PRELIMINARY FOUNDATION INVESTIGATION
AND DESIGN REPORT
WHITEMANS CREEK BRIDGE REPLACEMENT
SITE 1-85
HIGHWAY 24 REHABILITATION
GWP 336-97-00, AGREEMENT NUMBER 3006-E-0012
MINISTRY OF TRANSPORTATION - WEST REGION

Submitted to:

Delcan Corporation
1069 Wellington Road South, Suite 214
London, Ontario
N6E 2H6

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TABLE OF CONTENTS

SECTION PAGE

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

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SITE 1-85
HIGHWAY 24 REHABILITATION
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Delcan Corporation (Delcan) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a preliminary foundation investigation as part of the preliminary design work for GWP 336-97-00. The project involves the preliminary design for the rehabilitation of Highway 24 from Brant County Road 53 to just north of Highway 403 and includes:

- Improvements of the intersection of Highway 24 and Brant County Road 53 including upgrading traffic signals and illumination and extension of left turn lanes;
- Intersection improvements at Highway 24 and Bethel Road including signalization and left turn lanes;
- Minor intersection improvements at the remaining intersections;
- Lane and shoulder widening;
- Minor vertical curve corrections; and
- Replacement of the Whitemans Creek bridge, Site 1-85.

Improvements in the horizontal and vertical alignments are required for future widening to four lanes.

This report addresses the replacement of the existing bridge over Whitemans Creek. Replacement of the existing structure is currently under consideration either at the present location or at an alternate location either immediately east or west of the existing structure if the horizontal alignment is altered. A temporary detour bridge with approach embankments may be required if the replacement bridge is to be built at the existing location. Current design options which are given priority consideration are Option 1 (a structure with steel girders on the Alternate 7 alignment) and Option 2 (a structure with concrete girders on the Alternate 7 alignment). The Alternate 7 alignment is preferred and maintains the location of the proposed structure at the current crossing location.

The purpose of the preliminary 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 P61-3103 dated August 11, 2006, revised Proposal P61-3103-1 dated February 7, 2007 and our letters dated March 22, 2007 and April 16, 2007. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated November 7, 2006.

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

2.0 SITE DESCRIPTION

The existing bridge over Whitemans Creek is situated on Highway 24 (Rest Acres Road), approximately 2.8 kilometres south of Highway 403, between Bethel Road and Robinson Road. The site location is shown on the Key Plan on Figure 1 and Drawing 1. Site photographs are presented in Appendix B.

The existing structure was constructed in 1958. According to the County of Brant drawing entitled "Gurney Bridge General, Twp. Brantford, Co. Rd. No. 33, Lots 10 & 11, Concession 3" dated March 21, 1958, the bridge is 39.2 metres long with three spans. The bridge deck has a curb to curb width of 9.1 metres and is a continuous concrete "Tee" beam deck supported on standard abutments. The bridge deck is at approximately elevation 224.0 metres. The design drawings indicated that the abutments are supported by vertical and battered HP 8 x 36 (HP200 x 54) piles which extend to 7.6 metres below the cut off elevation at 222.3 metres to about elevation 214.7 metres. The bridge piers are supported on vertical HP 14 x 73 (HP360 x 109) piles which extend to 10.7 metres below the cut off elevation of 223.6 metres to approximately elevation 212.9 metres. No as-built information was available.

Bridge rehabilitation was conducted on three occasions. In 1963, the sidewalks were repaired. Parapet walls were added in 1982. In 2002, the steel pier columns were repaired with splice plates. Temporary shoring was installed in 2006 after wide shear cracks in the concrete tee beams were noted.

The Whitemans Creek bridge is located in a rural residential and agricultural area at the eastern boundary of Apps Mills Conservation Area. The banks of Whitemans Creek are well vegetated. Whitemans Creek flows from west to east under Highway 24. The topography features moderate to steep slopes towards Whitemans Creek with a terraced escarpment south of the creek. Elevations in the immediately vicinity of the crossing range between 220 metres beside Whitemans Creek to 230 metres in the cut area north of the bridge. The descent from Bethel Road to the north and from the plateau just north of Robinson Road involves changes in elevation of approximately 9 and 32 metres, respectively. The elevation of Whitemans Creek is about 218.4 metres. Much of the north approach to the bridge is in cut while the southern approach up to about 200 metres south of the bridge has fill embankments up to 10 metres high. Springs and continuous water flow in the ditches were noted adjacent to the existing bridge.

2.1 Site Geology

The area of the proposed Highway 24 rehabilitation lies in the physiographic region of southern Ontario known as the Norfolk Sand Plain[1]. The Norfolk Sand Plan is wedge shaped and extends from the Lake Erie shore to Brantford. The sands and silts were deposited from a significant meltwater discharge from the Grand River as it entered former glacial Lakes Whittlesey and Warren and formed a delta.

Based on the Ontario Department of Mines and Northern Affairs Map 2240 entitled “Pleistocene Geology of the Brantford Area”, the creek bed and floodplain of Whitemans Creek is composed of modern alluvium consisting of silty sand, gravel, clay and muck. A narrow band of till and other stratified sediments are present north of the creek. Older gravel and sand alluvium exist in remnants of terraces on the south side of the creek. Glaciofluvial outwash and deltaic deposits of gravel and gravely sand are present in the plateaus beside the Whitemans Creek valley. In many places, the coarse granular materials are overlain by several metres of sand.

The bedrock is reported to be shale with lenses of anhydrite and gypsum belonging to the Salina formation of Upper Silurian Age (Geological Survey of Canada, Map 1263A entitled “Geology, Toronto-Windsor Area”, dated 1969). The bedrock surface is near elevation 213 metres according to Ontario Department of Mines Map 2035.

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

3.0 INVESTIGATION PROCEDURES

The field work for this portion of the investigation was carried out between June 14 and 20, 2007, during which time two boreholes, numbered 1 and 2, were drilled adjacent to the ends of the existing structure to depths of 17.6 and 16.4 metres, respectively.

The investigation was carried out using an all-terrain vehicle mounted CME 85 power auger supplied and operated by a specialist drilling contractor. Samples of the overburden were obtained at 0.75 metre intervals of depth to about 9.0 metres depth and then at 1.5 metre intervals using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures. In addition, a dynamic cone penetration test was carried out adjacent to borehole 2 to confirm the relatively low N values measured in the fill. Wash samples of the underlying bedrock were obtained by triconing in borehole 1 and, in borehole 2, the rock was cored in N size to a depth of 3.5 metres below the bedrock surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. A deep piezometer and standpipe were installed in borehole 1 and a piezometer was installed in borehole 2. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 128/03.

The field work was supervised on a full-time basis by an experienced member of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in situ testing operations, logged and surveyed the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and routine classification testing. Index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A. The samples of rock core were forwarded to our Mississauga laboratory for examination by an experienced geologist.

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

BOREHOLE	LOCATION (m)		GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
	Northing	Easting		
1	4 777 852.6	234 264.4	223.97	17.61
2	4 777 900.3	234 232.5	223.88	16.40

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 surficial topsoil underlain by thick layers of granular fill underlain by deposits of sand and gravel, followed by sandy silt and sandy silt till overlying dolostone bedrock. Layers of silt, sand and clayey silt were encountered within the sandy silt.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on the attached 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.

4.1.1 Topsoil and Fill

Surficial layers of topsoil between 90 and 100 millimetres thick were found at the ground surface in both boreholes.

The topsoil was underlain by 3.4 to 4.9 metres of granular fill from elevation 223.9 metres. The fill was comprised of very loose to dense but generally loose layers of sand and gravel, sand and silty sand. The sand and gravel below elevation 221.9 metres at borehole 2 contained concrete rubble and trace amounts of topsoil were present in the fill immediately underlying the topsoil layers.

N values, as determined in the standard penetration testing, in the granular fill ranged from 3 to 33 blows per 0.3 metres. A sample of sand and gravel fill in borehole 1 was found to have a water content of 11 per cent. The results of a grain size analysis conducted on a sample of sand and gravel fill is shown on Figure A-1 in Appendix A.

4.1.2 Sand and Gravel

The fill was underlain by compact to dense sand and gravel from elevations 218.9 and 220.4 metres in boreholes 1 and 2, respectively. Cobbles were encountered in this deposit. The sand and gravel had measured N values of 17 to 32 blows per 0.3 metres. The natural water contents of two samples of the sand and gravel were 9 and 12 per cent.

The results of grain size testing on two samples of sand and gravel recovered from the standard penetration testing are presented on Figure A-2.

4.1.3 Sandy Silt

Compact to very dense sandy silt was encountered beneath the sand and gravel from elevations 217.0 to 218.4 metres. The sandy silt was interlayered with silt and fine sand from elevations 216.7 and 215.4 metres, respectively, in borehole 1 and clayey silt from elevation 217.2 metres in borehole 2. The sandy silt had measured N values of 28 to 67 blows per 0.3 metres of penetration. Water contents of 12 to 13 per cent were measured in the sandy silt. The sandy silt is a silt of low plasticity with a liquid limit of 14 per cent, a plastic limit of 11 per cent and a plasticity index of 3 per cent. The results of grain size analyses conducted on three samples of sandy silt are presented on Figure A-3. The results of the Atterberg limits testing are shown on Figure A-6.

4.1.4 Silt

A layer of dense silt containing some sand was encountered within the sandy silt layer in borehole 1 from elevation 216.7 metres. The results of grain size testing on a sample of silt recovered from the standard penetration testing are presented on Figure A-4.

The silt had a measured N value of 32 blows per 0.3 metres of penetration. The natural water content of a sample of silt was about 10 per cent.

4.1.5 Clayey Silt

Layers of clayey silt were encountered in borehole 2 within the sandy silt layer from elevation 217.2 metres and below the sandy silt from elevation 215.0 metres. The clayey silt had measured N values of 16 to 22 blows per 0.3 metres of penetration.

The natural water content of a sample of clayey silt was about 17 per cent. The plastic and liquid limits of one sample were 15 per cent and 28 per cent, respectively, with a plasticity index of 13 per cent. The results of the Atterberg limits testing on a single clayey silt sample are presented on the Plasticity Chart, Figure A-6.

The results of grain size testing on a sample of clayey silt recovered from the standard penetration testing are presented on Figure A-5.

4.1.6 Sand

Sand layers were encountered in borehole 1 within the sandy silt from approximately elevation 215.4 metres and below the sandy silt from elevation 210.9 metres. The sand is very dense with an N value greater than 100 blows per 0.3 metres.

4.1.7 Sandy Silt Till

A layer of dense sandy silt till was encountered beneath the clayey silt in borehole 2 from elevation 212.3 metres. The sandy silt till had an N value of 32 blows per 0.3 metres penetration.

4.1.8 Bedrock

Shaley dolostone bedrock of the Salina formation was found beneath approximately 13.9 and 13.0 metres of overburden at elevations 210.1 metres and 210.9 metres in boreholes 1 and 2, respectively.

In borehole 2, the bedrock was cored in N size for 3.0 metres below the depth of split spoon refusal. Examination of the rock cores indicated that the dolostone is slightly weathered and is gypsiferous below elevation 210.1 metres.

The rock quality designation (RQD) varied between 0 and 79 per cent with an average of 50 per cent indicating very poor to good quality rock. The quality of the rock core was noted to improve with depth. The rock core above elevation 210.1 metres in borehole 2 was found to be of very poor quality with an RQD of 0. The following table summarizes the RQD, total core recovery (TCR) and solid core recovery (SCR) for the rock cores.

	ELEVATION (m)				
BOREHOLE	From	To	RQD (%)	TCR (%)	SCR (%)
2	210.5	210.1	0	72	44
2	210.1	208.6	70	100	87
2	208.6	207.5	79	98	79
		AVERAGE	50	90	70

4.2 Groundwater Conditions

Groundwater conditions were observed during and upon completion of drilling. Groundwater was encountered in the sand and gravel at elevations 218.8 metres and 219.9 metres in boreholes 1 and 2, respectively. The most recent groundwater measurements were made on July 6, 2007. On this date, the groundwater levels in the deep piezometer and standpipe in borehole 1 were measured at elevation 220.0 metres and 218.9 metres, respectively. The groundwater level in the piezometer in borehole 2 was measured at elevation 220.6 metres.

The water level in Whitemans Creek was measured at elevation 218.8 metres in January 2007 and at elevation 218.39 metres on June 21, 2007.

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

BOREHOLE	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL		INSTALLATION	MEASURED GROUNDWATER LEVEL			
					June 19, 2007		July 6, 2007	
		Depth (m)	Elevation (m)		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
1	223.97	5.1	218.8	Standpipe	5.11	218.86	5.08	218.89
				Piezometer	4.09	219.88	3.96	220.01
2	223.88	4.0	219.9	Piezometer	3.40	220.48	3.25	220.63

The groundwater level in the sand and gravel is at elevation 219 metres. The groundwater level in the bedrock and the sandy silt till overlying bedrock is at elevation 220.5 metres. Both of these groundwater levels represent artesian conditions relative to the creek water level.

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

5.0 MISCELLANEOUS

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

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

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.
Geotechnical Engineer

Philip R. Bedell, P. Eng.
Principal

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

TP/DUP/PRB/FJH/cr
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PART B – PRELIMINARY FOUNDATION DESIGN REPORT

WHITEMANS CREEK BRIDGE REPLACEMENT
SITE 1-85
HIGHWAY 24 REHABILITATION
GWP 336-97-00, AGREEMENT NO. 3006-E-0012
MINISTRY OF TRANSPORTATION – SOUTHWESTERN REGION

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the preliminary design of the replacement of the existing bridge over Whitemans Creek on Highway 24 based on our interpretation of the factual information obtained during the preliminary foundation investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing three span bridge has a continuous concrete ‘Tee’ beam deck supported on standard abutments. The original 1958 bridge design drawings indicate that the piers are supported on HP 14 x 73 (HP360 x 109) piles that extend to approximately elevation 212.9 metres. The original design load for piles at the piers was 70 tons (about 625 kilonewtons) per pile. The abutments were to be supported on a pile group consisting of vertical and battered HP 8 x 36 (HP200 x 54) piles driven to approximately elevation 214.7 metres. The abutment piles had a design load of 20 tons (approximately 180 kilonewtons) per pile. No as-built information was available.

The alignment profile for the proposed rehabilitation of Highway 24 currently under consideration is Alternate 7. Two options, Option 1 and Option 2, are being contemplated for the replacement structure. In Option 1, the top of deck elevation will be 224.85 metres at the north abutment and 223.66 metres at the south abutment. The 14.5 metre wide concrete deck will be supported by steel girders. The bridge will feature pile supported integral abutments and will have a single 40 metre span and 6.0 metre long approach slabs. Option 1 allows more rapid construction. For Option 2, the top of deck elevation will be 224.83 and 224.66 at the north and south abutments, respectively. The deck width and materials will be identical to Option 1; however, the deck will be supported by concrete girders. The abutments will also be pile supported integral abutments. There will be a single, 35 metre long span with 6 metre long approach slabs. A minimum 3 metre navigational clearance and wildlife corridors 4 metres wide will be provided along the front slopes at each bank for both options. Rapid construction cannot be implemented with Option 2; however, the designers anticipate significant cost savings over Option 1 with this alternative. In Option 1, the grade raise is 0.4 to 0.8 metres and in Option 2 the grade raise is about 0.8 metres.

6.2 Foundations

The subsoils encountered in the boreholes put down during the investigation typically consist of surficial layers of topsoil over as much as 5 metres of very loose to dense granular fill to approximately elevation 220 metres. The fill is underlain by a compact to dense sand and gravel layer to about elevation 217 metres. Beneath the sand and gravel layer is compact to very dense sandy silt interlayered with silt, clayey silt and sand. Beneath the sandy silt, layers of very stiff clayey silt, very dense sand and sandy silt till were found overlying bedrock. Grey shaley and gypsiferous dolostone of the Salina formation was encountered from elevation 210.9 metres at the north abutment and 210.1 metres at the south abutment. The groundwater elevation in the sand and gravel deposit is 219 metres and the groundwater level is at elevation 220.5 metres in the deeper bedrock. The approximate water level in Whitemans Creek is at elevation 218 metres.

Spread footings are not considered practical for this site due to the presence of fills of 3.5 metres to 5.0 metres in depth necessitating founding in the underlying compact to dense native sand and gravel. Also, the groundwater level in the summer period was at or slightly above the upper surface of the pervious sand and gravel layer. For these reasons, it is recommended that the structure be supported on steel piles driven to bedrock.

The various foundation options considered for this site are compared in Table I. This table includes estimated foundation costs and summaries of the feasibility of each option. The costs given are rough estimates presented to give an order of magnitude cost comparison between alternatives rather than absolute figures.

6.2.1 Deep Foundations

Driven steel piles advanced to practical refusal on the bedrock are considered suitable to support the abutments and piers, if any, for the proposed replacement structure. The use of steel H-piles is the preferred founding option because they have the flexibility required for use with integral abutments. Deep foundations, such as driven steel piles, are the preferred technical alternative.

Geotechnical Axial Resistance – Driven Steel H-Piles

HP 310 x 110 piles driven to refusal in the dolostone bedrock at about elevation 210 metres may be designed using a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kilonewtons (kN). This value takes into account the structural capacity of the pile. A Serviceability Limit States (SLS) value is not provided because the bedrock is considered to be an unyielding material. Under such conditions, SLS values (for 25 millimetres of settlement) do not govern design because the SLS value is much higher than the ULS value. The surface elevation(s) and quality of the bedrock should be confirmed during the foundation investigation for detail design.

Vertically driven pile tips should be equipped with Type I driving shoes in accordance with current MTO practice (Ontario Provincial Standard Drawing (OPSD) 3000.100 and SP903S01). Battered piles should be equipped with Type II driving shoes to ensure adequate seating of the piles on the bedrock. The steel H-piles should be installed and monitored in accordance with SP903S01. The pile driving note to be added to the contract drawings is: "Piles to be driven to bedrock".

Cobbles were encountered in the sand and gravel during the field investigation. In addition to the sand and gravel, the presence of cobbles and boulders should be anticipated in the sandy silt till and these may impact pile driving operations.

Downdrag Load (Negative Skin Friction)

The existing overburden material is primarily cohesionless but does contain apparently discontinuous seams and layers of very stiff to hard clayey silt within the compact to very dense sandy silt strata. The magnitude of negative skin friction that will be developed for piles installed on this site will be minimal, especially if cohesionless fills are used for embankment construction. Settlement of the very stiff to hard cohesive deposits, and consequently the magnitude of negative skin friction, is expected to be low given the minor grade raise proposed.

Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of inclined piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. If integral abutments are proposed, there may also be a requirement for the piles to move sufficiently to accommodate deflections of the bridge deck. The horizontal reaction for an HP 310 x 110 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

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

d = pile width or diameter (m)

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

S_u = undrained shear strength of the soil (MPa)

z = depth below ground surface grade (m)

SOIL TYPE	ELEVATION (m)		n_h	S_u
	From	To	MPa/m	(kPa)
Embankment fill (granular)	Surface	220	1 – 3	-
Compact to dense sand and gravel	220	217	3 – 10	-
Dense to very dense sandy silt	217	215	10 – 12	-
Very stiff clayey silt	215	212	-	95 - 125
Compact to very dense sandy silt, sandy silt till and sand	215	210	7 - 12	-

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***[2]
8d	1.00
6d	0.70
4d	0.40
3d	0.25

A maximum lateral resistance of 160 kN at ULS and 60 kN at SLS is recommended for HP310 x 110 piles.

Frost Protection

The pile caps should be provided with a minimum of 1.2 metres of soil cover or thermal equivalent for frost protection.

Driving Pad

Artesian conditions exist relative to the creek water level for the groundwater level in the underlying sand and gravel, the sandy silt till overlying the bedrock and the bedrock itself. As a result, the possibility of groundwater migration around the pile annulus exists. To minimize the creation of a potential void around the pile as it is driven, driving shoes, reinforcement to flanges, splice plates and the like should add as little as possible to the pile cross-sectional dimensions.

[2] Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

Further, it is recommended that the piles be driven from a 0.5 metre thick pad of Granular A which would act as a filter should sustained groundwater flow occur. The blanket should be constructed in accordance with the Drainage Blanket Detail sheet provided in Appendix C.

6.3 Lateral Earth Pressures

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

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawings (OPSD) 3101.150, 3190.100 and 3121.150.

- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Clause 6.9.3. Compaction equipment should be used in accordance with SP105S10.

- In accordance with CHBDC Clause C6.9.1, the granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).

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

	GRANULAR A	GRANULAR B (Type III)
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

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

6.4 Excavations and Temporary Cut Slopes

Excavations for pile cap construction will extend through surficial topsoil and fill materials and may encounter the sand and gravel deposits. Based on the subsurface conditions encountered in the boreholes and the recent measurements of groundwater levels, the groundwater levels are expected to be near elevation 219 metres in the sand and gravel and 220.5 metres in the bedrock. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical.

Pumping from well filtered sumps located at the base of the excavations may be required to provide groundwater control during excavation for the pile caps. Sumps should be maintained outside of the actual footing limits. However, more aggressive dewatering will be required for the installation of the CSP surrounds required for the integral abutment H-piles and if excavations extend below the groundwater level. Preliminary design information provided by Delcan indicates that the CSP surrounds will be installed to a depth of 3 metres and will terminate in the sandy silt at elevation 216 to 217 metres. It is considered possible to install the CSP surrounds without dewatering by preaugering within a temporary casing to the desired depth. Particularly for piles installed at the south abutment, the casing should extend below elevation 217 metres to ensure the base of the casing is located in the less permeable sandy silt or silt. Alternatively, the CSP's could be replaced with a 6 millimetre thick steel casing with excavation and sand infill being done in the wet. It may also be necessary to maintain stability in the granular fills and native soils by flattening excavation slopes or blanketing the slopes with coarse free draining materials. The appropriate Non Standard Special Provision (NSSP) should be included in the contract documents. Surface water runoff should be directed away from the excavations at all times. Surficial water seepage into the excavations and saturation of portions the surficial granular fill should be expected. Seepage will be heavier during periods of sustained precipitation.

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, native sand and gravel and all other native granular soils below the groundwater level are considered to be Type 3 soils. Properly dewatered sand, sandy silt and sandy silt till would be classified as Type 2 soils.

Temporary Roadway Protection

Temporary roadway protection (shoring) will have to be provided to support the existing embankment fills and other excavations with space restrictions during construction. The roadway protection should conform to Performance Level 2 in accordance with SP 105S19. 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 raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

The support systems may be designed using the following parameters:

SOIL TYPE	COEFFICIENT OF EARTH PRESSURE			INTERNAL ANGLE OF FRICTION (degrees)	UNIT WEIGHT (kN/m³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Granular Fill	0.33	0.50	3.0	30	20
Sand and Gravel	0.27	0.43	3.7	35	21

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

6.5 Removal of Existing Pier Walls

Consideration is being given to removing the existing pier walls to some nominal depth below the creek bed and cutting off the H piles. The base of the pier walls was to be about 0.3 metres below the creek bottom or near elevation 217.4 metres. The pier walls are supported by the pier piles. The bottom of Whitemans Creek is at approximately elevation 218.3 metres at the north bank and 218.5 metres at the south bank.

Removal of the existing pier walls together with the required excavation and cutting of piles could be accomplished during a period of low flow by constructing a groundwater cutoff enclosure consisting of a cofferdam with interlocking steel sheet piling. Groundwater seepage from the sand and gravel within the enclosure can then be controlled using conventional sumping. This is the preferred technical solution. A Permit to Take Water (PTTW) would likely not be required since the area to be dewatered is surrounded by a cutoff. The sheet piling should be driven at least 0.5 metres into the dense to very dense sandy silt. Driving conditions within the sand and gravel deposit will be difficult, especially in dense layers where cobbles are present.

7.0 MISCELLANEOUS

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

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.
Geotechnical Engineer

Philip R. Bedell, P. Eng.
Principal

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

TP/DUP/PRB/FJH/cr
n:\active\2006\1130 - geotechnical\1130-100\06-1130-185-2 delcan - gwp 336-97-00 fdns - hwy 24\reports\0611301852-r01 bridge replacement\0611301852-r01 dec 19 08 - (final) parts a&b whitemans creek bridge replacement hwy 24.doc

TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Whitemans Creek Bridge Replacement
Site 1-85
Highway 24 Rehabilitation
GWP 336-97-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Spread footings supported on native sand and gravel	• Not considered to be practical	• Less expensive than deep foundation	• Difficult construction due to fill depths which are 3.5 to 5.0 metres and groundwater table within sand and gravel. • Geotechnical resistances limited due to groundwater table at or just above surface of the sand and gravel • Dewatering of sand and gravel deposit may be required. Artesian water pressure at depth.	• Costs not easily quantifiable due to depth of excavation and possible need for dewatering but expected to be less than deep foundations	• If spread footings are designed construction will be difficult with high likelihood of construction delays.
End bearing steel H-pile foundations driven to refusal in bedrock	• Feasible	• High bearing resistance • Negligible settlement	• Possibility of pile tip damage during driving in rock • Care must be taken with driving of battered piles to ensure that they do not deflect along the bedrock surface • More costly than shallow footings	• Estimated cost $35,000 per integral abutment • More expensive than shallow foundations	• Possible pile tip damage if piles are not adequately protected while driving in bedrock

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	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 $= \sigma'_p/\sigma'_{vo}$

(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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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

Golder Associates
London, Ontario

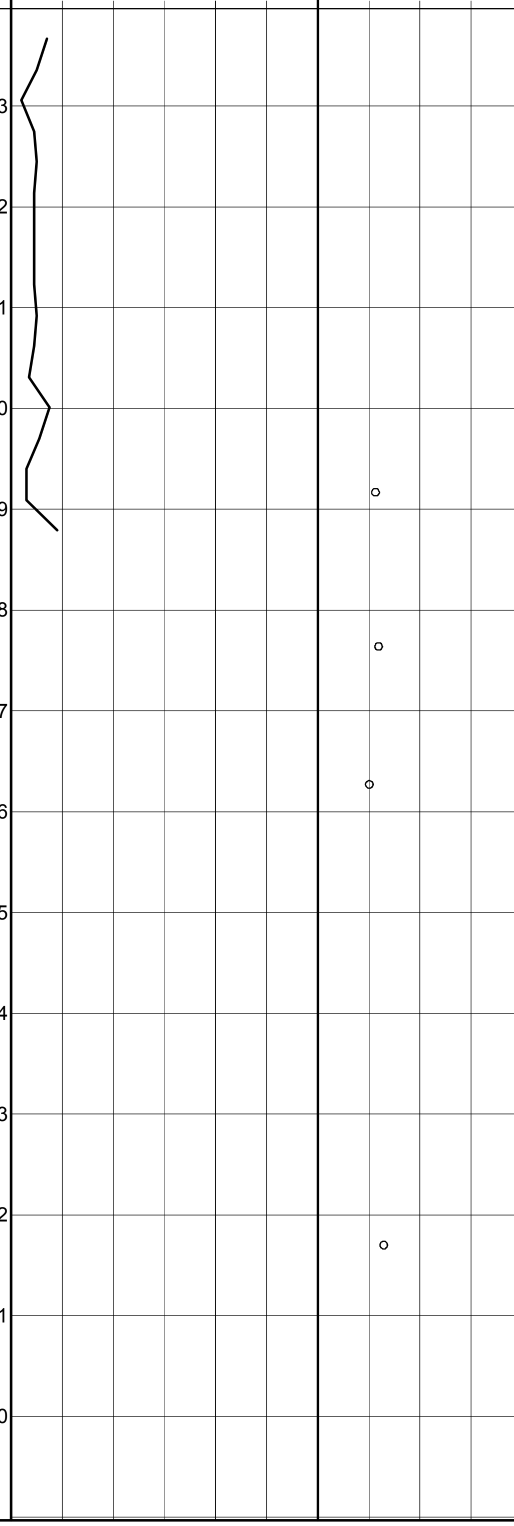
RECORD OF BOREHOLE No 1

METRIC

PROJECT 06-1130-185-2
G.W.P. 336-97-00
LOCATION N 4777852.6 ;E 234264.4
ORIGINATED BY M.A.
DIST 1 HWY 24
BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS/NW CASING & TRICONE
COMPILED BY BRS
DATUM GEODETIC
DATE June 14, 2007 - June 18, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
223.97	GROUND SURFACE							
0.10	TOPSOIL, sandy, some gravel Brown							
	FILL, silty sand, trace to some gravel, trace clay, trace topsoil, Loose Brown	1	SS	9				
		2	SS	6				
221.84 2.13	FILL, sand and gravel, trace to some silt, trace clay Loose to very loose Brown becoming grey below about elev. 219.6m	3	SS	6				
		4	SS	4				
		5	SS	3				
		6	SS	6	35	47	12	6
218.94 5.03	SAND AND GRAVEL, fine to coarse, trace silt, cobbles Compact to dense Brown	7	SS	17				
		8	SS	29	63	29	6	2
217.02 6.95	SANDY SILT, trace gravel, silt and clayey silt seams	9	SS	30				
216.65 7.32	Dense to very dense Brown becoming grey below about elev. 217.0m	10	SS	32	0	13	66	21
215.89 8.08	SILT, some sand, clayey silt seams Dense Grey							
215.37 8.60	SANDY SILT, trace gravel, silt and clayey silt seams Very dense Grey	11	SS	67				
8.84	FINE SAND, some silt Very dense Grey							
	SANDY SILT, some clay, clayey silt seams Compact to very dense Grey	12	SS	58				
		13	SS	51				
		14	SS	28	0	43	39	18
210.94 13.03	SAND, fine to medium, trace silt Very dense Grey	15	SS	100/ 175mm				
210.07 13.90	Grey, Shaley DOLOSTONE							

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (223 to 210); 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.



Continued Next Page

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

LDN_MTO_01 06-1130-185-2.GPJ LDN_MTO.GDT 12/19/08

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 1

METRIC

PROJECT 06-1130-185-2
G.W.P. 336-97-00
LOCATION N 4777852.6 ;E 234264.4
ORIGINATED BY M.A.
DIST 1 HWY 24
BOREHOLE TYPE POWER AUGER/HOLLOW STEM AUGERS/NW CASING & TRICONE
COMPILED BY BRS
DATUM GEODETIC
DATE June 14, 2007 - June 18, 2007
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
	Grey, Shaley DOLOSTONE		16	SS	100/ 0mm						
			17	WS	-		208				
							207				
206.36			18	WS	-						
17.61	END OF BOREHOLE Groundwater encountered at about elev. 218.79m during drilling on June 14, 2007. Groundwater measured in standpipe at elev. 218.86m on June 19, 2007. Groundwater measured in piezometer at elev. 219.88m on June 19, 2007. Groundwater measured in standpipe at elev. 218.89m on July 6, 2007. Groundwater measured in piezometer at elev. 220.01m on July 6, 2007.										

LDN_MTO_01 06-1130-185-2.GPJ LDN_MTO.GDT 12/19/08

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 2

 METRIC

PROJECT 06-1130-185-2
G.W.P. 336-97-00
LOCATION N 477900.3 ;E 234232.5
ORIGINATED BY M.A.
DIST 1 HWY 24
BOREHOLE TYPE POWER AUGER/HOLLOW STEM/NW CASING & CORE
COMPILED BY BRS
DATUM GEODETIC
DATE June 19, 2007 - June 20, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
223.88	GROUND SURFACE					
0.09	TOPSOIL, sandy, trace gravel Brown					
	FILL, sand and gravel, some silt, trace topsoil Loose Brown	1	SS	4	223	
222.51 1.37	FILL, fine to coarse sand, some gravel, trace silt Dense Brown	2	SS	33	222	
221.90 1.98	FILL, fine to coarse sand and gravel, trace silt, concrete rubble Loose Brown	3	SS	6	221	
		4	SS	4		
220.37 3.51	SAND AND GRAVEL, fine to coarse, some silt, cobbles, trace clay below elev. 218.9m Compact to dense Brown	5	SS	19	220	
		6	SS	32	219	50 35 12 3
218.36 5.52	SANDY SILT, trace gravel, trace to some clay Dense to very dense Grey	7	SS	46	218	
		8	SS	67		3 44 37 16
217.17 6.71	CLAYEY SILT with sandy silt seams and layers Hard Grey				217	
216.84 7.04	SANDY SILT with clayey silt seams and layers, trace gravel Very dense to dense Grey	9	SS	66		
		10	SS	46	216	
		11	SS	59		0 36 41 23
215.04 8.84	CLAYEY SILT, trace sand Very stiff Grey	12	SS	22	215	
					214	
		13	SS	16	213	0 4 51 45
212.30 11.58	SANDY SILT (TILL), trace gravel, trace clay Dense Grey	14	SS	32	212	
210.93 12.95	Slightly weathered, laminated, light grey, fine grained, low porosity, weak shaley DOLOSTONE	15	SS	100/ 0mm	211	
		16	NQ RC	-		T.C.R. 72 (%), S.C.R. 44 (%), R.Q.D. 0 (%)
210.08 13.80	Slightly weathered, laminated, light grey to white, fine grained, moderate porosity, weak gypsiferous DOLOSTONE with gypsum nodules	17	NQ RC	-	210	T.C.R. 100 (%), S.C.R. 87 (%), R.Q.D. 70 (%)
					209	

Water content plot (WATER CONTENT (%) 10 20 30; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L) shows data at samples 6, 8, 11 and 13. 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. UNIT WEIGHT γ kN/m³.

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 2

METRIC

PROJECT 06-1130-185-2
G.W.P. 336-97-00
LOCATION N 477900.3 ;E 234232.5
ORIGINATED BY M.A.
DIST 1 HWY 24
BOREHOLE TYPE POWER AUGER/HOLLOW STEM/NW CASING & CORE
COMPILED BY BRS
DATUM GEODETIC
DATE June 19, 2007 - June 20, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.55			17					100	87	70			
15.33 / 207.48	Slightly weathered, laminated, light grey to white, fine grained, moderate porosity, weak gypsiferous DOLOSTONE with gypsum nodules		18	NQ RC	-		208	98	79	79			
16.40	END OF BOREHOLE												

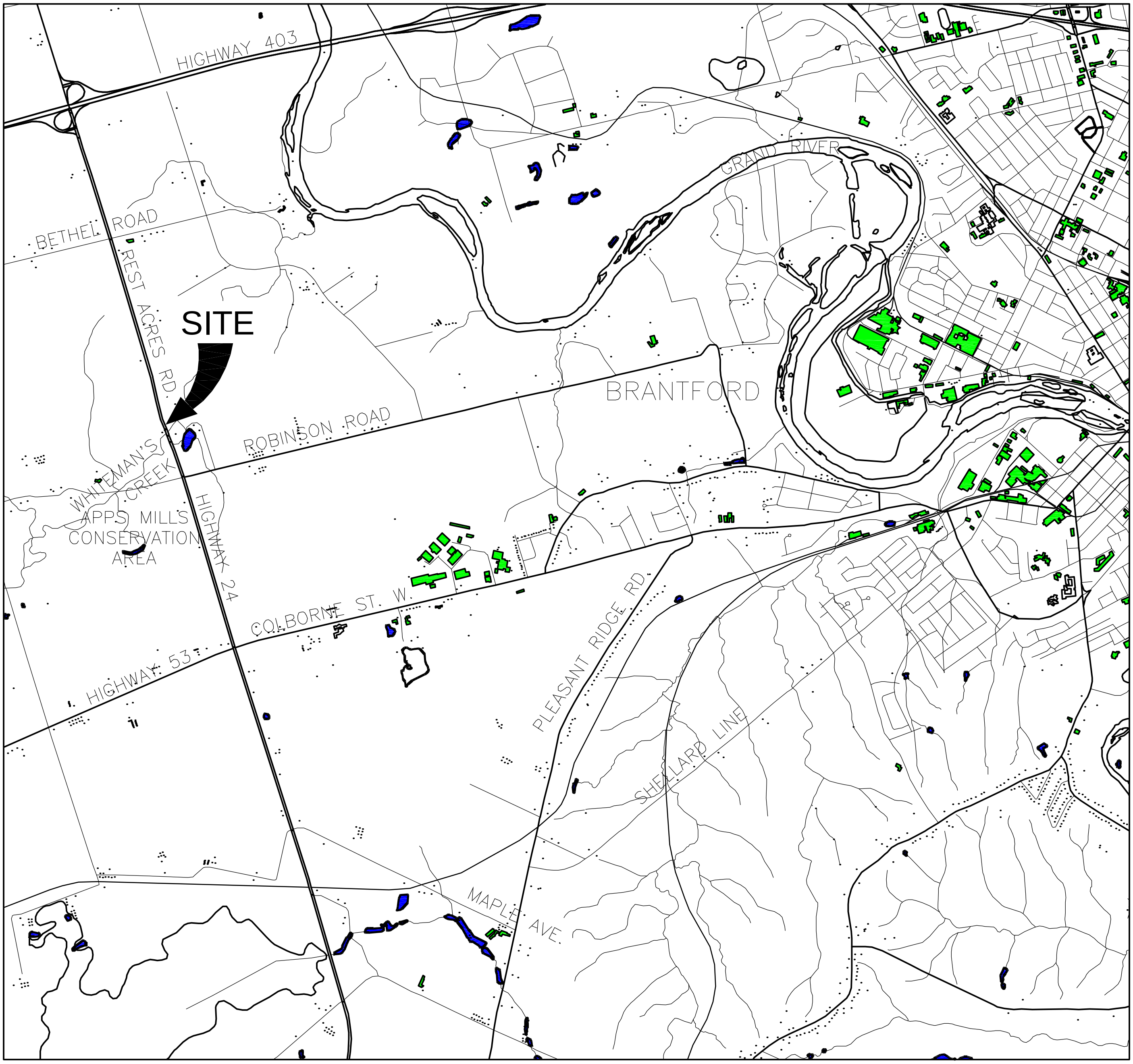
Groundwater encountered at about elev. 219.9m during drilling on June 19, 2007.

Groundwater measured in piezometer at elev. 220.48m on June 20, 2007.

Groundwater measured in piezometer at elev. 220.63m on July 6, 2007.

Chart headings: 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

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



SCALE IN METERS

0 1000 2000

1:50000

REFERENCE

DRAWING BASED ON CANMAP STREET FILES V2005.4

NOTES

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

ALL LOCATIONS ARE APPROXIMATE ONLY.

PROJECT

WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P. 336-97-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-185-2		FILE No.	061130185-F01001
			SCALE AS SHOWN	REV. 0
CADD	LMK	Oct. 28/08		
CHECK			FIGURE 1	

DRAWING FILE: 061130185-F01001.DWG Plot Date: Dec 19, 2008 - 1:54pm

MINISTRY OF TRANSPORTATION, ONTARIO

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

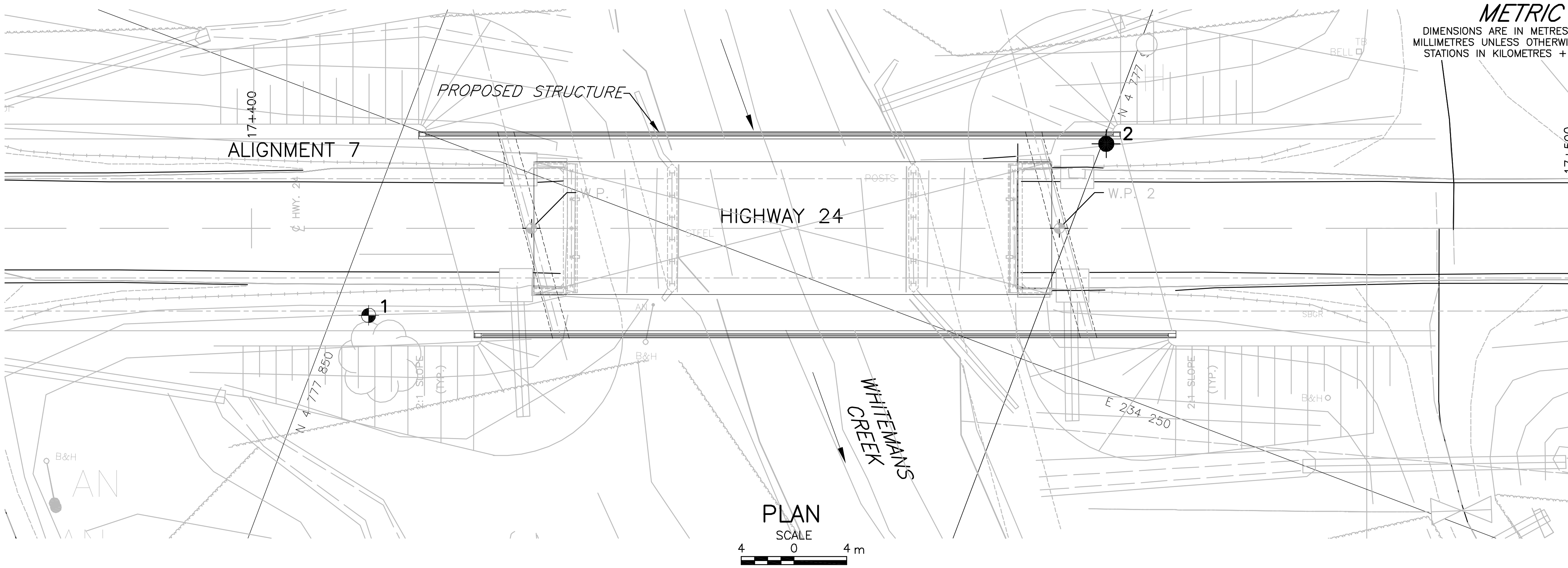
CONT No.
WP No. 336-97-00

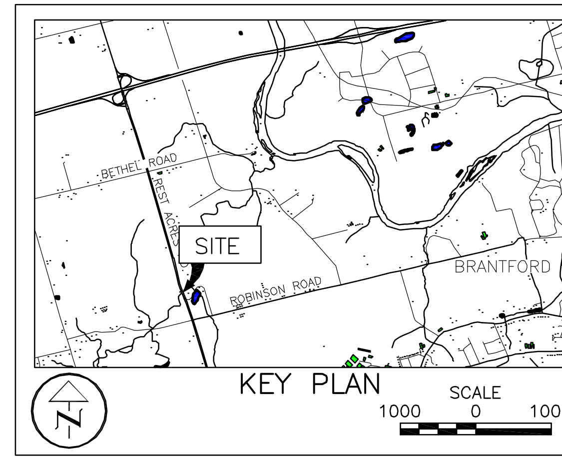
SHEET

HIGHWAY 24 REHABILITATION
WHIITEMANS CREEK BRIDGE REPLACEMENT

BOREHOLE LOCATION AND SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

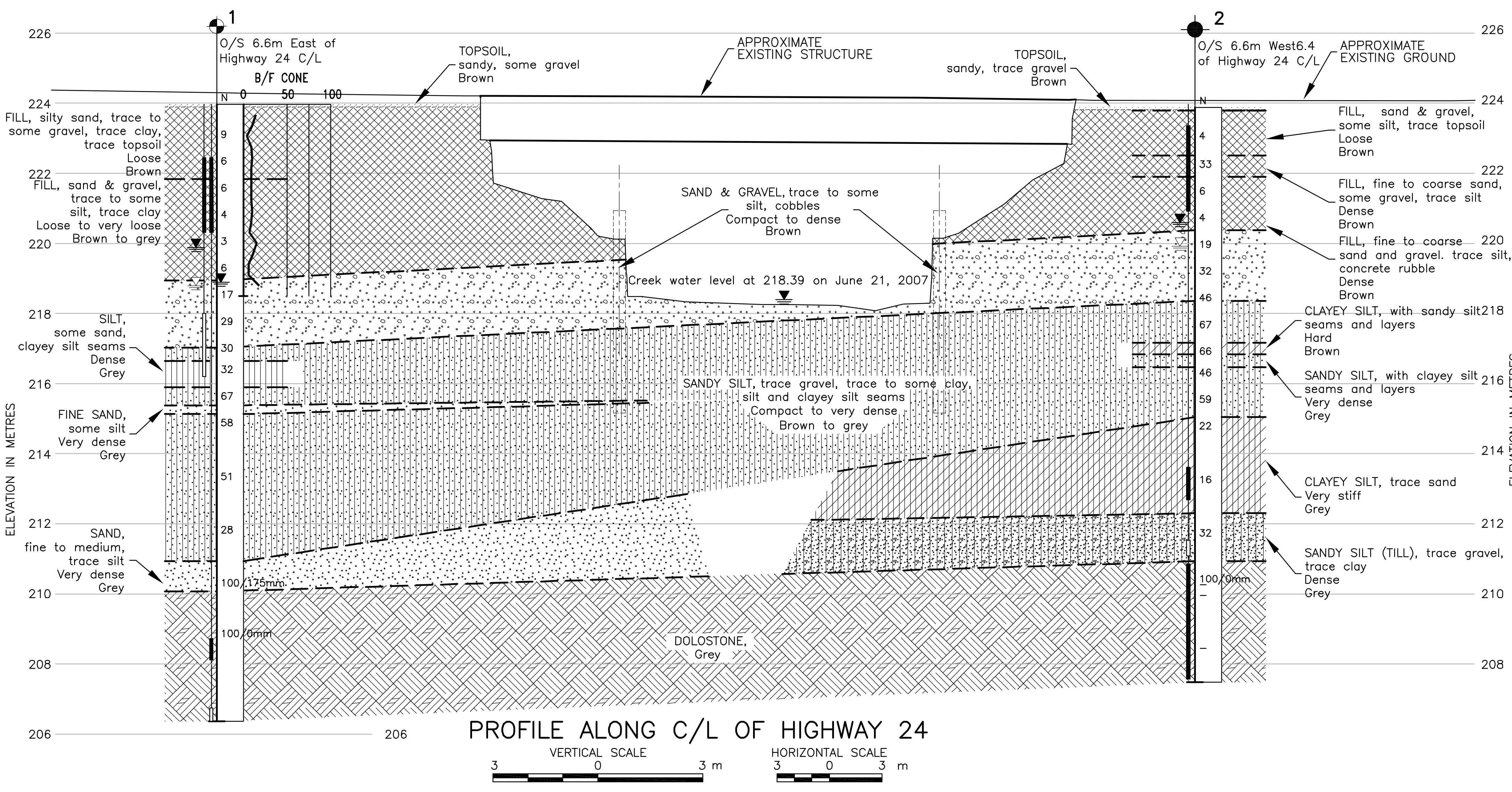




LEGEND

Symbol	Description
	Borehole – Current Investigation
	Borehole and Cone – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL in piezometer, measured on July 6, 2007
	WL encountered during drilling
DRY	Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM Zone 11) NORTHING	EASTING
1	223.97	4 777 852.6	234 264.4
2	223.88	4 777 900.3	234 232.5



NOTES

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

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

REFERENCE

Base plans provided in digital format by Delcan on July 5, 2007.

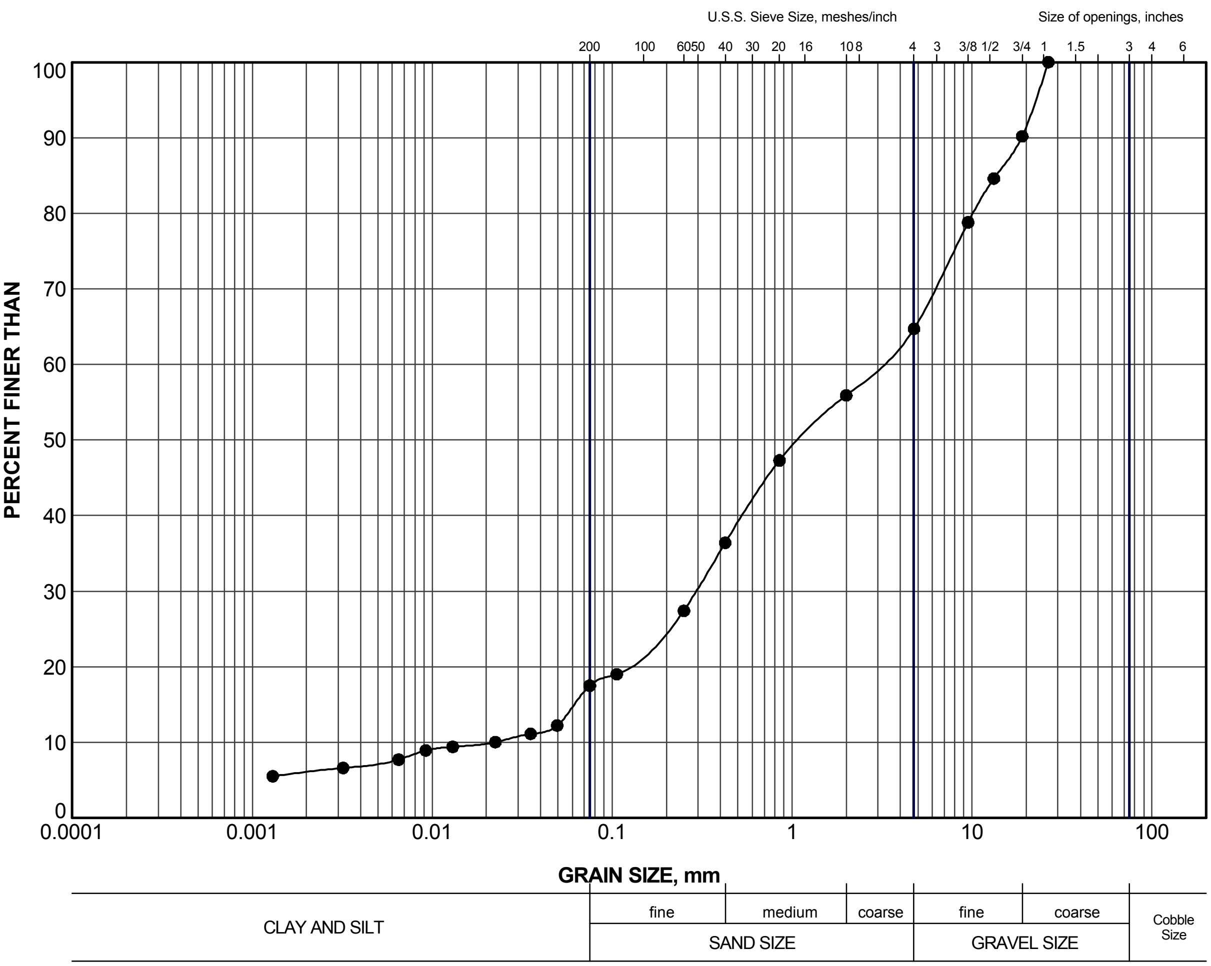
NO.	DATE	BY	REVISION

Geocres No.	40P1-100				
HWY.	24	PROJECT NO. 06-1130-185-2-1		DIST.	
SUBM'D.	DUP	CHKD.	DUP	DATE: Dec. 19/08	SITE: 1-85
DRAWN:	DCH	CHKD.		APPD.	DWG. 1

PLOT DATE: December 19, 2008

APPENDIX A

LABORATORY TEST DATA



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	6	219.2

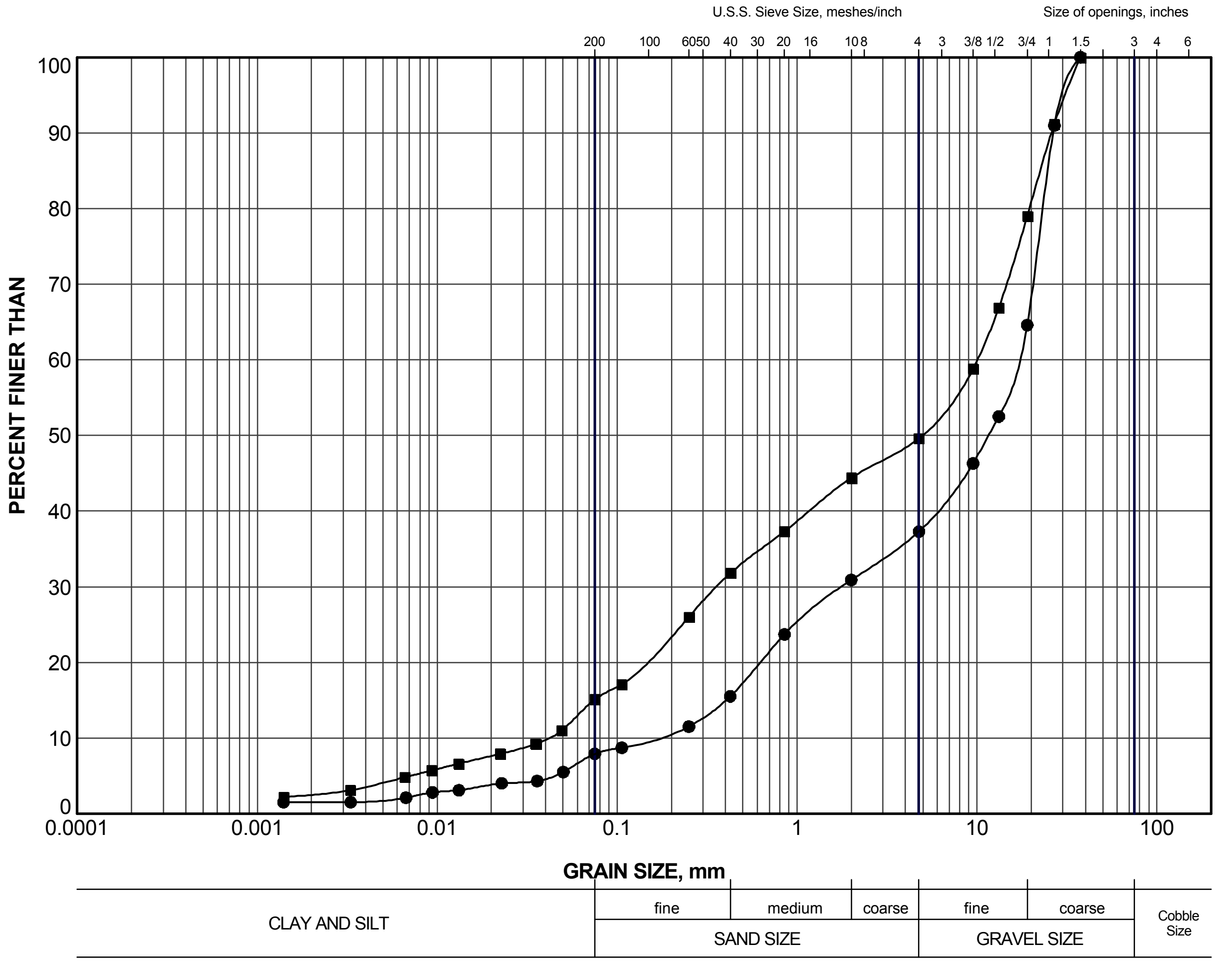
PROJECT
WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P 336-97-00

TITLE
GRAIN SIZE DISTRIBUTION
FILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-185-2		FILE No.	061130185-2-F010A1
			SCALE N/A	REV.
DRAWN	LMK	Oct 27/08	**FIGURE A-1**	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	8	217.6
■	2	6	219.1

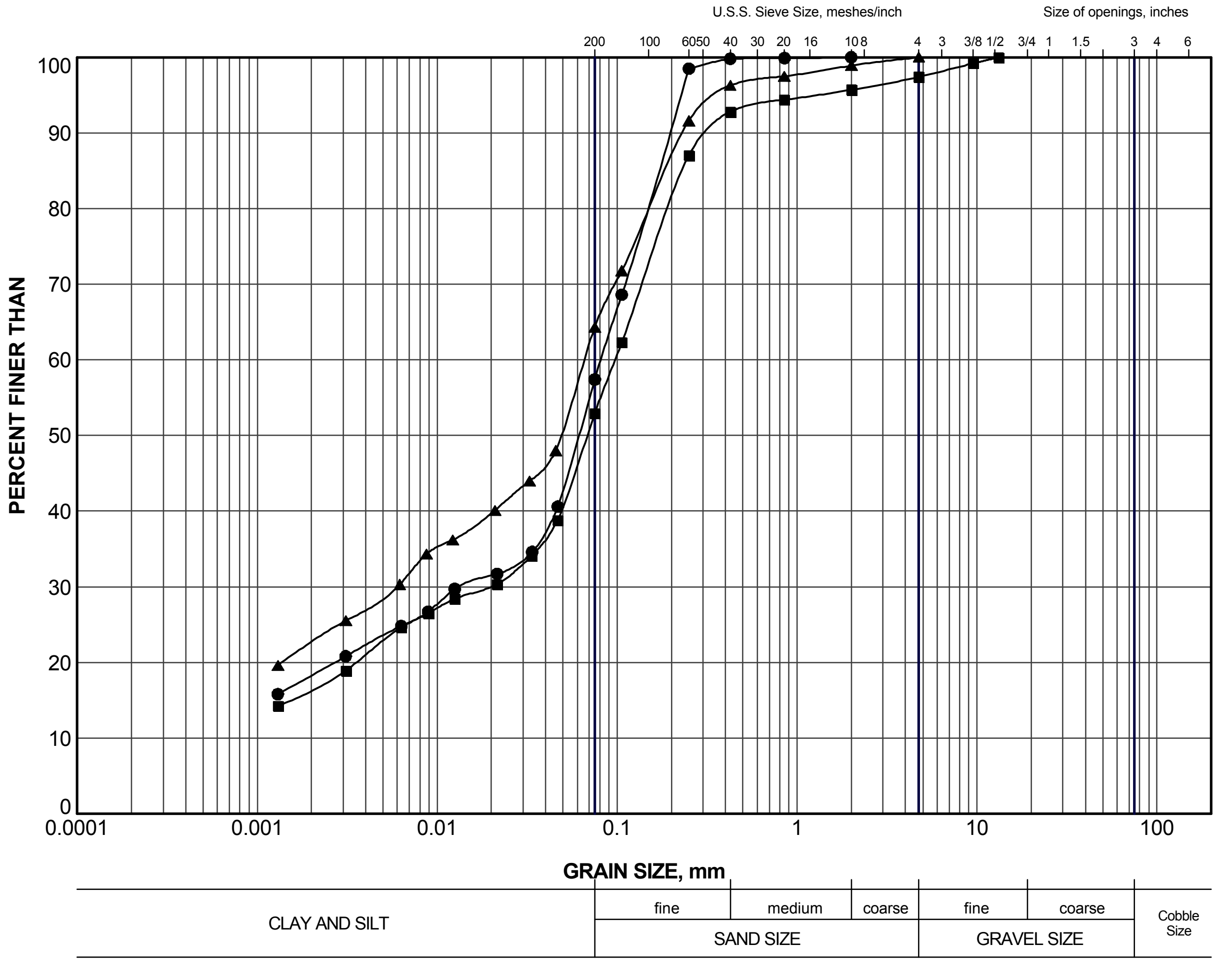
PROJECT
WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P 336-97-00

TITLE
GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-185-2	FILE No.	061135185-2-F010A2
		SCALE	N/A REV.
DRAWN	BRS Oct 27/08		
CHECK		FIGURE A-2	

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	14	211.7
■	2	8	217.6
▲	2	11	215.4

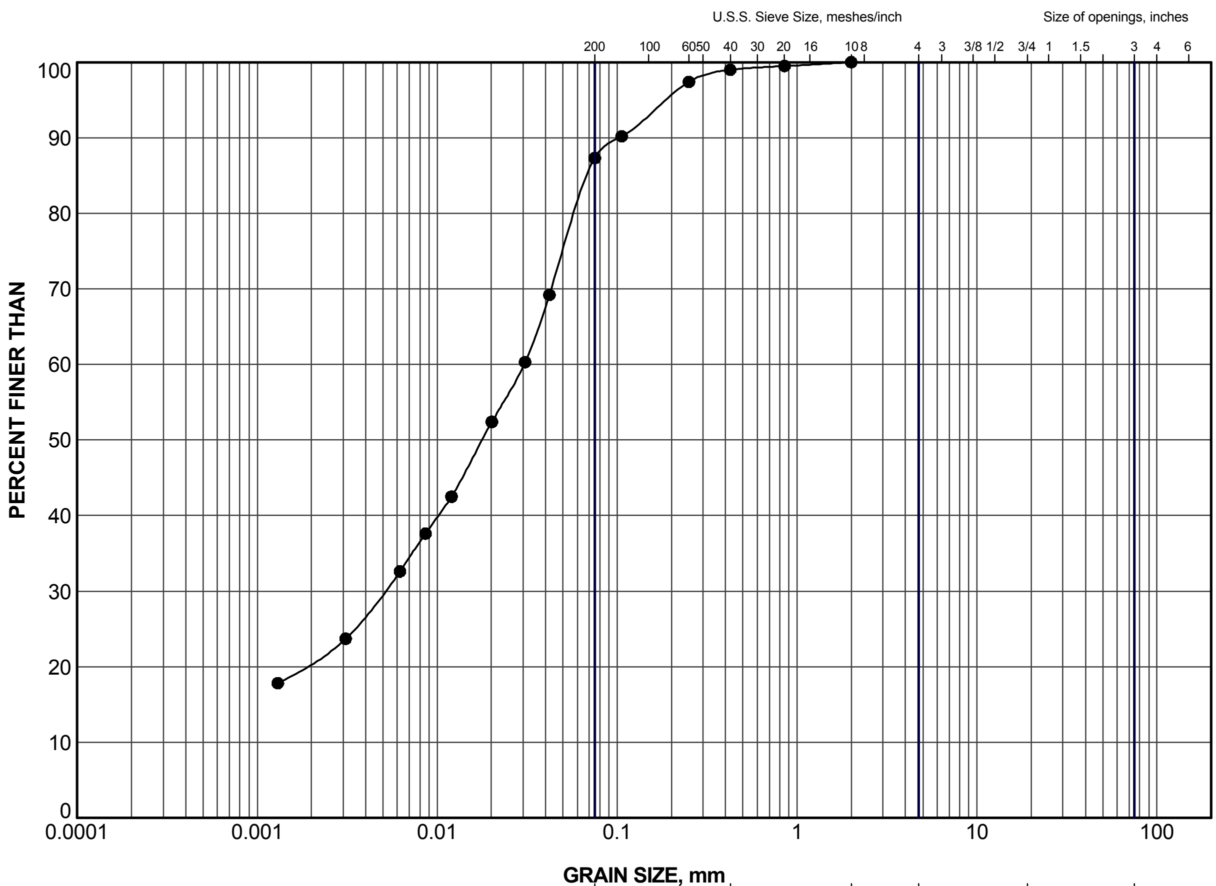
PROJECT
WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P 336-97-00

TITLE
GRAIN SIZE DISTRIBUTION
SANDY SILT, some clay

PROJECT No.	06-1130-185-2		FILE No.	061130185-2-F010A3
			SCALE N/A	REV.
DRAWN	LMK	Oct 31/08	**FIGURE A-3**	
CHECK				

Golder Associates
LONDON, ONTARIO

LDN_MTO_NEW GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	10	216.3

PROJECT
WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P 336-97-00

TITLE
GRAIN SIZE DISTRIBUTION
SILT

PROJECT No.	06-1130-185-2	FILE No.	061130185-2-F010A4
		SCALE	N/A REV.
DRAWN	BRS	Oct 27/08	**FIGURE A-4**
CHECK			

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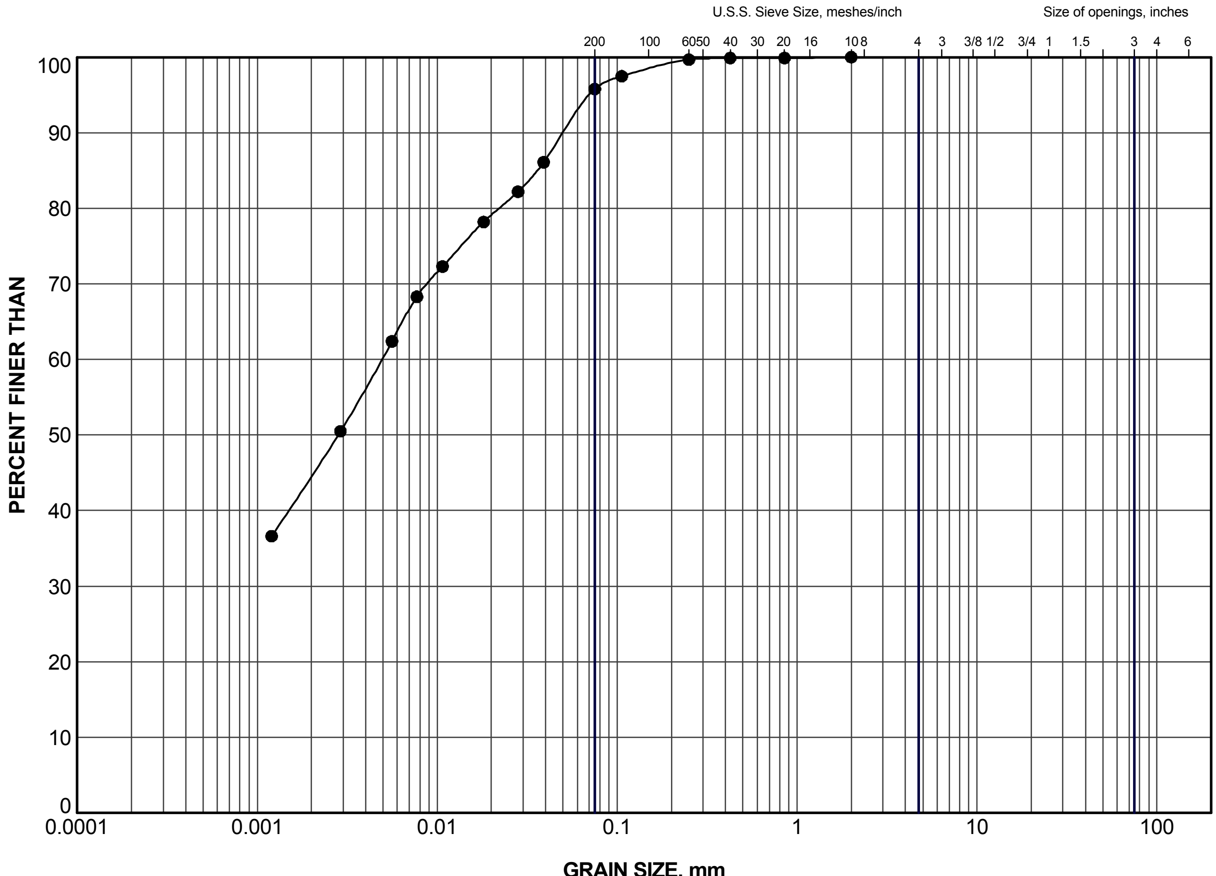
LDN_MTO_NEW GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	2	13	213.1

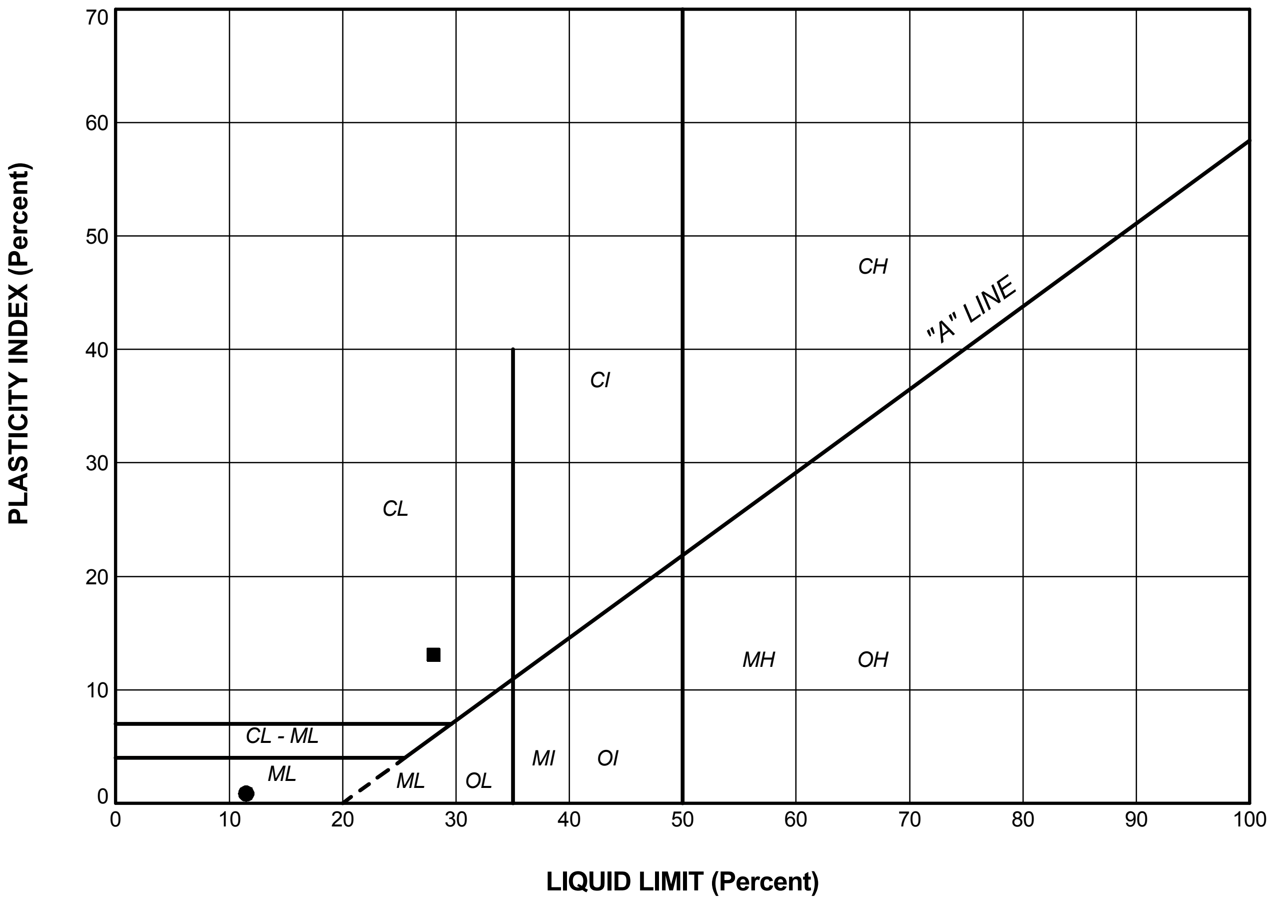
PROJECT
WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P 336-97-00

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-185-2		FILE No.	061130185-2-F010A5
			SCALE N/A	REV.
DRAWN	BRS	Oct 27/08	**FIGURE A-5**	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
SANDY SILT					
●	2	8	14.3	10.7	3.7
CLAYEY SILT					
■	2	13	28.0	14.9	13.1

PROJECT	WHITEMANS CREEK BRIDGE REPLACEMENT HIGHWAY 24 REHABILITATION G.W.P 336-97-00
TITLE	**PLASTICITY CHART**

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-185-2		FILE No.	061130185-2-F010A6
			SCALE N/A	REV.
DRAWN	LMK	Dec. 19/08	**FIGURE A-6**	
CHECK				

MTO_PI GLDR_LDN.GDT

APPENDIX B

SITE PHOTOGRAPHS

SITE PHOTOGRAPHS

Photo 1: Looking north along Highway 24 at Whitemans Creek Bridge from south abutment.

Photo 2: View of Whitemans Creek looking downstream at west side of bridge.

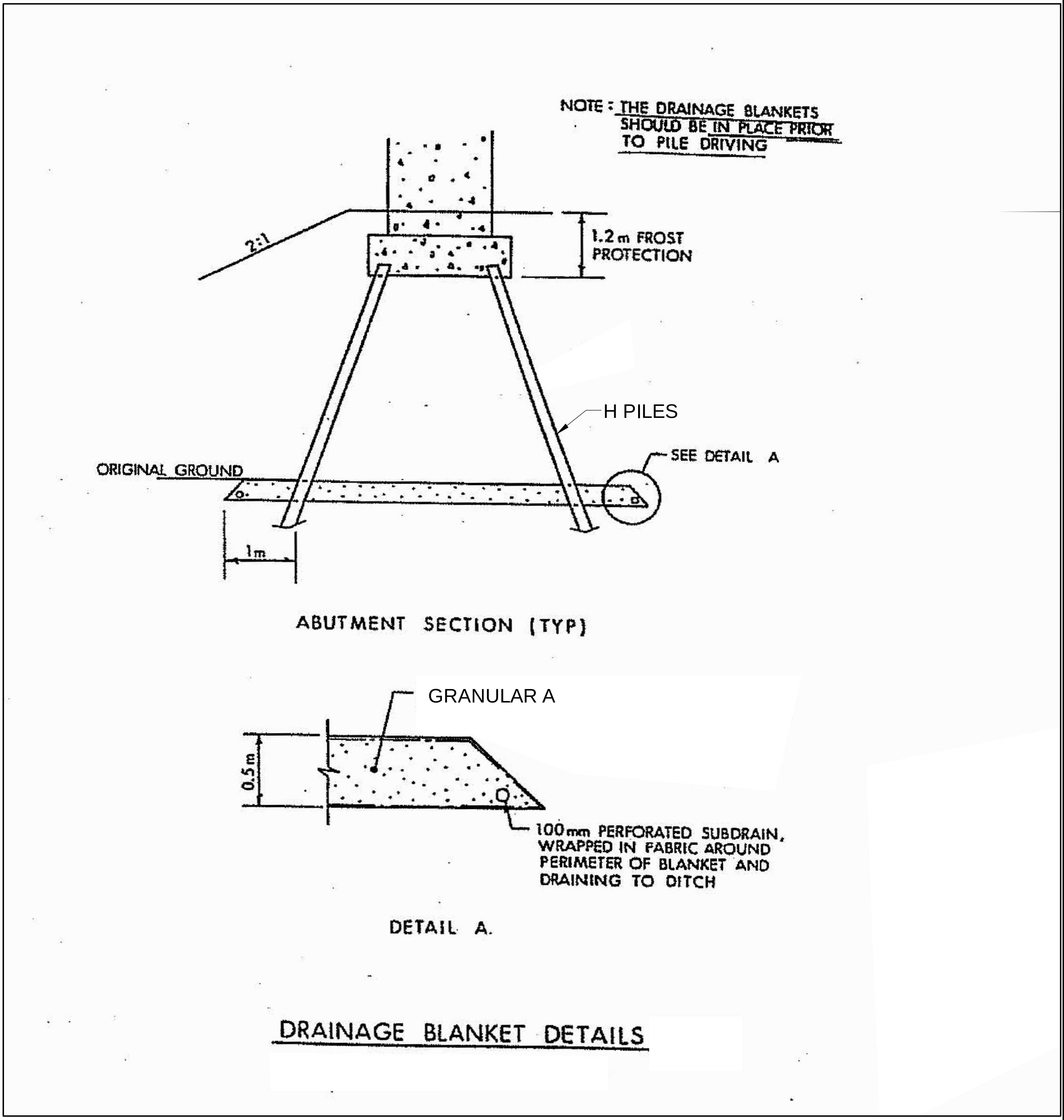
SITE PHOTOGRAPHS

Photo 3: View of south abutment and pier.

Photo 4: View of north abutment upstream of Whitemans Creek bridge.

APPENDIX C

DRAINAGE BLANKET DETAILS



NOTES

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

ALL LOCATIONS ARE APPROXIMATE ONLY.

PLAN NOT TO SCALE.

REFERENCE

SCHEMATIC BASED ON DRAWING "DRAINAGE BLANKET DETAILS FOR ABUTMENTS AND PIERS" PROVIDED BY MTO FOUNDATIONS OFFICE.

PROJECT
WHITEMANS CREEK BRIDGE REPLACEMENT
HIGHWAY 24 REHABILITATION
G.W.P. 336-97-00

TITLE

DRAINAGE BLANKET DETAILS

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-185-2	FILE No.	061130185-F010C1
		SCALE	N.T.S
		REV.	0
CADD	BRS	Dec. 16/08	
CHECK			

FIGURE C-1

Drawing file: 061130185-F010C1.dwg Dec 19, 2008 - 1:55pm