020A2	
		SCALE N/A	REV.
DRAWN	ZJB/LMK	Dec 7/16	FIGURE A-2
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT 14/11/16

APPENDIX B

Site Photographs

Photograph 1: West elevation (outlet) of Culvert Site 8-342/C, looking north.

Photograph 2: Looking southwest, at northeast quadrant of Culvert Site 8-342/C.

Photograph 3: Looking south from north side of Culvert Site 8-342/C on Highway 6.

Photograph 4: At southeast quadrant, looking north towards south spillway adjacent to Culvert Site 8-342/C.

Photograph 5: Note significant cracking of north wall of spillway at inlet of north culvert.

Photograph 6: Note retaining wall movement at south elevation (outlet) of Culvert Site 8-342/C.

Photograph 7: Note settlement of west sidewalk at Culvert Site 8-342/C.

APPENDIX C

Information from Geocres No. 41A-157

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1 METRIC

W P 123-83-01 LOCATION Sta. 17 + 553.2 Offset 5.3 m RT of ℄ ORIGINATED BY DLW

DIST 5 HWY 6 BOREHOLE TYPE Washboring and Rock Coring with BXL COMPILED BY DLW

DATUM Geodetic DATE 84 08 13 & 14 CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.7	Pavement Level*							
0.0	Sandy Gravel Traces of Silt and clay		1	SS	21	**		52 32 13 3
	Loose to Very Dense		2	SS	9		320	
	Frequent Boulders		~~3~~	~~SS~~	~~60/~~8 cm			
	(Boulders are over 50% of total overburden)		4	SS	39			74 18 7 1
			~~5~~	~~SS~~	~~60/~~8 cm		318	
			6	SS	42			
			7	SS	20		316	79 14 6 1
			8	WS	-			
			9	SS	44			
			10	SS	52		314	
			11	SS	32		312	63 27 9 1
310.8			~~12~~	~~SS~~	~~60/~~5 cm			
10.9	Dolostone Highly Weathered Unweathered Bedrock		13	BXL RC	95% REC		310	
308.4 / 13.3	End of Borehole						308	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 5 10 15

UNIT WEIGHT γ

* Pavement Thickness: 10 cm

** Water Level not established

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 -◇- 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2 METRIC

W P 123-83-01 LOCATION Sta. 17 + 536.4 Offset 5.4 m RT of ℄ ORIGINATED BY DLW
DIST 5 HWY 6 BOREHOLE TYPE Washboring and Rock Coring with BXL COMPILED BY DLW
DATUM Geodetic DATE 84 08 15 CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
321.7	Pavement Level										
0.0	Concrete										
	Boulders (Dolostone & Granite)										
319.7							320				
2.0	End of Borehole										

$+^3, \times^5$: Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3

METRIC

W P 123-83-01 LOCATION Sta. 17 + 534.2 Offset 5.2 m RT of ℄ ORIGINATED BY DLW

DIST 5 HWY 6 BOREHOLE TYPE Washboring and Rock Coring with BXL COMPILED BY DLW

DATUM Geodetic DATE 84 08 15 & 16 CHECKED BY

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.7	Pavement Level						
0.0	Bridge Deck						
321.1	(Asphalt & Concrete						
0.6							
	5 cm water					320	
318.9	Concrete						
2.8	15 cm Spillway						
	Sandy Gravel					318	
	traces of silt & clay	1	SS	30/	8 cm		89 8 (3)
	Very Dense						
	Frequent Boulders						
	(Boulders are	2	SS	81/	25 cm	316	
	over 50% of	3	SS	51			
	overburden)	4	SS	80/	30 cm		80 14 5 1
		9	SS	53		314	
313.5							
8.2	End of Borehole						
	* Water Level not established						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 5 10 15

UNIT WEIGHT γ

$+^3, \times^5$: Numbers refer to Sensitivity   20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4 METRIC

W P 123-83-01 LOCATION Sta. 17 + 530.4 Offset 3.1 m LT of ℄ ORIGINATED BY DLW

DIST 5 HWY 6 BOREHOLE TYPE Washboring and Rockcoring with BXL COMPILED BY DLW

DATUM Geodetic DATE 84 08 16 & 17 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.8	Pavement Level							
0.0	Bridge Deck							
321.0 / 0.8	(Asphalt & Concrete)							
	10 cm water					320		
319.1 / 2.7	Concrete, 22 cm Spillway	1	SS	65/8 cm				
	Sandy Gravel traces of silt and clay	2	SS	27		318	~10	54 26 18 2
	Compact to Dense	3	SS	36				
	Frequent Boulders (Boulders are over 50% of overburden)	4	SS	43		316	~6	80 14 4 2
		5	SS	39	▽			
		6	SS	108/23 cm		314		
		7	SS	100/28 cm		312	~6	62 28 8 2
310.9 / 10.9	Dolostone Slightly Weathered Bedrock	8	BXL RC	89% REC		310		
309.4 / 12.4	End of Borehole							

Column headings also printed: STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 5 10 15; UNIT WEIGHT γ.

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 123-83-01
LOCATION Sta. 17 + 541.0 Offset 3.1 m LT of ℄
ORIGINATED BY DLW

DIST 5 HWY 6
BOREHOLE TYPE Washboring and Rock Coring with BXL
COMPILED BY DLW

DATUM Geodetic
DATE 84 08 20 & 21
CHECKED BY

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
321.8	Pavement Level
0.0	Asphalt & Concrete
	Sandy Gravel traces of silt and clay Loose to Very Dense Frequent Boulders (The Boulders are over 50% of overburden)
311.1 / 10.7	Dolostone Bedrock: Highly Weathered / Unweathered
309.4 / 12.4	End of Borehole

* Water Level not established.

SAMPLES

NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8	*	
2	SS	72/18 cm		
3	SS	54		53 33 13 1
4	SS	26		
5	SS	35		
6	SS	92		
7	SS	22		59 30 9 2
8	SS	60		
9	SS	71/28 cm		
10	SS	85/25 cm		
11	SS	74/18 cm		80 12 (8)
12	BXL RC	84% REC		

ELEVATION SCALE: 320, 318, 316, 314, 312, 310

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

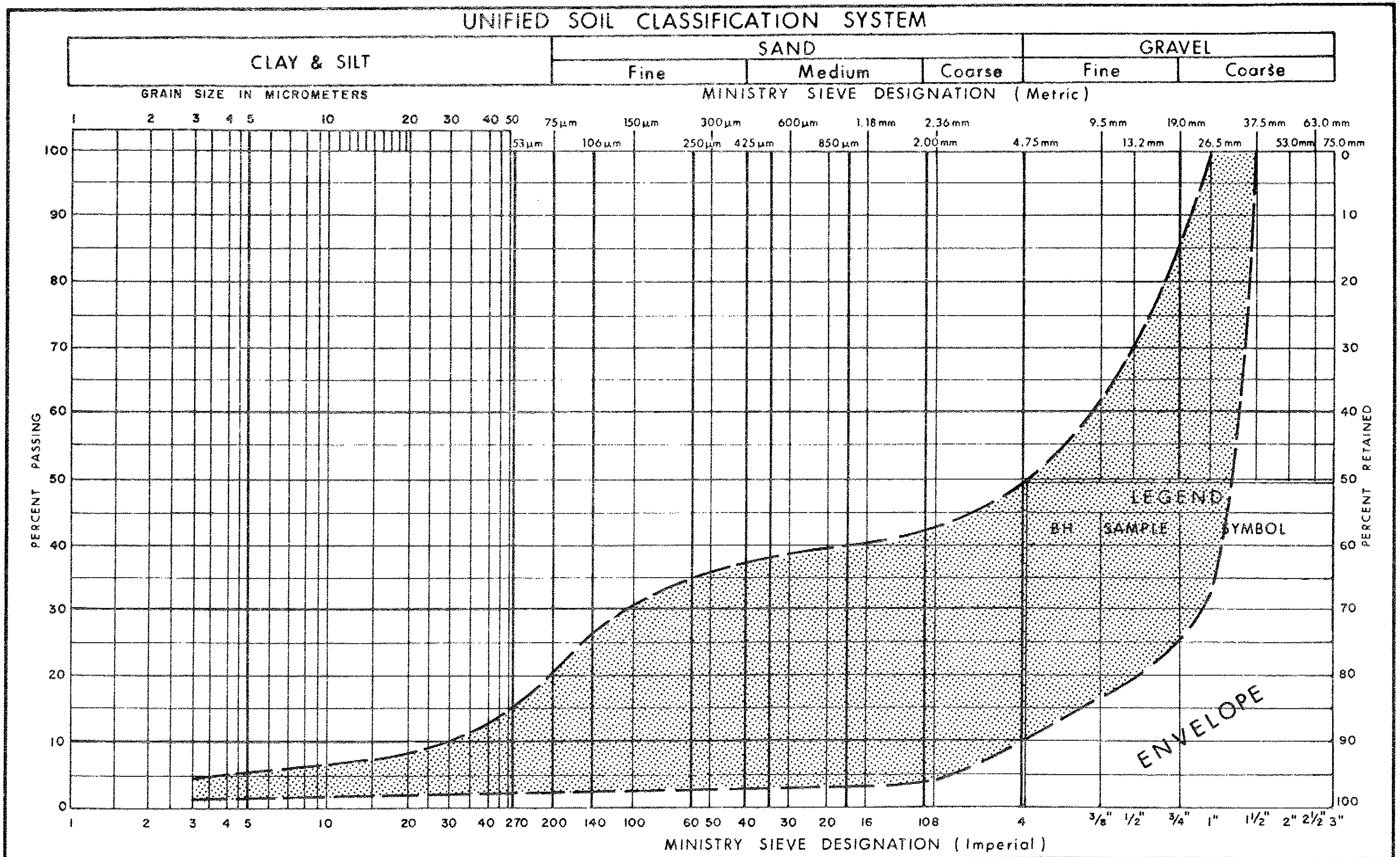
WATER CONTENT (%): 5 10 15

UNIT WEIGHT γ

+3, ×5 : Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

78 12 M



Ministry of Transportation and Communications

Ontario

GRAIN SIZE DISTRIBUTION

SANDY GRAVEL

TRACES OF SILT AND CLAY

FIG No	1
W P	123-83-01

BOREHOLE NUMBER	DEPTH (M)	% CR*	% RQD*	CORE DESCRIPTION DEPTH (M)	DESCRIPTION
1	10.79 - 11.64	96.4	85.7	10.79 - 13.32	Dolostone (Guelph Formation), cream, porous with occasional vugs* 1.0 to 10.0 mm, unweathered with moderately spaced joints. Void or highly weathered zone at 11.64 m to 11.89 m (loose gravel at top of core is not bedrock).
	- 11.89	25.0	0		
	- 13.32	100.0	82.9		
2	0.09 - 0.39	75.0	-	0.09 - 0.39	Concrete
	- 1.52	51.1	-	0.39 - 2.13	Boulders (dolostone and granite)
	- 2.13	16.6	-		
4	9.45 - 9.66	100.0	-	9.45 - 10.49	Boulders and sand
	- 9.88	0	-	10.49 - 12.37	Dolostone (Guelph Formation) cream to buff, porous containing occasional vugs* 1.0 mm to 10.0 mm, slightly weathered, with close to moderately spaced joints.
	- 10.49	45.0	-		
	- 10.67	100.0	58.3		
	- 12:37	92.9	73.2		
5	10.69 - 10.88	66.6	66.6	10.69 - 11.40	Dolostone (Guelph Formation), brown to buff, porous and friable (assumed core loss), moderately to highly weathered, with closely spaced joints.
	- 12.41	84.0	72.0		
				11.40 - 12.41	Dolostone (Guelph Formation), buff to cream, porous with occasional vugs* 1.0 mm to 10.0 mm, slightly weathered becoming unweathered with moderately spaced joints.

* CR = CORE RECOVERY; RQD = ROCK QUALITY DESIGNATION; Vug = a tiny cavity typical of carbonate rocks

METRIC

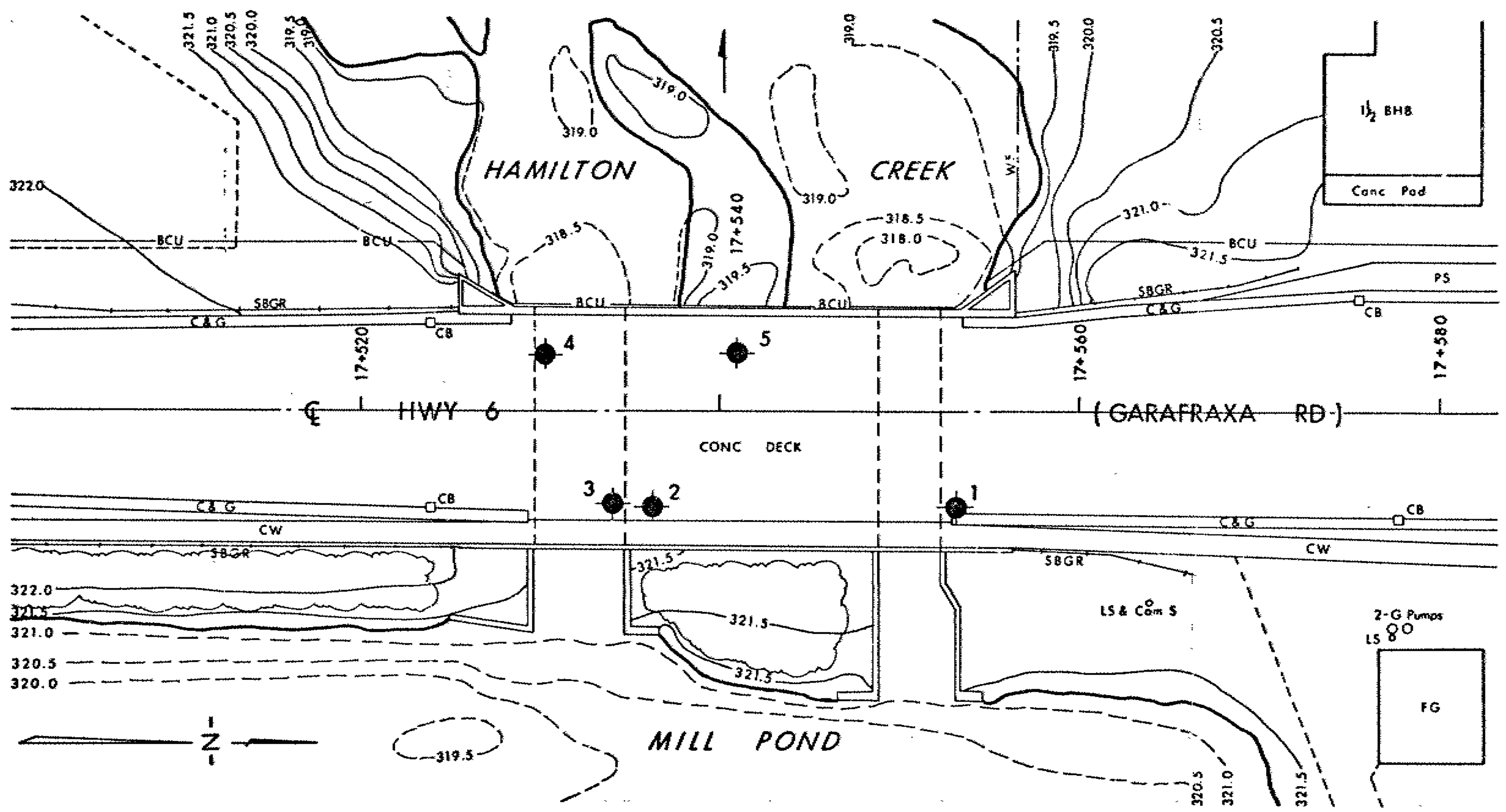
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 123-83-01

HAMILTON CREEK

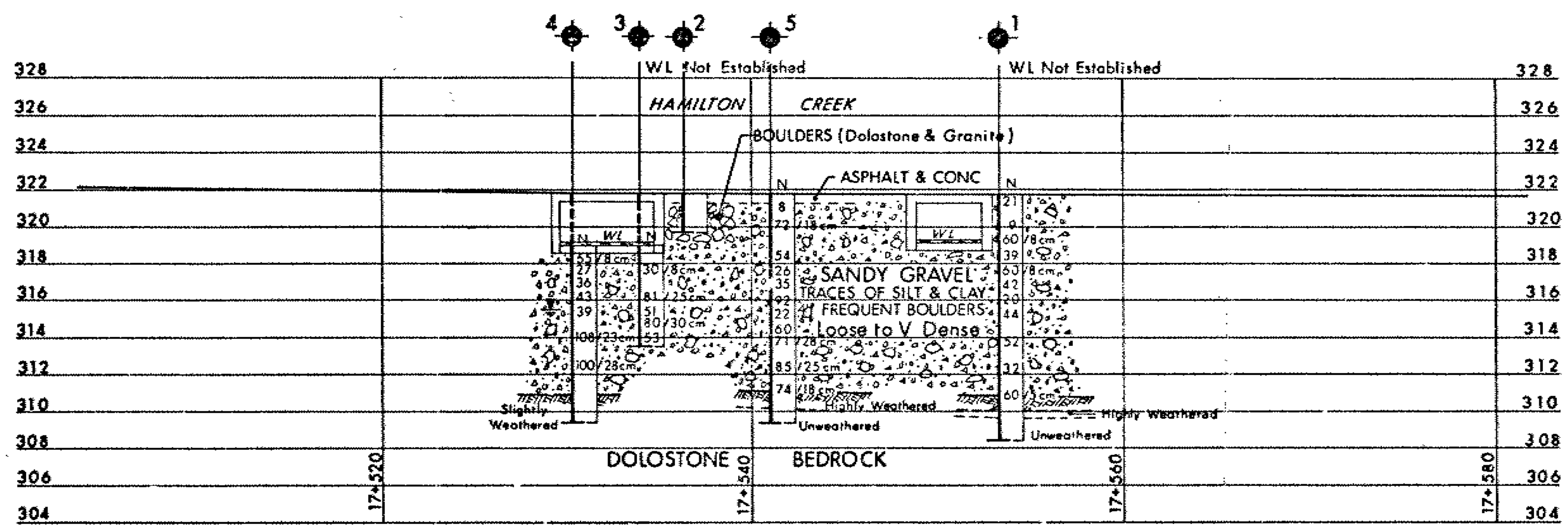
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



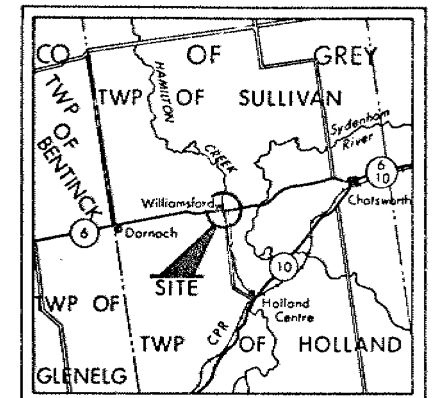
PLAN

SCALE 4m 2 0 4m



℄ PROFILE HWY 6

SCALE 4m 2 0 4m



KEY PLAN

SCALE 5km 0 5km

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N — Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE — Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 84 08

No	ELEVATION	STATION	OFFSET
1	321.7	17+553.2	5.3 m RT
2	321.7	17+536.4	5.4 m RT
3	321.7	17+534.2	5.2 m RT
4	321.8	17+530.4	3.1 m LT
5	321.8	17+541.0	3.1 m LT

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV.	DATE	BY	DESCRIPTION

Geocres No 41A-157

HWY No 6			DIST 5
SUBM'D PP	CHECKED	DATE 84 11 13	SITE 8-159-342
DRAWN SO	CHECKED	APPROVED	DWG 1238301-A

REF NO E-10017-1 84 07

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