FOUNDATION INVESTIGATION
AND DESIGN REPORT
CULVERT REPLACEMENTS
HIGHWAY 62 FROM 5.3 KM NORTH OF CLEVELAND ROAD
TO 300 M SOUTH OF COUNTY ROAD 620
BANCROFT, ONTARIO
G.W.P. 66-99-00

Submitted to:

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PART A

FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by GENIVAR on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detailed design for the rehabilitation of Highway 62 from 5.3 km north of Cleveland Road to 300 m south of County Road 620, south of Bancroft, Ontario.

This report addresses the potential replacement of seven drainage culverts located within the project limits, as follows:

Culvert Number	*Approximate Station*	*Township*
1	21+369	Tudor and Cashel
2	23+418	Tudor and Cashel
3	24+124/24+126	Tudor and Cashel
4	24+320/24+322	Tudor and Cashel
5	25+057	Tudor and Cashel
6	25+529	Tudor and Cashel
7	17+379	Limerick

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal for Agreement No. 4006-E-0027, and outlined in Golder's Proposal No. P71-1508 dated May 4, 2007, which forms Section 6.8 of GENIVAR's Technical Proposal for this assignment.

2.0 SITE DESCRIPTION

The seven culvert sites addressed in this report are located along Highway 62 between 5.3 km north of Cleveland Road and 300 m south of County Road 620, south of Bancroft, Ontario. The location, culvert dimension and type, and approximate highway embankment height for each of the sites is summarized in the following table:

Approximate Station	*Existing/Original Culvert Dimensions*	*Approximate Embankment Height*	*Invert Elevation*
21+369	900 mm diameter CSP 28 m long	3 m	307.4 m (west) 307.2 m (east)
23+418	1,200 mm diameter CSP 29.6 m long	3.7 m	308.7 m (west) 308.8 m (east)
24+124/24+126	1,800 mm diameter CSP 23.1 m long	1.5 m	310.3 m (west) 310.1 m (east)
24+320/24+322	1,800 mm diameter CSP 23.1 m long	1.5 m	310.5 m (west) 310.9 m (east)
25+057	1,200 mm diameter CSP 21.9 m long	1.5 m	309.6 m (west) 310.1 m (east)
25+529	1,200 mm diameter CSP 21.6 m long	2.2 m	311.9 m (west) 311.8 m (east)
17+379	900 mm diameter CSP 20.2 m long	1.3 m	345.6 m (west) 345.5 m (east)

The natural ground surface within the swamps at the sites of the six southerly culverts in Tudor and Cashel Township varies from about Elevation 308 m to 313 m, and the natural ground surface at the site of the northernmost culvert in Limerick Township (Station 17+379) is at about Elevation 346 m. The existing Highway 62 embankment is between about 1.3 m and 3.7 m in height at the culvert locations, relative to the adjacent ground surface.

The culvert sites are located in flat terrain, within poorly drained, swampy areas. The culverts are generally submerged (Station 21+369, 23+418, and 25+057), partially buried (Station 24+124/24+126), and/or in poor condition (Stations 25+529 and 17+379). The original 1.8 m diameter pipe culvert at Station 24+320/24+322 failed and was abandoned in place by grouting and replaced with multiple small diameter pipe culverts; the original culvert is now buried and unable to be found.

The culverts at Stations 23+418, 24+124/24+126, 24+320.24+322, 25+057 and 25+529 are located within sections of Highway 62 that have a history of poor performance; these embankment areas have been surcharged and repaired under a previous contract, and are currently exhibiting severe distortion and rutting.

3.0 INVESTIGATION PROCEDURES

A site investigation was carried out in October and November 2007, at which time 21 boreholes (Boreholes 07-1 to 07-21) were advanced to investigate the subsurface conditions at each of seven culvert sites. Three boreholes were advanced at each culvert site; the borehole locations are shown on Drawings 1 to 5.

The boreholes were drilled to depths ranging from 3.3 m to 13.8 m below the ground surface at the borehole locations. The majority of the boreholes were drilled with a truck-mounted CME-75 drill rig supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario, using 200 mm outside diameter hollow stem augers. Two boreholes (Boreholes 07-7 and 07-8) were drilled with portable drilling equipment supplied and operated by Ohlmann Geotechnical Services (OGS) Inc. of Almonte, Ontario, using 50 mm diameter solid stem augers. Six of the boreholes were extended 2.5 m to 3 m into bedrock using NQ-coring equipment. The remainder of the boreholes were terminated within the overburden soils, either when sampler and auger refusal was met, or when material that would offer resistance to settlement and instability of the culvert and embankment was encountered.

Soil samples were obtained from boreholes at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure.

Groundwater conditions were observed in the open boreholes during and immediately following completion of drilling operations. A single standpipe piezometer was installed in one borehole at each culvert site (Boreholes 07-2, 07-5, 07-8, 07-11, 07-13, 07-17 and 07-20) to permit monitoring of the groundwater level at culvert sites. Each standpipe piezometer consists of a 20 mm diameter slotted screen, 0.6 m in length, installed within a filter sand pack, and sealed with bentonite; soil cutting materials were replaced within the bentonite seals, where it was possible to do so. Details of the piezometer installations are shown on the borehole records appended to this report. For boreholes in which piezometers were not installed, the boreholes were backfilled to ground surface using bentonite pellets, in places mixed with the soil cutting materials that had been removed from the borehole. All seven piezometers were decommissioned in June 2009 by removing the protective surface casing and standpipe and sealing up to the ground surface with bentonite.

The field work was supervised by one of Golder’s senior technicians, who located the boreholes, directed the drilling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder’s Ottawa geotechnical laboratory where the samples underwent further visual examination and geotechnical classification testing (including water content, Atterberg limits, grain size distribution, and organic content). All of the laboratory tests were carried out according to MTO and/or ASTM standards as appropriate.

The locations (northing and easting coordinates referenced to NAD83 MTM Zone 12) and the ground surface elevations (relative to geodetic datum) at the borehole locations were provided by GENIVAR. The northing and easting coordinates and ground surface elevation are included on the borehole records and are summarized in the following table:

Culvert Location	***Borehole Number***	***MTM NAD83 Northing (m)***	***MTM NAD83 Easting (m)***	***Ground Surface Elevation (m)***
Station 21+369	07-4	4,958,474.0	216,362.9	309.8
	07-5	4,958,485.0	216,373.5	309.7
	07-6	4,958,475.7	216,370.5	310.2
Station 23+418	07-7	4,960,517.8	216,158.7	309.4
	07-8	4,960,520.5	216,186.5	309.9
	07-9	4,960,524.3	216,174.2	312.4
Station 24+124/24+126	07-10	4,961,231.3	216,098.6	312.2
	07-11	4,961,231.4	216,109.5	312.3
	07-12	4,961,225.1	216,106.7	312.6
Station 24+320/24+322	07-13	4,961,414.4	216,080.3	312.0
	07-14	4,961,414.6	216,092.9	312.0
	07-15	4,961,421.2	216,087.6	312.4
Station 25+057	07-16	4,962,141.9	215,965.0	312.0
	07-17	4,962,144.4	215,975.8	312.2
	07-18	4,962,137.5	215,974.2	312.5
Station 25+529	07-19	4,962,589.9	215,813.5	313.4
	07-20	4,962,593.7	215,824.9	313.5
	07-21	4,962,585.1	215,824.3	313.8
Station 17+379	07-1	4,971,902.1	209,206.4	347.1
	07-2	4,971,902.5	209,218.7	346.8
	07-3	4,971,896.3	209,211.0	347.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area for this assignment lies within the physiographic region known as the Algonquin Highlands, as delineated by Chapman and Putnam in *The Physiography of Southern Ontario*[1].

The Algonquin Highlands region is characterized by frequent outcrops of granite and other strong Precambrian bedrock. The outcrops can extend as high as 160 m above the surrounding land. The thickness of soils over the bedrock can vary greatly over short distances, with many of the valleys between the bedrock outcrops floored with outwashed sand, silt and gravel. Several areas within this region have deeper deposits of glacial till with few bedrock outcrops.

4.2 Subsurface Conditions

The soil and groundwater conditions encountered in boreholes and the results of in situ and laboratory tests are shown on the borehole records and on Figures 1 to 5, appended to this report. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic sections shown on Drawings 1 to 5 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

A brief overview of the subsurface conditions at each of the seven culvert sites along Highway 62 is provided in the following table:

Culvert Location	*Borehole Nos.*	*General Subsurface Conditions*
Station 21+369	07-4 07-5 07-6	Boreholes 07-4, 07-5 and 07-6 were drilled through the Highway 62 embankment, and encountered between 2.7 m and 3.3 m of sand and gravel to crushed rock fill; Borehole 07-6 was terminated in the fill. As shown in the interpreted stratigraphic section on Drawing 1, in Boreholes 07-4 and 07-5, the embankment fill is underlain by a deposit of peat that is about 1.5 m to 1.9 m in thickness; the surface of the peat was encountered at Elevations 306.8 m and 307.0 m. The peat is underlain by thin sand to silty sand deposits, overlying gabbro bedrock which was encountered at depths of 4.7 m and 5.1 m below the Highway 62 grade, at Elevations 305.1 m and 304.4 m.

[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

Culvert Location	*Borehole Nos.*	*General Subsurface Conditions*
Station 23+418	07-7 07-8 07-9	Borehole 07-9 was drilled through the Highway 62 pavement and encountered approximately 7.2 m of loose to compact fill consisting of loose to compact sand, silty sand, or cobbles and boulders containing some sand and gravel. Borehole 07-7 and 07-8 were drilled using portable equipment beyond the highway shoulder near the embankment toe, and encountered about 3.1 m and 2.7 m of loose to very loose sand to gravelly sand fill, respectively. In all three boreholes, as shown in the interpreted stratigraphic section on Drawing 2, the fill is underlain by a 1.6 m to 3.6 m thick deposit of peat; the surface of the peat was encountered at about Elevation 306.3 m under the west toe, Elevation 305.3 under the main embankment, and Elevation 307.2 m under the east toe. The peat is underlain in Boreholes 07-7 and 07-9 by a 0.8 m to 2.0 m thick deposit of firm to stiff grey clayey silt to silty clay. Below this clayey silt to silty clay, and below the peat in Borehole 07-8, the boreholes encountered loose to very dense sand, silty sand, sand and gravel, and silty sand till. The boreholes were terminated within the dense to very dense soils at depths of 9.3 m to 13.8 m (Elevations 298.7 m to 300.3 m).
Station 24+124/ 24+126	07-10 07-11 07-12	Boreholes 07-10, 07-11 and 07-12 were drilled through the Highway 62 embankment and encountered 4.3 m to 5.2 m of very loose to compact sand and gravel to sand fill. In all three boreholes, as shown in the interpreted stratigraphic section on Drawing 3, the fill is underlain by 3.0 m to 4.0 m of peat; the surface of the peat was encountered between Elevations 307.4 m and 308.0 m. The peat is underlain by a 0.5 m to 1.2 m thick deposit of soft to firm clayey silt in Boreholes 07-10 and 07-11. The clayey silt in these two boreholes and the peat in Borehole 07-12 are underlain by loose sand to silty sand, in which all three boreholes were terminated due to sampler and/or auger refusal at a depth of about 8.5 m to 9.8 m (Elevation 302.4 m to 303.8 m).
Station 24+320/ 24+322	07-13 07-14 07-15	Boreholes 07-13, 07-14 and 07-15 were drilled through the Highway 62 embankment and encountered 3.4 m to 5.0 m of very loose to dense sand and gravel, sand and crushed rock fill. In all three boreholes, as shown in the interpreted stratigraphic section on Drawing 3, the fill is underlain by a 2.5 m to 3.5 m thick peat deposit; the surface of the peat was encountered between Elevations 307.4 m and 308.7 m, and was lowest below the central portion of the embankment and higher below the highway shoulders. In Boreholes 07-14, the peat is underlain by about 2.7 m of very loose to loose sand to sand and silt, containing cobbles, and in Borehole 07-15, the peat is underlain by 0.5 m of loose silty sand and 1.5 m of soft to very stiff clayey silt. These boreholes were terminated within these soils due to sampler and auger refusal at depths of 9.4 m and 9.6 m (Elevation 302.4 m to 303.0 m). Borehole 07-13 was terminated at a depth of 6.5 m (Elevation 305.5 m) at the base of the peat, also due to sampler and auger refusal.

Culvert Location	***Borehole Nos.***	***General Subsurface Conditions***
Station 25+057	07-16 07-17 07-18	Boreholes 07-16, 07-17 and 07-18 were drilled through the Highway 62 embankment and encountered between 1.8 m and 3.1 m of very loose to compact sand and gravel, gravelly sand and sand fill. As shown in the interpreted stratigraphic section on Drawing 4, in all three boreholes, the fill is underlain by 1.5 m to 2.0 m of peat, the surface of which was encountered between Elevations 308.9 m and 310.5 m. In Borehole 07-16 under the west embankment shoulder, the peat is underlain by schist bedrock. In Boreholes 07-17 and 07-18, the peat is underlain by a thin (0.2 m to 0.7 m thick) deposit of loose to compact sand and silt or sand and gravel, in turn underlain by bedrock. The surface of the schist bedrock was encountered at Elevations 306.9 m and 307.8 m in Boreholes 07-16 and 07-17, where the bedrock was cored; in Borehole 07-18, auger refusal on possible bedrock occurred at about Elevation 308.8 m.
Station 25+529	07-19 07-20 07-21	Boreholes 07-19, 07-20 and 07-21 were drilled through the Highway 62 embankment and encountered 2 m to 2.1 m of very loose to compact sand and gravel, gravelly sand and sand fill; portions of the fill were observed to contain cobbles. Borehole 07-21 was terminated at a depth of 2.1 m in the fill, upon sampler and auger refusal. As shown in the interpreted stratigraphic section on Drawing 4, Boreholes 07-19 and 07-20 encountered approximately 1.2 m and 0.3 m of peat, respectively, with the surface of the peat encountered at about Elevation 311.4 m in both boreholes. In Borehole 07-19, the peat is underlain by schist bedrock, while in Borehole 07-20 the peat is underlain by about 0.8 m of compact sandy silt which is, in turn, underlain by schist bedrock. The surface of the schist bedrock was encountered at Elevation 310.2 m and 310.3 m in these boreholes, at a depth of 3.2 m below the Highway 62 grade.
Station 17+379	07-1 07-2 07-3	Boreholes 07-1, 07-2 and 07-3 were drilled through the Highway 62 embankment and encountered 1.4 m to 1.8 m of sand to sand and gravel fill. In Boreholes 07-2 and 07-3, as shown in the interpreted stratigraphic section on Drawing 5, the fill is underlain by a 0.4 m to 0.6 m thick layer of peat, the surface of which was encountered at Elevations 345.4 m to 345.6 m. The fill and peat are underlain by a deposit of very loose to compact sand to sand and silt, which is in turn underlain by schist bedrock. The surface of the bedrock was encountered at Elevation 338.9 m (at a depth of 7.9 m below the Highway 62 grade) in Borehole 07-2, and auger refusal on possible bedrock was encountered at Elevations 340.3 m and 339.9 m (at 6.9 m and 7.5 m depth) in Boreholes 07-1 and 07-3, respectively.

Further information regarding the soil deposits and bedrock encountered in the boreholes at the culvert sites is provided in the following sub-sections.

4.2.1 Sand and Gravel to Sand to Crushed Rock Fill

All of the boreholes at the seven culvert sites were advanced through the existing Highway 62 embankment, and encountered between 1.4 m and 7.2 m of fill.

The fill varied in composition from sand and gravel, to gravelly sand, to sand, to silty sand; cobbles were observed in these fill materials in many of the boreholes, and such instances are noted on the borehole records. Portions of the fill in Boreholes 07-4 and 07-6 at the site of the culvert at Station 21+369, and in Borehole 07-15 at the site of the twin culverts at Station 24+320/24+322, have been classified as crushed rock fill containing sand and gravel. The results of grain size distribution tests on seventeen selected samples of the sand and gravel to silty sand portions of the fill are shown on Figures 1A to 1C following the text of this report; due to sampler size limitations, cobbles are not included in these test results.

The Standard Penetration Test (SPT) "N" values measured within the sand and gravel to silty san fill range from 1 blow to greater than 50 blows per 0.3 m of penetration, indicating a variable, very loose to very dense relative density.

4.2.2 Peat

Very soft to firm fibrous or amorphous peat was encountered below the fill in all boreholes except Boreholes 07-1, 07-6 and 07-21; the latter two boreholes were terminated in or at the base of the fill upon sampler and auger refusal. The depth to the peat surface, the peat surface elevation and the peat thickness in each of the boreholes at the culvert sites are summarized in the following table:

Culvert Station	*Borehole Nos.*	*Depth to Peat Surface*	*Peat Surface Elevation*	*Peat Thickness*
21+369	07-4	3.1 m	306.8 m	1.5 m
	07-5	2.7 m	307.0 m	1.9 m
	07-6	-	-	-
23+418	07-7	3.1 m	306.3 m	3.6 m
	07-8	2.7 m	307.2 m	3.2 m
	07-9	7.2 m	305.3 m	1.6 m
24+124/ 24+126	07-10	4.7 m	307.4 m	3.2 m
	07-11	4.3 m	308.0 m	3.0 m
	07-12	5.2 m	307.4 m	4.0 m
24+320/ 24+322	07-13	3.7 m	308.4 m	2.8 m
	07-14	3.4 m	308.7 m	3.5 m
	07-15	5.0 m	307.4 m	2.5 m
25+057	07-16	3.1 m	308.9 m	2.0 m
	07-17	1.8 m	310.4 m	1.9 m
	07-18	2.0 m	310.5 m	1.5 m
25+529	07-19	2.0 m	311.4 m	1.2 m
	07-20	2.1 m	311.4 m	0.3 m
	07-21	-	-	-

Culvert Station	*Borehole Nos.*	*Depth to Peat Surface*	*Peat Surface Elevation*	*Peat Thickness*
17+379	07-1	-	-	-
	07-2	1.4 m	345.4 m	0.4 m
	07-3	1.8 m	345.6 m	0.6 m

Water content tests were carried out on 52 samples of the peat, and were measured between 43 and 766 per cent. Organic content tests, measured on ten samples, ranged from 8 to 89 per cent; the lower organic content measurements were obtained for samples that contained seams or layers of sand and alluvium.

4.2.3 Clayey Silt to Clay

Thin deposits of clayey silt to clay, between 0.5 m and 2.0 m in thickness, were encountered below the peat in five of the boreholes at three of the culvert sites: Boreholes 07-7 and 07-9 (Station 23+418), Boreholes 07-10 and 07-11 (Station 24+124/24+126), and Borehole 07-15 (Station 24+320/24+322). The surface of the clayey silt to clay was encountered between Elevations 302.7 m and 305.0 m in these boreholes.

Atterberg limit testing was carried out on three samples of the clayey silt to clay from these boreholes, and measured the following results:

Borehole and Sample No.	*Plastic Limit*	*Liquid Limit*	*Plasticity Index*
07-7 Sa 12	31 %	137 %	106 %
07-9 Sa 13B	16 %	23 %	7 %
07-15 Sa 11B	18 %	30 %	12 %

These results, which are plotted on a plasticity chart on Figure 2, demonstrate that these thin cohesive deposits vary from clayey silt of low plasticity to clay of high plasticity.

The SPT "N" values measured within the clayey silt to clay generally ranged from 2 to 8 blows per 0.3 m of penetration, indicative of a soft to stiff consistency; a higher SPT "N" value measured in the clayey silt at the base of Borehole 07-15 is attributable to sampler refusal on possible bedrock or a boulder.

4.2.4 Sand and Gravel to Sand and Silt

Cohesionless soil deposits were encountered below the fill (where the peat was absent in Borehole 07-1), and immediately below the peat and the clayey silt to clay (where present) in nearly all of the boreholes, except at the following locations:

- Boreholes 07-6 and 07-21, which were terminated at sampler and auger refusal within or near the base of the fill;

- Borehole 07-13, which was terminated at sampler and auger refusal near the base of the peat; and

- Boreholes 07-16 and 07-19, where the deposit is absent and peat directly overlies the bedrock.

These cohesionless soil deposits vary from about 0.2 m to 6.1 m in total thickness as encountered in the boreholes at the culvert sites; Boreholes 07-1, 07-3, 07-7, 07-8, 07-10, 07-11, 07-12, 07-14 and 07-18 were terminated in or at the base of this deposit on sampler and/or auger refusal to further advance.

The deposits vary in composition from sand containing trace to some silt and trace to some gravel, to silty sand, to sand and silt, to sand and gravel containing trace to some silt. Cobbles were noted within the deposits at some of the borehole locations, and such instances are noted on the borehole records. The results of grain size distribution tests completed on fifteen selected samples of the deposits are shown on Figures 3A and 3B; due to sampler size limitations, cobbles are not included in these test results.

The SPT "N" values measured within the deposits range from 1 to greater than 100 blows per 0.3 m of penetration, indicating a variable, very loose to very dense relative density.

4.2.5 Silty Sand Till

A layer of silty sand till was encountered at the base of Borehole 07-9 (Station 23+418). The surface of the till was encountered at a depth of 13.1 m (Elevation 299.3 m), and the deposit was penetrated for a thickness of 0.7 m; the borehole was terminated within this deposit.

The till consists of a heterogeneous mixture of silty sand containing some gravel and trace clay; the result of a grain size distribution test completed on one sample of this deposit is presented on Figure 4.

One measured SPT "N" value within silty sand till of 180 blows per 0.3 m of penetration indicates that the deposit has a very dense relative density.

4.2.6 Bedrock

Schist and gabbro bedrock were encountered below the peat and soil deposits in several of the boreholes, as evidenced by rock coring. Refusal to sampler and auger advance was observed in many other boreholes, and may represent the bedrock surface; however, refusal to sampler and auger advance could also be attributable to the presence of cobbles and/or boulders within the soils at the refusal depth. The depth to the bedrock surface and bedrock surface elevation as encountered by coring or inferred from refusal are summarized in the following table:

Culvert Station	*Borehole No.*	*Ground Surface Elevation (m)*	*Depth to Bedrock (m)*	*Bedrock/Refusal Elevation (m)*	*Notes*
21+369	07-4	309.8	4.7	305.1	Cored
	07-5	309.7	5.3	304.4	Cored
	07-6	310.2	3.3	306.9	Inferred
23+418	07-7	309.4	10.4	298.9	Inferred
	07-8	309.9	9.3	300.6	Inferred
	07-9	309.9	9.3	298.7	Inferred
24+124/ 24+126	07-10	312.2	9.8	302.4	Inferred
	07-11	312.3	8.5	303.8	Inferred
	07-12	312.6	9.6	303.0	Inferred
24+320/ 24+322	07-13	312.0	6.5	305.5	Inferred
	07-14	312.0	9.6	302.4	Inferred
	07-15	312.4	9.4	303.0	Inferred
25+057	07-16	312.0	5.1	306.9	Cored
	07-17	312.2	4.4	307.8	Cored
	07-18	312.5	3.7	308.8	Inferred
25+529	07-19	313.4	3.2	310.2	Cored
	07-20	313.5	3.2	310.3	Cored
	07-21	313.8	2.1	311.6	Inferred
17+379	07-1	347.1	6.9	340.3	Inferred
	07-2	346.8	7.9	338.9	Cored
	07-3	347.4	7.5	339.9	Inferred

Where bedrock core samples were recovered, the schist and gabbro bedrock encountered at the culvert sites was observed to be fresh and medium strong. The total core recovery in the schist and the gabbro bedrock ranged from 88 to 100 per cent, and the measured Rock Quality Designation (RQD) values ranged from 0 to 100 per cent, but were generally above 50 per cent indicating that the bedrock is typically of fair to good quality. Definitions for the terms used in the description of the bedrock are provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the borehole records included in this report.

4.2.7 Groundwater Conditions

A standpipe piezometer was installed in one borehole at each culvert site, sealed within the granular fill and underlying peat; details of the piezometer installations are shown on the borehole records. The water levels measured in the piezometers are summarized in the following table:

		November 9, 2007		*June 18-22, 2009*	
Culvert Station	***Borehole No.***	***Depth to Groundwater***	***Groundwater Elevation***	***Depth to Groundwater***	***Groundwater Elevation***
21+369	07-5	1.8 m	307.9 m	1.6 m	308.1 m
23+418	07-8	0.3 m	309.6 m	0.3 m	309.6 m
24+124/ 24+126	07-11	0.7 m	311.6 m	0.6 m	311.7 m
24+320/ 24+322	07-13	0.7 m	311.3 m	0.6 m	311.4 m
25+057	07-17	0.8 m	311.4 m	0.6 m	311.6 m
25+529	07-20	1.2 m	312.3 m	1.1 m	312.4 m
17+379	07-2	0.8 m	346.0 m	0.6 m	346.2 m

Based on the piezometer measurements as summarized above, as well as the groundwater conditions observed in the open boreholes during and on completion of drilling (as noted on the borehole records and as depicted on the interpreted stratigraphic sections on Drawings 1 to 5), the groundwater level at each of the culvert sites is typically at or slightly above the natural ground surface in the adjacent swamp, and similar to the water level in the swamp.

It should be expected that the groundwater level at the culvert sites will be subject to seasonal fluctuations, and will be higher during wet periods of the year (i.e. during spring conditions).

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Sen Hu, E.I.T., and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
CULVERT REPLACEMENTS

HIGHWAY 62 FROM 5.3 KM NORTH OF CLEVELAND ROAD
TO 300 M SOUTH OF COUNTY ROAD 620
BANCROFT, ONTARIO
G.W.P. 66-99-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical/foundation design recommendations for the proposed replacement of seven culverts along Highway 62 between 5.3 km north of Cleveland Road and 300 m south of County Road 620, south of Bancroft, Ontario. It is noted that during the course of detail design, it was determined by GENIVAR that the culvert at Station 23+418 did not require replacement; however, the recommendations for this culvert site have been maintained in this section of the report for future consideration, where appropriate.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a subsurface investigation at the culvert sites. The interpretation and recommendations are intended to provide the designers with sufficient information to assess feasible geotechnical/foundation options and to design the proposed culvert replacements.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or constraints may be required in the Contract Documents. 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, scheduling and the like.

6.2 Summary of Existing Conditions

Based on the borehole results, and as shown on the interpreted stratigraphic sections on Drawings 1 to 5, peat is present below the existing Highway 62 embankment fill and below the existing culverts at all seven culvert sites. The water level at the culvert sites as measured in the boreholes and piezometers is relatively high – typically less than 1.5 m below the Highway 62 pavement grade, near or above the surrounding ground surface and similar to the water level in the adjacent swamps.

As a result of the settlement of the peat below the existing highway embankment loadings, the culverts are submerged (Station 21+369, 23+418, and 25+057), partially buried (Station 24+124/24+126) or unable to be found (Station 24+320/24+322). The existing CSP culverts are generally in poor condition.

As presented in Appendix B, static and seismic stability analyses indicate that with the peat in place, the existing embankments at the culvert locations have an acceptable factor of safety against global instability (greater than 1.3 for static global stability, and greater than 1.0 for seismic stability); this is corroborated by visual observations of the embankment stability conditions at these locations.

As discussed in Appendix C, the primary consolidation settlement of the peat below the existing embankments has been completed, and the peat is expected to undergo secondary (or "creep") settlement of approximately 25 mm to 75 mm over the next 75 years. This creep settlement will be variable across and along the Highway 62 embankment at the culvert locations, given the variable thickness and properties of the peat.

These stability and settlement results demonstrate that the existing embankment configurations are stable on top of the peat, and that primary consolidation of the peat has been achieved below the existing embankment loadings. If the Highway 62 grade is raised or the existing embankment widened, the increase in embankment loading would induce additional primary consolidation settlement and additional creep settlement of the embankment and culvert, and could contribute to instability of widened embankment side slopes at these locations. However, it is understood that the current plans are for the existing Highway 62 grade and width to be maintained at the culvert replacement locations.

6.3 Peat Subexcavation Versus Leaving In Place

With respect to the presence of peat below the culverts, there are two main options related to the replacement of the culverts: subexcavate the peat and reconstruct the culverts, or leave the peat in place and install new culverts. Partial subexcavation of the peat could also be considered. The depth of excavation required at each of the culvert sites is summarized in the following table (see column for "depth to base of peat"):

Culvert Station	*Borehole Nos.*	*Depth to Peat Surface*	*Peat Surface Elevation*	*Peat Thickness*	*Depth to Base of Peat*
21+369	07-4	3.0 m	306.8 m	1.5 m	4.5 m
	07-5	2.7 m	307.0 m	1.9 m	4.6 m
	07-6	-	-	-	-
23+418	07-7	3.1 m	306.3 m	3.6 m	6.7 m
	07-8	2.7 m	307.2 m	3.2 m	5.9 m
	07-9	7.2 m	305.3 m	1.6 m	8.8 m
24+124/ 24+126	07-10	4.7 m	307.4 m	3.2 m	7.9 m
	07-11	4.3 m	308.0 m	3.0 m	7.3 m
	07-12	5.2 m	307.4 m	4.0 m	9.2 m
24+320/ 24+322	07-13	3.7 m	308.4 m	2.8 m	6.5 m
	07-14	3.4 m	308.7 m	3.5 m	6.9 m
	07-15	5.0 m	307.4 m	2.5 m	7.5 m
25+057	07-16	3.1 m	308.9 m	2.0 m	5.1 m
	07-17	1.8 m	310.4 m	1.9 m	3.7 m
	07-18	2.0 m	310.5 m	1.5 m	3.5 m
25+529	07-19	2.0 m	311.4 m	1.2 m	3.2 m
	07-20	2.1 m	311.4 m	0.3 m	2.4 m
	07-21	-	-	-	-
17+379	07-1	-	-	-	-
	07-2	1.4 m	345.4 m	0.4 m	1.8 m
	07-3	1.8 m	345.6 m	0.6 m	2.4 m

For the culverts at Stations 23+418, 24+124/24+126 and 24+320/24+322, subexcavation depths of approximately 5.9 m to 9.2 m (below Highway 62 pavement grade) would be required to reach the base of the peat. For the culverts at Stations 21+369 and 25+057, subexcavation depths of about 3.5 m to 5.1 m would be required to reach the base of the peat. For the two northernmost culverts (Stations 25+529 and 17+379), shallower subexcavation depths of about 1.8 m to 3.2 m would be required to reach the base of the peat.

If the peat is fully subexcavated from below the new culverts, there will be negligible risk of settlement of the new culverts and embankments in the immediate vicinity of the culverts. However, for the thick and deep peat deposits that are present for the southern five culvert sites, there would be some risk related to creep instability of open-cut excavations through the peat unless very flat (3H:1V) temporary excavation side slopes are used during the construction works. It would therefore be very difficult to advance excavations to the depth required for peat subexcavation while maintaining one lane of traffic on Highway 62, particularly for the culverts at Stations 23+418, 24+124/24+126, and 24+320/24+322; however, traffic could be maintained by constructing a detour embankment around the subexcavation area, similar to the option discussed in Section 6.6.5.

Given the stability and settlement analysis results as summarized in Section 6.2 (and presented in greater detail in Appendices B and C of this report), the peat could be left in place and new culverts installed at a higher elevation. In this case, there will be some ongoing "creep" settlement over the life of the new culverts and surrounding embankments, and some ongoing road/pavement maintenance would be required. However, deep excavation would not be required, dewatering would be reduced, and construction staging would be easier as part of this option.

Given that the predicted creep settlement of the peat is expected to be on the order of 25 mm to 75 mm over the next 75 years (as presented in Appendix C), it is not considered cost effective to undertake even partial subexcavation of the peat below the culvert sites to attempt to reduce these settlements. Partial removal of the peat would still require excavations to depths of at least 3 m to 5 m, with associated shoring, detour construction and/or traffic control costs as noted above for the full subexcavation option, and could still result in creep settlements of up to 25 mm to 50 mm over the next 75 years.

From a geotechnical/foundations perspective, the preferred option is to leave the peat in place at the southern five culvert sites. This preferred option is predicated on the understanding that there is no grade raise or embankment widening planned for these areas of Highway 62 as part of the proposed highway rehabilitation works. This option is considered to have acceptable risk in terms of embankment stability and primary consolidation settlement (although maintenance will be required to accommodate ongoing creep settlement), and to have a lower cost given that the depth of excavation and associated groundwater control will be reduced as compared with the full or partial peat subexcavation options.

At the sites of the two northernmost culverts at Stations 25+529 and 17+379, where the depth for peat subexcavation would vary from about 1.8 m to 3.2 m below the Highway 62 grade, full or partial peat subexcavation is considered to be more feasible than at the other five culvert sites. However, in this case, consideration must be given to the peat treatment in the immediate vicinity of the culvert relative to the peat treatment throughout the embankment area to the north and south: if the peat is fully or partially subexcavated from below the culvert but not the remainder of the embankment crossing these two swamp areas, these two culvert locations would eventually develop into a "bump" on Highway 62 as the adjacent sections of embankment/pavement undergo creep settlement. The most cost-effective solution for the culverts at Stations 25+529 and 17+379 would be to leave the peat in place below the new culverts and the adjacent embankments (as recommended for the other five culvert sites); because the peat is thinnest at these two culvert sites, the ongoing creep settlement will be less and the future performance of these two culverts and their associated embankment areas is anticipated to be better than for the areas around the other five culvert sites.

6.4 Removal Versus In-Place Abandonment of Existing Culverts

The existing culverts have undergone settlement, along with the existing Highway 62 embankment, since their original construction. The following table summarizes the potential depth and elevation of the top of the existing culverts relative to the Highway 62 pavement grade, based on the estimated magnitude of primary consolidation settlement that has occurred in the peat and clayey silt to clay (where present) below the existing embankments, as summarized in Appendix C.

Culvert Station	*Existing Culvert Diameter*	*Original (Design) Invert Elevation*	*Estimated Magnitude of Settlement*	*Potential Elevation of Top of Culvert**	*Potential Depth to Top of Culvert**
21+369	900 mm	307.4 m (west) 307.2 m (east)	1.1 m to 1.2 m	306.9 m to 307.1 m	3.1 m to 3.3 m
23+418	1,200 mm	308.7 m (west) 308.8 m (east)	1.2 m to 5.0 m	305.0 m to 308.9 m	3.5 m to 7.4 m
24+124/ 24+126	1,800 mm	310.3 m (west) 310.1 m (east)	3.1 m to 4.4 m	307.6 m to 308.9 m	3.7 m to 5.0 m
24+320/ 24+322	1,800 mm	310.5 m (west) 310.9 m (east)	3.4 m to 4.9 m	307.6 m to 309.3 m	3.1 m to 4.8 m
25+057	1,200 mm	309.6 m (west) 310.1 m (east)	1.2 m to 2.1 m	309.0 m to 310.1 m	2.4 m to 3.5 m
25+529	1,200 mm	311.9 m (west) 311.8 m (east)	0.4 m to 1.7 m	311.4 m to 312.6 m	1.2 m to 2.4 m
17+379	900 mm	345.6 m (west) 345.5 m (east)	0.3 m	346.1 m to 346.2 m	1.2 m to 1.3 m

* Relative to Highway 62 pavement.

As a result of the settlement of the peat below the existing embankment, the culverts are submerged (Station 21+369, 23+418, and 25+057), partially buried (Station 24+124/24+126) or unable to be found (Station 24+320/24+322). The existing CSP culverts are generally in poor condition, and there is a moderate risk that this degradation will continue and potentially collapse, resulting in settlement or the formation of sinkholes at the Highway 62 grade, similar to that which occurred in the past at the culvert site at Station 24+320/24+322. Consideration was also given to in-place abandonment of these existing culverts, for example by breaking into the top of the existing culverts and grouting any remaining void space. However, the composition, density and volume of existing infilling material within these CSP culverts is not known, and there is a moderate risk that in-place abandonment would not be fully successful.

Based on discussions with the MTO team during the course of detail design, the potential risk associated with leaving these existing culverts in place was considered unacceptable, except at Station 24+320/24+322 where the existing culvert was previously abandoned in place by grouting, and is no longer able to be found.

Based on the “shallower” potential depth to the obvert and invert for the culverts at Stations 21+369, 25+057, 25+529 and 17+379, it is considered that these existing culverts could be removed and the excavation be backfilled as part of the conventional culvert replacement works. For the culverts at Stations 23+418 and 24+124/24+126, the excavation depth for removal is generally expected to be greater than 5 m, and could be more than 7 m in depth relative to the Highway 62 pavement grade. While excavation to this depth is feasible, temporary excavation support systems or open-cut excavations with a road closure and/or temporary detour will be required (as discussed in Section 6.6).

Following removal of the existing culverts, the new culverts could be installed on the existing culvert alignment, or could be offset by a distance of approximately 3 m from the existing alignment. It is anticipated that the culvert removal will be carried out without full dewatering and, as discussed further in Sections 6.8 and 6.9.3, OPSS 1010 Granular B Type II is recommended as backfill following removal of the existing culverts in this case. The Granular B Type II material will perform at least as well as the existing loose silty sand fill that comprises the Highway 62 embankment, and so the subgrade conditions below the culvert bedding level should be similar or better at the existing culvert location as compared with a location approximately 3 m away. Further, the placement of the new culvert on the existing alignment will most closely result in a “zero net load increase” at the original culvert location, and so will cause less differential settlement longitudinally along Highway 62.

6.5 Replacement Culvert Type

From a geotechnical/foundations perspective, assuming that the peat is left in place below the culvert sites, new pipe culverts are preferred over the use of concrete box culverts or open footing culverts, as flexible pipe culverts will be more tolerant of the ongoing creep settlement than rigid concrete box or open footing culverts. A high density polyethylene (HDPE) pipe is considered to be a better option than a corrugated steel pipe, given the acidic nature of the soils and water in these swamp crossing areas. As an alternative, consideration could also be given to using a concrete pipe culvert, particularly if pipe ramming is adopted for any of the culvert replacement locations.

It is also preferred, from a geotechnical perspective, that the invert elevation be maintained as high as possible to minimize excavation and dewatering requirements. Based on discussions with GENIVAR, it is understood that higher invert elevations (as compared with the original design invert elevations for the existing culverts) are feasible from a drainage perspective.

6.6 Culvert Removal and Replacement Staging Options

Assuming that the peat remains in place below the Highway 62 embankments and culverts at these seven culvert sites, the following staging options could be considered for the culvert abandonment and replacement operations:

1. Vertical-sided, temporarily shored excavation with staged abandonment and replacement of each culvert (i.e., east half, then west half), while maintaining one lane of traffic on existing Highway 62 embankment.

2. Open-cut excavation with staged abandonment and replacement of each culvert (i.e., east half, then west half), while maintaining one lane of traffic on existing Highway 62 embankment.

3. Trenchless installation of new culverts to allow traffic to be maintained on existing Highway 62 embankment.

4. Temporary or permanent widening of the existing Highway 62 embankment adjacent to the culvert replacement areas, to allow maintenance of traffic with more room for culvert abandonment and replacement works (via temporary shoring or open-cut excavation methods).

5. Temporary "stand-alone" detour to the east or west of the existing Highway 62 embankment, to allow maintenance of traffic and to eliminate requirement for staging each culvert abandonment and replacement in two halves (via temporary shoring or open-cut excavation methods).

The feasibility, advantages, disadvantages and risks associated with these replacement options are presented in the following subsections and are summarized in Table 1 following the text of this report.

6.6.1 Temporary Excavation Support

This option would consist of removing each existing culvert and installing each new culvert in two stages (i.e., east half, then west half) within shored, vertical-sided excavations. With temporary protection systems in place, the culvert replacement works will occupy a smaller footprint and make it easier to maintain traffic on Highway 62 than for an open-cut option (presented in Section 6.2.3.2), with the potential for a wider lane and greater safety for the workers and the travelling public. However, the temporary protection system would have to accommodate the presence of the existing culvert as part of any removal operations. Further, installation of the culvert in two halves would require that a connection be made between the two halves in the centre of the highway.

As discussed further in Section 6.9 (Construction Considerations), the protection systems at the culvert sites should be designed and constructed in accordance with MTO Special Provision SP105S19, using Performance Level 2 as defined in this Special Provision.

It is anticipated that temporary shoring for the culvert sites would have to consist of driven, closed steel sheet pile systems. A cantilevered, heavy sheet pile system should be feasible for shallower excavations, while for deeper excavations the system would likely have to be supported internally by braces/rakers. Soil or rock anchors would be more difficult to use than internal bracing, as at most sites the anchors would have to be quite long to extend below the peat to native soils or bedrock. There is some risk that the sheet piles will encounter cobbles and/or boulders during installation through the existing embankment fill and native soils at the site; however, with a heavy sheet pile section, it is considered that the risk of being unable to drive the sheet piles to the design depth will be relatively low.

This type of interlocking system driven well below the bottom of the excavation would contribute to groundwater control in the excavation, with reduced groundwater control costs as compared to an open-cut excavation option. However, active dewatering (likely pumping from sumps within the sheet-piled excavation) would still be required.

6.6.2 Open-Cut Excavation

This option would consist of removing the existing culverts and installing the new culverts within open-cut excavations. For this option, dewatering will be critical to cutting and maintaining the excavation side slopes without sloughing or caving. Given that the fill below the water level is typically composed of fine sand to silty sand, it is anticipated that it will be very difficult to fully dewater the existing fill, and therefore it is unlikely that temporary side slopes of 1H:1V will be achievable. Practically speaking, for excavation depths of 2 m to 3 m for installation of new culverts or access to the top of existing culverts for abandonment, it is expected that overall side slopes on the order of 2H:1V will be achievable to account for the fact that dewatering of the existing fill is not likely to be fully effective.

With these temporary excavation side slope configurations, depending on the depth of excavation, it may not be possible to maintain one lane of traffic on the existing Highway 62 embankment at some culvert sites, even with culvert removal and replacement in two halves. It may be necessary to temporarily widen the Highway 62 embankment or create a stand-alone detour around the culvert replacement sites to maintain a lane of traffic. If widening is adopted (as discussed further in Section 6.6.4), there is a high risk of additional settlement below the existing and widened portions of the embankment, as well as a risk of instability of the widened embankment side slope; therefore, embankment widening is not recommended as a means to maintain traffic on Highway 62 during the culvert replacement works.

6.6.3 Trenchless Installation

In this option, the new culverts could be installed using pipe ramming methods. A dewatered driven steel sheet pile (cofferdam) system would be required on the east and west sides of the Highway 62 embankment for the entry and exit pits at those crossings where sufficient room is available; however, excavation through the Highway 62 roadway fill itself would not be required as part of the new culvert installation, and so disruption to traffic would be minimized as compared with the two excavation options presented above. However, the trenchless method addresses only installation of new sewer pipes and not removal of the existing pipes. Excavation to remove or abandon the existing culverts would still be required, effectively negating the advantages of trenchless installation. Further, the protection systems/coffer dams at the culvert ends would be installed in thicker deposits of peat, rather than into granular fill as would be the case for shoring at the highway centreline, likely resulting in greater sheetpile lengths and horizontal restraint requirements, increasing the cost of the protection systems/coffer dams.

Depending on the invert depth and diameter for the proposed replacement sewers, the thickness of cover over top of the sewer may be less than is normally desired for trenchless installation below MTO highways (i.e., a minimum cover thickness of 1.5 m or two tunnel diameters, whichever is greater). It is considered that pipe ramming could be carried out through the water-bearing cohesionless fill soils with a low risk of ground loss and settlement, although there is some risk of encountering obstructions (boulders or a "nest" of cobbles) within the embankment fill that could impact this installation; there is also a risk of "heave" of the ground over the sewer pipe during installation, particularly if the thickness of the cover material is less than 1.5 m or two tunnel diameters. It may be possible to address potential settlement or heave over the pipe ramming installation by temporarily closing the lane under which the pipe ramming is actively occurring and monitoring for settlement/heave; such settlement or heave could then be corrected before the pavement rehabilitation work is completed.

6.6.4 Widening of Existing Highway 62 Embankment

To maintain traffic during excavation works for the culvert removal and replacement, consideration has been given to widening the existing Highway 62 embankment by approximately 3.5 m. The placement of additional fill on the existing Highway 62 embankment side slope will induce additional consolidation in the underlying peat, resulting in 0.5 m or more of settlement of the existing Highway 62 shoulder and 0.3 m or more of settlement of the outside portion of the nearest lane. The majority of this settlement will take place over a one- to two-month period, and ongoing maintenance of the roadway at the culvert sites would be required during this period.

This option may be feasible if traffic could be restricted to one lane on the opposite side of the existing Highway 62 embankment during and immediately following placement of fill for a widening, to allow the settlements to "stabilize" before permitting traffic on the closest lane. However, consolidation settlement in the peat and the clayey silt to clay deposits (where present) will continue beyond this period, and could impact the performance of the newly installed culvert as well as the future performance of the Highway 62 embankment.

In addition, the widening of the existing embankment may still not permit sufficient room for excavation at those sites where the existing culverts are at depths of greater than 5 m, and culvert removal and replacement would still have to be staged in two halves.

6.6.5 Stand-Alone Detour

GENIVAR has examined the available space within the MTO right-of-way for a "stand-alone" detour embankment, and has proposed that a 5.5 m wide platform, about 100 m in length, could be constructed approximately 21.5 m (centreline-to-centreline) to the east of the existing Highway 62 embankment, to maintain traffic adjacent to Highway 62 during culvert abandonment and replacement works. This option has the advantage that it would permit the abandonment and replacement works to be completed in a single stage (i.e. not in two halves), and would allow for excavation to a greater depth (such as for removal of the existing culverts) than if traffic is to be maintained on the existing Highway 62 embankment.

It is anticipated that stand-alone detour embankments would be constructed on top of the existing peat deposit, without any subexcavation. As discussed for the permanent Highway 62 embankment, full subexcavation of the peat could be carried out at each of the detour sites and would improve the performance of the detour embankments constructed at the sites. However, from a cost perspective, full subexcavation of the existing peat is not recommended below the detour embankments, based on the following:

- The peat thicknesses are relatively large and significant effort and cost would be required for full subexcavation.

- Full subexcavation could impact the stability of the existing Highway 62 embankment.

- Each detour embankment would be required only for a short period of time (understood to be less than one week of active construction for each culvert abandonment and replacement), and therefore the longer-term settlement performance is not relevant, beyond the initial "stabilization" of settlement that would be required to open the detour to traffic.

6.6.5.1 Detour Embankment Stability

Global slope stability analyses were completed for the two detour options at a "worst case" location at Station 24+124/24+126.

In order to achieve a factor of safety of at least 1.3 (the minimum required) against global instability of the detour embankment side slopes, it would be necessary to incorporate a geotextile mat below a heavy geogrid placed on the peat at the base of the embankment fill as well as within the embankment fill, construct the embankment with side slopes no steeper than 2H:1V, and limit the embankment height to a maximum of 1.3 m above the original ground surface in the swamp. Where water depths of greater than about 0.3 m existing along the detour alignment, the grade could be raised to the water level by rock fill, and then topped with Granular B Type II fill prior to placement of the geotextile mat.

Rock fill could also be used for the construction of the temporary detour embankments. In this case, in order to achieve a minimum factor of safety of 1.3 against global instability of the detour embankment side slopes, it would be necessary to construct the embankment with side slopes no steeper than 1.5H:1V, and limit the embankment height to a maximum of 1.3 m above the original ground surface in the swamp. As for the granular fill option above, it would also be necessary to incorporate a geotextile mat below a heavy geogrid placed on the peat at the base of the embankment fill as well as within the embankment fill. To prevent the rock fill from "punching through" the geogrid and geotextile mat, a layer of Granular B Type II fill is recommended immediately below and above the geotextile/geogrid layers.

Higher embankment heights could be achieved if required; however, it would be necessary to stage the grade raises over a longer period of time in order to achieve the target factor of safety of 1.3.

6.6.5.2 Settlement of Detour Embankments

Significant settlement of the existing peat will occur below the new fill that is placed for a stand-alone detour embankment. The following table summarizes the estimated duration required to complete the primary consolidation settlement of the peat and the predicted magnitude of settlement under the detour embankment loading at the seven culvert sites:

Culvert Location	***Time Required to "Stabilize" Fill***	***Predicted Magnitude of Settlement of Peat****
Station 21+369	2 months	1.5 m
Station 23+418	3 to 12 months	2.5 m
Station 24+124/24+126	3 to 4 months	2.5 m
Station 24+320/24+322	3 to 4 months	2.0 m
Station 25+057	2 to 3 months	1.9 m
Station 25+529	1 month	1.9 m
Station 17+379	1 month	0.8 m

NOTE: Predicted magnitude of settlement of peat given for end of two-month period following initial placement of fill, to allow settlements to "stabilize" and open the detour embankment to traffic.

The durations given in the above table represent the time to complete the majority of the primary consolidation settlement in the peat. For practical purposes related to the construction and opening of the temporary detour, it is anticipated that sufficient settlement of the peat would occur within two months following initial placement of the fill for the detour embankment; within this time period, it is anticipated that between about 1 m and 2.5 m of settlement would occur within the peat, and the width of the detour embankment platform would have to be constructed to accommodate this settlement. If the detour embankment is left in place longer before being opened to traffic (as may occur over a winter shut-down period), a greater proportion of the primary consolidation will have occurred in the peat; essentially, the detour embankment platform would have to be constructed wider for a longer preloading period.

Recommendations for embankment platform overbuilding have been developed in accordance with MTO's Directive NRE 98-200 to accommodate the anticipated embankment settlement. Assuming the use of granular fill with 2H:1V side slopes, the following table provides a summary of the required width of overbuilding on each side of the detour embankment, for two-month and six-month preloading durations. The platform overbuilding width should be re-assessed if the preloading period varies significantly from those given in the table (in particular, for the Culvert 2 site where it is estimated that a longer time period will be required to complete the primary consolidation settlement in the peat).

Detour Embankment Constructed of Granular Fill (2H:1V Side Slopes)				
	Two-Month Preloading Period		Six-Month Preloading Period	
Culvert Location	Estimated Magnitude of Settlement of Peat*	Platform Overbuilding Width (Per Side)	Estimated Magnitude of Settlement of Peat*	Platform Overbuilding Width (Per Side)
21+369	1.5 m	3.0 m	1.8 m	3.6 m
23+418	2.5 m	5.0 m	3.5 m	7.0 m
24+124/ 24+126	2.5 m	5.0 m	3.1 m	6.2 m
24+320/ 24+322	2.0 m	5.5 m	3.1 m	6.2 m
25+057	1.9 m	3.8 m	1.9 m	3.8 m
25+529	1.9 m	3.8 m	1.9 m	3.8 m
17+379	0.8 m	1.6 m	0.8 m	1.6 m

Assuming the use of rock fill with 1.5H:1V side slopes, the following table provides a summary of the required width of overbuilding on each side of the detour embankment, for two-month and six-month preloading durations. The platform overbuilding width should be re-assessed if the preloading period varies significantly from those given in the table (in particular, for the Culvert 2 site where it is estimated that a longer time period will be required to complete the primary consolidation settlement in the peat).

Detour Embankment Constructed of Rock Fill (1.5H:1V Side Slopes)				
	Two-Month Preloading Period		Six-Month Preloading Period	
Culvert Station	Estimated Magnitude of Settlement of Peat*	Platform Overbuilding Width (Per Side)	Estimated Magnitude of Settlement of Peat*	Platform Overbuilding Width (Per Side)
21+369	1.3 m	2.0 m	1.6 m	2.4 m
23+418	2.2 m	3.3 m	3.0 m	4.5 m
24+124/ 24+126	2.2 m	3.3 m	2.7 m	4.1 m
24+320/ 24+322	1.7 m	2.6 m	2.7 m	4.1 m
25+057	1.7 m	2.6 m	1.6 m	2.4 m
25+529	1.7 m	2.6 m	1.6 m	2.4 m
17+379	0.7 m	1.1 m	0.7 m	1.1 m

The predicted magnitude of settlement below the detour embankments would also have to be considered with respect to the placement of any temporary culverts that may be required below the detour embankments.

If detour embankments are adopted for this project, it is recommended that settlement monitoring be carried out to monitor the magnitude and rate of settlement following initial placement of the detour embankment fill prior to and following opening the detour embankments to traffic. This aspect is discussed further in Section 6.9 (Construction Considerations).

6.6.5.3 Conceptual Sequence for Detour Embankment Construction

The following conceptual sequence is recommended for detour embankment construction around the culvert sites, based on the use of coarse, 6-inch minus (OPSS 1010 Granular B Type II) fill.

- Place coarse granular fill or rock fill topped by granular fill on top of the peat, to raise the grade above the water level in the swamp and permit initial placement of a base layer of geotextile and geogrid at or above the water level. The base layer of fill should be placed over the full width of the detour embankment, including platform widening to accommodate the predicted magnitude of settlement of the peat, as discussed above.

- Place additional Granular B Type II fill for the embankment construction above the bottom layer of geogrid and geotextile, incorporating an additional layer of geogrid and geotextile.

- Construct the detour embankment with side slopes at 2H:1V to a maximum height of 1.3 m above the original ground surface in the swamp.

- Place additional fill throughout the peat consolidation period to accommodate the predicted 1 m to 2.5 m of settlement of the peat and maintain the vertical profile of the detour.

The above recommendations are conceptual only, and are similar to the approach that was successfully used for the temporary construction access road at the Highway 417 bridge over the Mississippi River near Arnprior, Ontario. Detailed design for the geotextile and geogrid layers and transition treatments will be required for the soil conditions at the culvert sites if stand-alone detour embankments are selected for any of the culvert replacement sites.

6.6.6 Preferred Option from Geotechnical/Foundations Perspective

Based on the advantages, disadvantages and risks presented in the preceding sections and in Table 1, shored excavations represent the lowest risk in terms of excavation stability and impact to traffic on the existing Highway 62 embankment, as compared to open-cut excavations. Trenchless installation methods, while feasible and low-risk in terms of minimizing impact to traffic operations, would not address the removal or abandonment of the existing culverts.

Widening of the existing Highway 62 embankment to maintain traffic during excavation work at the culvert sites would involve additional loading on the subsoil that would induce significant settlement under the existing highway shoulder and outside portion of the lane, and require maintenance throughout the construction period. Therefore, the widening option is not recommended to facilitate the culvert replacements.

The construction of a stand-alone detour embankment is considered to be a geotechnically-feasible and viable option where deeper excavation is required (such as for the removal of the deeper culverts at Stations 23+418, 24+124/24+126 and 24+320/24+322), as this would allow traffic to be maintained while permitting the deeper shored or open-cut excavations through the existing Highway 62 embankment. However, the cost to construct stand-alone detour embankments has been assessed by GENIVAR to be higher than the cost for vertical-sided, shored excavations.

Therefore, from a geotechnical/foundations perspective, shored excavations are preferred for the culvert replacement works.

6.7 Culvert Foundation Recommendations

The following table summarizes the proposed diameter and proposed invert elevations for the replacement culverts, based on information provided by GENIVAR, along with the required depth of excavation, excavation depth below the groundwater level, and estimated secondary creep settlement (as described in Appendix C) for the 75-year design life of an HDPE pipe culvert.

Culvert Station	*Proposed Diameter*	*Upstream Invert Elevation*	*Downstream Invert Elevation*	*Depth of Excavation*	*Estimated Depth Below Groundwater Level*	*Estimated Creep Settlement (Centreline)*	*Estimated Creep Settlement (Toe)*
21+369	1.0 m	307.2 m	307.2 m	3.0 m	0.9 m	50 mm	30 mm
23+418	This existing culvert is not being replaced as part of this assignment					70 mm	50-65 mm
24+124/24+126	2 – 1.0 m	310.9 m	310.7 m	1.8 m	0.8 – 1.0 m	75 mm	60-70 mm
24+320/24+322	2 – 1.0 m	310.9 m	310.7 m	1.7 m	0.5 – 0.7 m	80 mm	60-70 mm
25+057	1.4 m	310.1 m	309.6 m	2.7 m	1.5 – 2.0 m	50 mm	30-40 mm
25+529	1.4 m	311.7 m	311.6 m	2.2 m	0.7 – 0.8 m	40 mm	10-30 mm
17+379	1.0 m	345.3 m	345.3 m	2.1 m	0.9 m	5 mm	5 mm

As summarized above, excavations to the bedding grade for all of the culvert replacements are expected to extend to between 0.5 m and 2.0 m below the groundwater level at the sites. As discussed further in Section 6.9, groundwater control will be required to facilitate placement of the bedding and the new pipe culverts. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address groundwater control for the culvert sites; a sample NSSP is included in Appendix D to this report.

The settlement estimates presented in the table above assume that the peat is left in place below the embankment at the culvert locations, and that no widening or grade raise is carried out in the vicinity of the culvert sites. As described in Appendix C and summarized in the table above, it is anticipated that the culvert replacements and embankments will undergo about 5 mm to 80 mm of secondary creep settlement over the 75-year design life for an HDPE culvert, and the culvert should be sized and/or cambered to accommodate this magnitude and duration of settlement.

These recommendations should be re-checked once the culvert locations, diameters and invert elevations are established relative to the existing culvert locations, to assess any net load changes due to removal of the existing culvert and placement of backfill (as per Section 6.8), plus installation of the new culvert.

6.8 Culvert Bedding, Backfill and Erosion Protection

6.8.1 Backfill Following Removal of Existing Culverts

Backfill following removal of the existing culverts should consist of OPSS 1010 Granular B Type II, placed up to the underside of the bedding for the replacement culvert. This recommendation is made assuming that the removal of the existing culverts will be completed without full dewatering, and the backfill will be placed in wet conditions or subaqueously, as Granular B Type II material does not undergo significant segregation during placement, and will offer improved settlement performance over other types of granular or earth fill material placed under subaqueous conditions.

6.8.2 Replacement Culverts

The bedding for the replacement culverts should be installed as per Ontario Provincial Standard Drawing (OPSD) 802.010, assuming the use of flexible pipe culverts. The bedding should consist of a minimum of 150 mm of Ontario Provincial Standard Specifications (OPSS) 1010 Granular A, and should conform to the pipe manufacturer's requirements.

Trench backfill above the pipe cover material may consist of approved excavated material (such as the existing pavement granular), imported select subgrade material (SSM) or OPSS 1010 Granular A or Granular B Type II. Pipe culvert frost treatment should be according to OPSD 803.030 and OPSD 803.031, with culverts ideally provided with a minimum of 1.8 m of earth cover or equivalent for frost protection purposes. For frost heave compatibility, it is recommended that the trench backfill within the 1.8 m deep frost zone match the soils exposed in the trench or excavation walls.

To prevent surface water from flowing either beneath the culverts (potentially causing undermining and scouring) or around the culverts (creating seepage through the embankment fill and potentially causing erosion and loss of fines), a clay seal is recommended to be provided at the upstream or inlet side of the culverts. The clay seal should have a minimum thickness of 0.3 m. It should be keyed into the natural subsoil and extend to a minimum horizontal distance of 2.0 m on either side of the culvert inlet opening and extend vertically to the high water level. The material for the clay seal shall be as per OPSS 1205. As an alternative to a clay seal, a concrete apron could be installed around the culvert inlet to serve the same purpose.

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert replacements should be assessed by the hydraulic design engineer. Rip-rap treatment for the culvert outlets should be consistent with the standard presented in OPSD 810.010 Rip-Rap Treatment Type A. Erosion protection for the culvert inlets should follow the standard presented in OPSD 810.010, similar to Rip-Rap Treatment Type A with the rip-rap placed up to the toe of slope level. Similarly, rip-rap should be provided over the full extent of the clay seal or clay blanket, including the embankment fill slope adjacent to the culverts.

6.9 Construction Considerations

6.9.1 Surface Water and Groundwater Control

Control of the surface water and groundwater will be necessary at the culvert replacement sites to allow for the pipe culvert replacements to be installed in dry conditions. The culvert removal operations may be carried out in dry conditions, or may alternatively be carried out without full dewatering (i.e., in wet conditions or subaqueously) provided that appropriate backfill material (OPSS 1010 Granular B Type II) is used as discussed in Sections 6.8.1 and 6.9.3.

Depending on the water conditions in the swamp areas at the time of construction, the surface water flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam. Surface water should be directed away from the excavation areas.

As discussed in Section 6.6, groundwater control will be required for each of the culvert replacement operations. A sample NSSP to address this aspect is provided in Appendix D, for inclusion in the Contract Documents. The NSSP requires appropriate dewatering of the water-bearing granular fill to draw the water level down to below the bedding level for the replacement pipe culverts.

A wellpoint or eductor system, designed and installed by a specialist dewatering contractor, is expected to be necessary and appropriate for dewatering of the granular fill at these sites. If an interlocking sheet pile system is adopted for temporary excavation support, this system would also control groundwater seepage through the excavation side walls; however, sheet pile systems would still have to be supplemented with pumping from sumps or wellpoints located within the excavation to draw the groundwater level down to below the excavation base.

6.9.2 Excavations and Temporary Protection Systems

Temporary excavations for the culvert replacements will be made through the existing Highway 62 embankment fill and peat. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act and Regulations for Construction

Projects. The existing embankment fill would be classified as Type 3 soil, according to the OHSA, assuming that proper groundwater control is in place to dewater cohesionless soil deposits prior to excavation, and the peat would be classified as Type 4 soil. Where space permits, temporary open-cut excavations should be made with side slopes formed no steeper than 2H:1V through the granular fill; side slopes oriented at 3H:1V are expected to be required in the peat.

Depending on which option is adopted for staging of the culvert replacements, temporary roadway protection may be required. The temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19, provided that any adjacent utilities can tolerate this magnitude of deformation. Otherwise, protection/support or relocation of adjacent underground utilities will be required.

Where temporary excavation support is adopted for the culvert replacements, it is considered that a driven, interlocking sheetpile system would be required. A cantilevered, heavy sheet pile system should be feasible for shallower excavations, while for deeper excavations the system would likely have to be supported internally by braces/rakers. Soil or rock anchors would be more difficult to use than internal bracing, as at most sites the anchors would have to be quite long to extend below the peat to native soils or bedrock. There is some risk that the sheet piles will encounter cobbles and/or boulders during installation through the existing embankment fill and native soils at the site; however, with a heavy sheet pile section, it is considered that the risk of being unable to drive the sheet piles to the design depth will be relatively low.

For the culvert site at Station 24+320/24+322, the original 1.8 m diameter CSP culvert was abandoned in place as part of a previous contract by pumping grout into the culvert. This culvert is planned to remain in place (i.e., it is not planned to remove this culvert as part of the current contract). The presence of this existing culvert must be considered in the design and installation of the protection system(s) for the new culvert installation at this site. A sample NSSP to address the presence of this culvert is provided in Appendix D, for inclusion in the Contract Documents.

6.9.3 Subaqueous Backfilling Following Removal of Pipe Culverts

Where removal of existing pipe culverts is completed without full dewatering, such that the groundwater level in the excavation for the removal has not been lowered to below the base of the excavation, then the excavation shall be backfilled subaqueously using OPSS 1010 Granular B Type II fill, up to the underside of the bedding for the replacement culvert. This type of fill material does not undergo significant segregation during such placement, and will offer improved settlement performance over other types of granular or earth fill material placed under subaqueous conditions.

A sample NSSP to address this item has been provided in Appendix D, for inclusion in the Contract Documents.

6.9.4 Final Paving Over Culvert Removal and Replacement Locations

It is recommended that final paving over the culvert removal and replacement sites be scheduled as late as possible following the culvert abandonment and replacement works, to allow for completion of the majority of any primary consolidation settlement in the peat associated with minor loading changes, and settlement of the trench/excavation backfill. A two- to three-month period between the culvert replacement works and placement of the final asphalt lift is recommended.

6.9.5 Settlement Monitoring for Detour Embankments

If detour embankments are adopted for this project, it is recommended that settlement monitoring be carried out to monitor the magnitude and rate of settlement following initial placement of the detour embankment fill prior to and following opening the detour embankments to traffic. The monitoring program should consist of installation of a series of settlement plates within the embankment widening areas, which would be surveyed at regular intervals for the duration of the preloading period.

If detour embankments are adopted, Golder will develop an NSSP, monitoring instrument location plans and typical instrumentation details for the relevant detour sites, along with recommendations regarding monitoring frequency for use in the Contract Administrator assignment.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Sen Hu, E.I.T., and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent quality control review of the report.

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COMPARISON OF CULVERT REPLACEMENT/STAGING OPTIONS
HIGHWAY 62 REHABILITATION
G.W.P. 66-99-00

Replacement Option	Feasibility	Advantages	Disadvantages	Risks/Consequences
Temporary shoring to install new culvert in two (staged) halves	• Temporary shoring could consist of driven sheet piles, in either a cantilever configuration or with internal bracing / rakers • Soil/rock anchors would be more difficult than internal bracing, as the anchors would have to be quite long to extend below peat to native soils/bedrock	• Driven sheet piles could form part of the groundwater control scheme (acting as a cut-off, though some pumping from sumps would still be required) • Easier to maintain traffic on Highway 62 than for open-cut option, with the potential for wider lane and greater safety for construction workers and vehicle traffic than for open-cut option • Driven sheet piles could be removed and re-used at other culvert replacement areas	• Possibility of encountering obstructions in the fill, or cobbles and boulders in the native soils, during driving of sheet piles • Dewatering still required for sheeted excavation, though not as difficult/critical as for open-cut option • Connection between two halves of culvert could be difficult	• Low risk of encountering obstructions (boulders or nests of cobbles) during driving of sheet piles • Low risk of excavation instability provided that temporary shoring system is designed and installed in accordance with MTO SP105S19
Open-cut excavation to install new culvert in two (staged) halves	• Not feasible to achieve 1H:1V excavation side slopes as dewatering in the fine cohesionless fill soils expected to be difficult; flatter overall side slopes of 2H:1V expected be achievable in the fill, and 3H:1V in the peat	• Potentially less expensive than temporary shoring or trenchless installation options; however, must factor in higher costs associated with dewatering and traffic staging	• Dewatering of the relatively fine sandy fill expected to be difficult (would require a vacuum well-point system), and would be critical to achieve a 1H:1V excavation side slope; based on anticipated dewatering difficulties, expect flatter excavation side slopes (2H:1V to 3H:1V) would be required • For deeper excavations (i.e., for removal of existing culverts), may not be possible to carry out open-cut excavation while maintaining traffic on Highway 62, so embankment widening or temporary detour likely required • Connection between two halves of culvert could be difficult	• Risk of inadequate dewatering of fine cohesionless fill soils, leading to sloughing/caving of temporary excavation side slopes that could impact worker safety or traffic staging • If temporary widening of the existing embankment is required to maintain traffic on the highway shoulder, moderate to high risk of settlement of the existing embankment and culvert
Trenchless techniques (pipe ramming) to install new culvert	• Steel liner or concrete sewer pipe could be installed using pipe ramming, which is feasible in the water-bearing cohesionless soils at this site • Shored ("cofferdam") areas would be required on the west and east sides of the highway embankment for the entry and exit pits	• Lowest impact to traffic, although short-term/temporary lane closure may be required to monitor for heave over active area of pipe ramming (see "Risks/ Consequences") depending on the thickness of cover over the top of the pipe	• More expensive option, given construction of "cofferdam" areas for entry and exit pits and mobilization of pipe ramming contractor and equipment to site; however, some offset by lower traffic staging costs • Some dewatering of the entry and exit pit areas would still be required • Trenchless techniques do not address removal of existing culverts	• Low risk of loss of ground/ settlement over culvert using pipe ramming techniques • Low to moderate risk of heave over sewer installation if there is inadequate cover (depending on culvert invert level and pipe diameter); however, lane under which active pipe ramming is taking place could be temporarily closed to monitor for heave, with heave area treated temporarily before pavement rehabilitation work is completed

COMPARISON OF CULVERT REPLACEMENT/STAGING OPTIONS (Continued)
HIGHWAY 62 REHABILITATION
G.W.P. 66-99-00

Staging Option	Feasibility	Advantages	Disadvantages	Risks/Consequences
Temporary widening of Highway 62 embankment to maintain traffic during culvert abandonment and replacement	• Feasible with the use of heavy geogrid and geotextile over peat to enhance stability of the widened portion of the embankment over the peat; however, high potential for significant settlement of existing Highway 62 shoulder and lane • In addition, this widening option would still require staged removal and replacement of the culverts (i.e. in two halves)	• Allows maintenance of traffic on Highway 62 with more room for temporary shoring option (or possibly open-cut option) for culvert replacement • With use of geogrid, adequate factor of safety can be achieved against slope instability; however, lower embankment height or staged grade raises required to maintain an adequate factor of safety throughout construction and operation	• Placement of new embankment fill for temporary detour will induce about 1 m to 2 m of primary consolidation settlement in the underlying peat, resulting in significant settlement of the existing embankment and detour widening; a “preloading” period will be required, with placement of additional fill as necessary to maintain the detour profile • Staged abandonment and replacement of existing culverts would still be required, as it would not be possible to access the portion of the existing/new culvert that is adjacent to/below the detour widening	• High risk of instability of detour widening without use of geogrid below embankment widening • Low risk of instability of detour widening provided that geogrid and geotextile are incorporated into widening • High potential for 0.5 m or more of settlement of existing highway shoulder and outside portion of lane due to placement of fill for detour widening, which is likely to impact traffic operations • Sufficient time (minimum two months, depending on culvert site) required to “stabilize” settlements to permit traffic onto detour widening
Temporary detour embankment constructed about 21.5 m east (centreline to centreline) of existing Highway 62 embankment to maintain traffic during culvert abandonment and replacement	• Feasible with the use of heavy geogrid and geotextile on surface of peat to enhance stability of the new detour embankment	• Allows maintenance of traffic around the culvert sites during construction, and permits abandonment and replacement of culverts without staging of two halves • With use of geogrid, adequate factor of safety can be achieved against slope instability for detour embankment; however, lower embankment height required for the detour to maintain an adequate factor of safety • Distance between temporary detour and existing embankment will minimize settlement of existing highway; for the thickest peat deposits, it is anticipated that up to about 0.4 m of settlement will occur at the existing shoulder at the transition to the detour, reducing to less than 0.1 m of settlement of the existing shoulder in the “central” portion of the detour	• Soil conditions along detour alignment 20 m east of Highway 62 not known in detail; if required, boreholes for detour embankment would require drilling from raft • Placement of new embankment fill for the temporary detour embankment will induce about 1 m to 2.5 m of primary consolidation settlement in the peat below the detour embankment, requiring placement of additional fill to maintain profile grade • Sufficient time required to allow settlement to “stabilize”; primary consolidation settlement anticipated to be complete for the purposes of opening the detour within one to two months, depending on culvert site	• High risk of instability of detour embankment without use of geogrid below new embankment • Low risk of instability of detour embankment provided that geogrid and geotextile are incorporated into the embankment construction • Sufficient time (one to two months, depending on the culvert site) required to “stabilize” settlement to permit traffic onto the detour embankment • Smaller settlement of existing highway shoulder for stand-alone detour as compared with temporary widening option will result in less impact to traffic operations

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.) 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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 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.

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LIST OF SYMBOLS

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

I. General

π	3.1416
in 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 (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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

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LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
‖ - Parallel To	
⊥ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

RECORD OF BOREHOLE No 07-01

 METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 | LOCATION N 4971902.1; E 209206.4 | ORIGINATED BY P.A.H.

DIST Eastern HWY 62 | BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger | COMPILED BY S.L.

DATUM Geodetic | DATE October 18, 2007 | CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
347.1	GROUND SURFACE					
0.0	Sand and gravel (FILL)					
0.1	Sand, some gravel, containing cobbles (FILL) Red brown Moist	1	AS	-		
346.1	Organic material containing sand and cobbles (FILL) Dark brown to black Wet	2	AS	-	∇ (Elev. 345.9)	
345.8 / 1.4	SAND, trace silt Compact to very loose Grey-brown Wet	3	SS	13		
		4	SS	2		
		5	SS	2		0 94 (6)
		6	SS	1		
342.6 / 4.6	SAND and SILT Very loose to loose Grey Wet	7	SS	1		
		8	SS	2		0 53 47 0
		9	SS	5		
340.6	SAND, some gravel Loose Grey Wet					
340.3 / 6.9	End of Borehole Auger Refusal					

Note:
Water level in open borehole at a depth of 1.2 m (Elev. 345.9 m) upon completion of drilling.

Chart columns: STRAT PLOT; ELEVATION SCALE (347, 346, 345, 344, 343, 342, 341); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³.

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

Golder Associates

RECORD OF BOREHOLE No 07-02

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00
LOCATION N 4971902.5; E 209218.7
ORIGINATED BY P.A.H.
DIST Eastern HWY 62
BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger
COMPILED BY S.L.
DATUM Geodetic
DATE October 17, 2007
CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
346.8	GROUND SURFACE							
0.0	Sand and gravel (FILL)	1	AS	-				
0.2	Grey brown, Moist							
	Sand, some gravel, trace to some silt (FILL), Loose, Red brown, Wet	2	SS	4	346			13 76 (11)
345.7 / 345.4 / 1.4	Sand, some gravel, containing cobbles and organic material (FILL), Grey, Wet							
345.0 / 1.8	PEAT, amorphous, containing silty sand layers, Soft, Dark brown, Wet	3	SS	3	345	o	OC = 8%	
	SAND, trace to some gravel, trace silt, Very loose to compact, Grey-brown to grey, Wet	4	SS	13	344			
		5	SS	2	343			0 96 (4)
		6	SS	2				
		7	SS	2	342			
		8	SS	5	341			
		9	SS	7	340			10 81 (9)
338.9		10	SS	12/0.15	339			
7.9	Schist (BEDROCK), Fresh, Dark grey and olive grey, Medium strong	1	RC	REC 98%	338			RQD = 43%
		2	RC	REC 100%	337			RQD = 92%
336.4 / 10.4	End of Borehole							

Note:
Water level in piezometer at a depth of 0.8 m (Elev. 346.0 m) on Nov. 9, 2007, and at 0.6 m (Elev. 346.2 m) on June 22, 2009.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No 07-03

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00
LOCATION N 4971896.3; E 209211.0
ORIGINATED BY P.A.H.
DIST Eastern HWY 62
BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger
COMPILED BY S.L.
DATUM Geodetic
DATE October 25, 2007
CHECKED BY L.C.C.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³ / REMARKS	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
347.4	GROUND SURFACE								
0.0	ASPHALTIC CONCRETE								
0.4	Sand and gravel (FILL) Red brown Moist	1	AS	-		347			
	Sand, trace gravel (FILL) Compact Red brown Moist to wet	2	SS	16	▽	346			
345.6 / 1.8	PEAT and ALLUVIUM, fibrous Stiff/Compact Dark brown Wet	3	SS	10			~80	OC = 18%	
345.0 / 2.4	SAND Loose to very loose Grey-brown Wet	4	SS	6		345	~37		
		5	SS	5		344			
		6	SS	2		343			
342.8 / 4.6	SAND and SILT, trace gravel Stratified Very loose Dark grey Wet	7	SS	2					
		8	SS	3		342			2 55 (43)
		9	SS	3		341			
339.9 / 7.5	End of Borehole Auger Refusal					340			

Note:
Water level in open borehole at a depth of 1.4 m depth (Elev. 346.0 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L
WATER CONTENT (%): 40 80 120

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

Golder Associates

RECORD OF BOREHOLE No 07-04

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4958474.0; E 216362.9 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE November 2, 2007 CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT / UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.8	GROUND SURFACE					
0.0	Sand and gravel (FILL)					
0.2	Grey brown, Moist					
	Sand and gravel, containing cobbles and asphaltic concrete pieces (FILL), Loose, Brown, Moist	1	SS	5		
308.3 / 1.5	Crushed rock with sand and gravel, trace to some silt, trace clay (FILL), Loose, Grey, Wet	2	SS	4		41 45 12 2
306.8 / 3.1	PEAT, fibrous, Soft, Black to dark brown, Wet	3	SS	3		
		4	SS	3	285.5 OC = 47%	
305.3	SAND, trace gravel, Grey, Wet	5	SS	1/0.08		
4.7	Gabbro (BEDROCK), Fresh, Dark grey to green, Medium strong	1	RC	REC 100%		RQD = 73%
		2	RC	REC 100%		RQD = 44%
		3	RC	REC 100%		RQD = 71%
302.1 / 7.7	End of Borehole					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (309, 308, 307, 306, 305, 304, 303); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³.

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

Golder Associates

RECORD OF BOREHOLE No 07-05

 METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 LOCATION N 4958485.0; E 216373.5 ORIGINATED BY P.A.H.

DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.

DATUM Geodetic DATE November 1, 2007 CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.7	GROUND SURFACE					
0.0	Sand and gravel (FILL) Grey brown Moist					
0.2	Sand and gravel, some silt, containing asphaltic concrete pieces (FILL) Loose to very loose Red brown to brown Moist to wet	1	SS	6		
		2	SS	7		31 51 (18)
		3	SS	3		
307.0 / 2.7	PEAT, fibrous, containing gravel and cobbles in samples 3 and 4 Soft to firm Black Wet	4	SS	10		
		5	SS	WH	216	
305.1 / 4.6	Silty SAND Compact Grey Wet	6	SS	12		
304.4 / 5.3	Gabbro (BEDROCK) Fresh Dark grey to olive Medium strong	1	RC	REC 100%		RQD = 29%
		2	RC	REC 100%		RQD = 50%
		3	RC	REC 100%		RQD = 52%
301.5 / 8.2	End of Borehole Note: Water level in piezometer at a depth of1.8 m (Elev. 307.9 m) on Nov. 9, 2007, and at 1.6 m (Elev. 308.1 m) on June 22, 2009.					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (309, 308, 307, 306, 305, 304, 303, 302); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No 07-06

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4958475.7; E 216370.5 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE November 1, 2007 CHECKED BY L.C.C.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 40 80 120	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.2	GROUND SURFACE										
0.0 / 309.9	ASPHALTIC CONCRETE						310				
0.3	Sand and gravel, trace to some silt (FILL) Loose Brown Moist to wet		1	AS	--						
			2	SS	8		309				
			3	SS	6	∇					37 52 (11)
308.1 / 2.1	Crushed rock with sand and gravel (FILL) Compact Grey Wet						308				
			4	SS	13						
306.9			5	SS	3/0.08		307				
3.3	End of Borehole Sampler Refusal Auger Refusal Note: Water level in open borehole at a depth of 1.8 m depth (Elev. 308.4 m) upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No 07-07

 METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 LOCATION N 4960517.8; E 216158.7 ORIGINATED BY D.G.

DIST Eastern HWY 62 BOREHOLE TYPE Portable Equipment, 50 mm Dia. Solid Stem Augers COMPILED BY S.L.

DATUM Geodetic DATE October 29, 2007 CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT / REMARKS	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.4	GROUND SURFACE						
0.0	Gravelly sand, trace to some silt (FILL) Loose Brown Wet	1	SS	4			
		2	SS	6			24 68 (8)
		3	SS	6			
307.7 / 1.7	Sand, trace gravel, trace to some silt, containing organics (FILL) Loose Black to brown Wet	4	SS	10			
		5	SS	10			
306.3 / 3.1	PEAT, fibrous Soft to firm Black Wet	6	SS	2	647.3		
		7	SS	4	611.2	OC = 89%	
		8	SS	6	587.8		
		9	SS	5	583		
303.9 / 5.5	PEAT, amorphous, some silt Firm Black Wet	10	SS	4	582.2		
		11	SS	6	226.8		
302.7 / 6.7	CLAY, containing organic matter Firm Grey Wet	12	SS	5		OC = 14%	
301.9 / 7.5	SAND Compact Grey Wet	13	SS	15			
301.5 / 7.9	Silty SAND, trace gravel Loose to compact Grey Wet	14	SS	9			
		15	SS	22			4 72 22 2
300.2 / 9.1	SAND, some gravel, trace to some silt Compact to very dense Grey Wet	16	SS	25			
		17	SS	84			20 67 (13)
		18	SS	25/0.08			
298.9 / 10.4	End of Borehole Sampler Refusal						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 309 308 307 306 305 304 303 302 301 300 299

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

Golder Associates

RECORD OF BOREHOLE No 07-08

 METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 LOCATION N 4960520.5; E 216186.5 ORIGINATED BY P.A.H.

DIST Eastern HWY 62 BOREHOLE TYPE Portable Equipment, 50 mm Dia. Solid Stem Augers COMPILED BY S.L.

DATUM Geodetic DATE October 30, 2007 CHECKED BY L.C.C.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (40, 80, 120) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION
309.9	GROUND SURFACE
0.0	Sand, some gravel, trace silt (FILL) Loose to very loose Grey-brown to grey Wet
307.2 2.7	PEAT, fibrous Soft Black Wet
305.0 4.9	PEAT, amorphous, some silt Firm Black Wet
304.6 5.5	SAND Wet
304.0 5.9	CLAYEY SILT Firm Grey Wet
	PEAT, some silt Firm Black Wet
	SAND and GRAVEL, trace to some silt Compact to very dense Grey Wet
300.6 9.3	End of Borehole Sampler Refusal Note: Water level in piezometer at a depth of 0.3 m (Elev. 309.6 m) on Nov. 9, 2007, and on June 18, 2009.

NUMBER	TYPE	"N" VALUES	Water content (%)	Remarks: GR SA SI CL
1	SS	8		14 74 (12)
2	SS	4		
3	SS	2		
4	SS	2	641.4	
5	SS	2	711.8	
6	SS	2	765.5	
7	SS	5	636.1	
8	SS	10	571	
9	SS	27		
10	SS	26		40 53 (7)
11	SS	58		
12	SS	42		
13	SS	39		
14	SS	100/0.20		

Elevation scale: 309, 308, 307, 306, 305, 304, 303, 302, 301

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

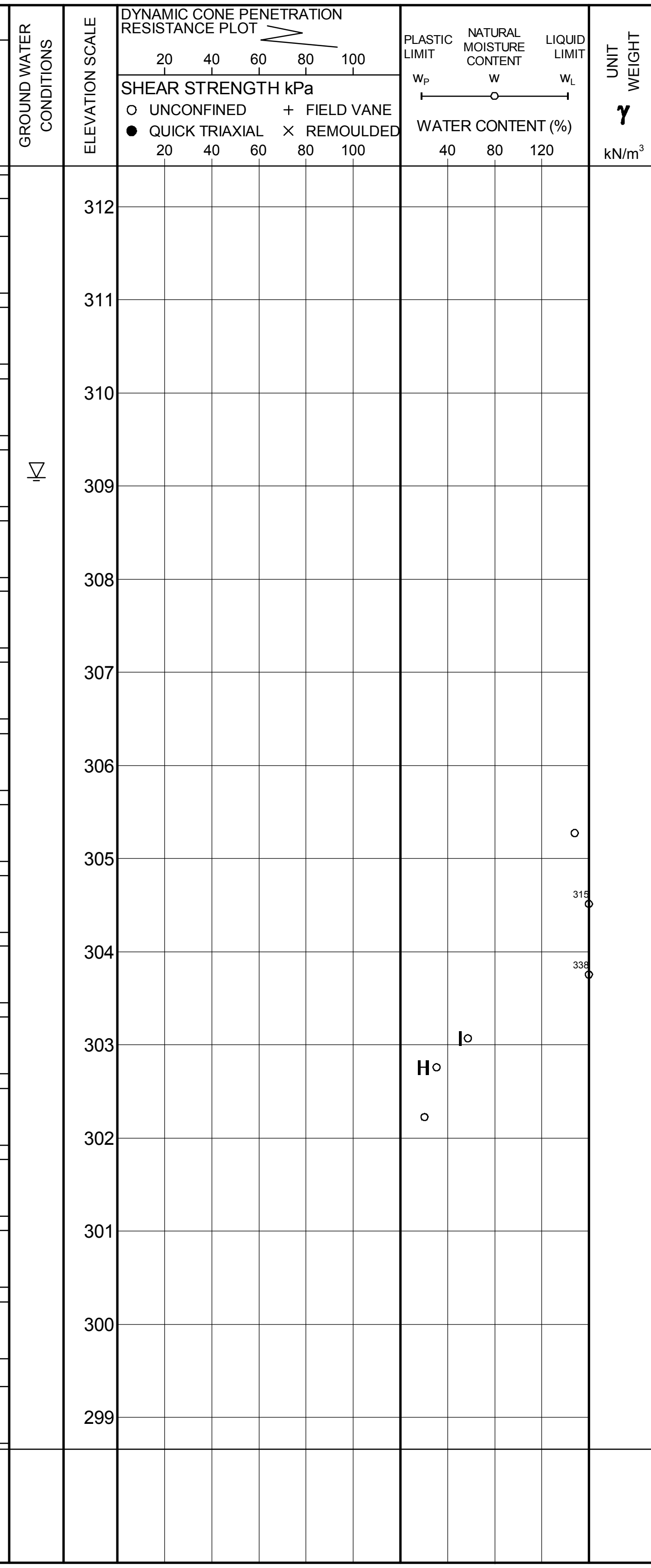
Golder Associates

RECORD OF BOREHOLE No 07-09

 METRIC

PROJECT 07-1111-0044			
W.P. 66-99-00	LOCATION N 4960524.3; E 216174.2	ORIGINATED BY P.A.H.	
DIST Eastern HWY 62	BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger	COMPILED BY S.L.	
DATUM Geodetic	DATE October 31, 2007	CHECKED BY L.C.C.	

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.4	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
0.3	Sand and gravel (FILL) Grey	1	AS	-	
	Silty sand, some gravel (FILL) Compact Brown and red brown Moist to wet	2	SS	21	
		3	SS	17	
		4	SS	17	19 58 (23)
		5	SS	16	
308.6 / 3.8	Sand, some gravel, trace organic matter (FILL) Loose Grey brown Wet	6	SS	4	
307.7 / 4.7	Cobbles and boulders, some sand and gravel (FILL) Compact Wet	7	SS	12	
		8	SS	11	
		9	SS	17	
305.3 / 7.2	PEAT, fibrous Firm Dark brown Wet	10	SS	7	
		11	SS	7	
		12	SS	7	
303.6 / 8.8	CLAYEY SILT Firm Dark brown to grey brown Wet	13	SS	4	
302.8	Silty CLAY Firm Grey Wet				
302.5 / 10.0	CLAYEY SILT containing silty sand layers Stiff Grey Wet	14	SS	8	
301.6 / 10.8	SAND and GRAVEL, trace to some silt Compact Grey Wet	15	SS	21	
		16	SS	24	42 48 9 1
		17	SS	28	
299.3 / 13.1	Silty SAND, some gravel, trace clay (TILL) Very dense Grey Wet	18	SS	180	18 52 27 3
298.7 / 13.8					



Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 07-09

 METRIC

PROJECT 07-1111-0044			
W.P. 66-99-00	LOCATION N 4960524.3; E 216174.2	ORIGINATED BY P.A.H.
DIST Eastern HWY 62	BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger	COMPILED BY S.L.
DATUM Geodetic	DATE October 31, 2007	CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 40 80 120	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	End of Borehole Auger Refusal Note: Water level in open borehole at a depth of 3.3 m (Elev. 309.1 m) upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No 07-10

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4961231.3; E 216098.6 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE November 5, 2007 CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.2	GROUND SURFACE						
310.6 0.0–1.5	Sand and gravel, containing cobbles (FILL) Loose to compact Red brown to grey brown Wet	1	SS	23	∇ (Water level 311.5)		
308.5 1.5–3.7	Sand, some gravel and silt, trace clay (FILL) Loose Brown to grey Wet	2	SS	6			18 60 18 4
		3	SS	4			
		4	SS	7			13 63 (24)
307.4 3.7–4.7	Sand, trace to some organic matter (FILL) Loose Grey Wet	5	SS	4			
304.2 4.7–7.9	PEAT, fibrous Soft to firm Dark brown to black Wet	6	SS	8		175	
		7	SS	5			
		8	SS	2			
		9	SS	2		257.3	
		10	SS	3		280.6	
303.1 7.9–9.1	CLAYEY SILT Soft to firm Grey to brown Wet	11	SS	3			
302.4 9.1–9.8	SAND, some silt, trace to some gravel, trace clay Loose Grey Wet	12	SS	7			9 71 18 2
9.8	End of Borehole Sampler Refusal						

Note:
Water level in open borehole at a depth of 0.7 m (Elev. 311.5 m) upon completion of drilling.

Elevation scale: 312, 311, 310, 309, 308, 307, 306, 305, 304, 303

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120
UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No 07-11

METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 | LOCATION N 4961231.4; E 216109.5 | ORIGINATED BY P.A.H.

DIST Eastern HWY 62 | BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger | COMPILED BY S.L.

DATUM Geodetic | DATE October 30, 2007 | CHECKED BY L.C.C.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
312.3	GROUND SURFACE
0.0	Sand and gravel, containing cobbles (FILL) Compact Brown to red-brown Moist to wet
310.8 / 1.5	Sand and gravel, containing organic matter (FILL) Very loose Dark grey-brown Wet
310.2 / 2.1	Sand, trace to some gravel, containing organic matter (FILL) Very loose to loose Grey Wet
308.0 / 4.3	PEAT, fibrous Soft to firm Dark brown to black Wet
305.0 / 7.3	CLAYEY SILT Soft to firm Dark brown Wet
304.5 / 7.8	Silty SAND, trace gravel and silt Loose Grey Wet
303.8 / 8.5	End of Borehole Auger Refusal

Note:
Water level in piezometer at a depth of 0.7 m (Elev. 311.6 m) on Nov. 9, 2007, and at 0.6 m (Elev. 311.7 m) on June 18, 2009.

SAMPLES

NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)
1	SS	14	
2	SS	2	
3	SS	1	
4	SS	5	
5	SS	4	
6	SS	4	
7	SS	2	354.4
8	SS	3	551.7
9	SS	3	356.9
10	SS	5	

GROUND WATER CONDITIONS | ELEVATION SCALE: 312, 311, 310, 309, 308, 307, 306, 305, 304

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 40 80 120

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

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

RECORD OF BOREHOLE No 07-12

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00
LOCATION N 4961225.1; E 216106.7
ORIGINATED BY P.A.H.
DIST Eastern HWY 62
BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger
COMPILED BY S.L.
DATUM Geodetic
DATE October 29, 2007
CHECKED BY L.C.C.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L | WATER CONTENT (%) (40, 80, 120) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS (GR SA SI CL)
312.6	GROUND SURFACE							
0.0	ASPHALTIC CONCRETE							
0.3	Sand and gravel (FILL) Red brown		1	AS	--			
	Sand, some gravel, trace to some silt, trace clay (FILL) Loose to dense Brown and dark grey Wet		2	SS	19	▽		18 69 (13)
			3	SS	8			
			4	SS	19			
			5	SS	20			18 60 19 3
			6	SS	39			
			7	SS	19			
307.4 5.2	PEAT, fibrous Soft to firm Dark brown Wet		8	SS	3		245.3	
			9	SS	5		261.2	OC = 73%
			10	SS	5		354.8	
			11	SS	5		314.9	
303.4	SAND, some silt, trace clay Loose Grey Wet		12	SS	4			0 77 (23)
303.0 9.6	End of Borehole Sampler Refusal							

Elevation scale: 312, 311, 310, 309, 308, 307, 306, 305, 304

Note:
Water level in open borehole at a depth of 1.1 m (Elev. 311.5 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No 07-13

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4961414.4; E 216080.3 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE October 30, 2007 CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.0	GROUND SURFACE						
0.0	Crushed stone (FILL)						
0.2	Grey Gravelly sand, trace to some silt (FILL) Compact Brown Wet	1	SS	10	311		24 67 (9)
310.7 / 1.4	Sand, some gravel, containing asphaltic concrete pieces and organic matter (FILL) Very loose to loose Grey brown Wet	2	SS	9	310		
		3	SS	2			
		4	SS	4	309		
308.4 / 3.7	PEAT, fibrous Soft Dark brown Wet	5	SS	2	308		
		6	SS	3	307	433.3	
		7	SS	2		524.8	
		8	SS	>50	306		
305.5 / 6.5	End of Borehole Sampler Refusal Auger Refusal						

Note:
Water level in piezometer at a depth of 0.7 m (Elev. 311.3 m) on Nov. 9, 2007, and at 0.6 m (Elev. 311.4 m) on June 18, 2009.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No 07-14

 METRIC

PROJECT 07-1111-0044			
W.P. 66-99-00	LOCATION N 4961414.6; E 216092.9		ORIGINATED BY P.A.H.
DIST Eastern HWY 62	BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger		COMPILED BY S.L.
DATUM Geodetic	DATE October 29, 2007		CHECKED BY L.C.C.

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (40, 80, 120); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS / GR SA SI CL
312.0	GROUND SURFACE						
0.0	Sand and gravel, containing cobbles (FILL) Compact Red brown, brown and grey Wet	1	SS	21	311		
310.7 1.4	Sand, trace gravel and silt (FILL) Very loose Grey Wet	2	SS	1	310		1 95 (4)
		3	SS	WH	309		
308.7 3.4	PEAT, fibrous Soft to firm Dark brown Wet	4	SS	WH		353.9	
		5	SS	2	308	232.4	
		6	SS	4	307	390.2	
		7	SS	3		344.5	OC = 69%
		8	SS	2	306	171.1	
305.2 6.9	SAND and SILT, trace to some gravel, trace clay, containing cobbles Very loose to loose Grey Wet	9	SS	5	305	o	5 41 49 5
		10	SS	1	304		4 49 (47)
		11	SS	7	303	o	18 42 37 3
302.4 9.6	End of Borehole Auger Refusal Note: Wet soils encountered during drilling below a depth of 0.8 m (Elev. 311.2 m).						

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

Golder Associates

RECORD OF BOREHOLE No 07-15

 METRIC

PROJECT 07-1111-0044			
W.P. 66-99-00	LOCATION N 4961421.2; E 216087.6		ORIGINATED BY P.A.H.
DIST Eastern HWY 62	BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger		COMPILED BY S.L.
DATUM Geodetic	DATE October 26, 2007		CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.4	GROUND SURFACE							
0.0	ASPHALTIC CONCRETE	1	AS	-				
0.3	Sand and gravel (FILL) Red brown and grey					312		
	Silty sand, trace to some gravel, containing cobbles (FILL) Compact Grey-brown and red brown Wet	2	SS	17	∇			
		3	SS	14		311		
		4	SS	22		310		
		5	SS	19		309		10 62 (28)
		6	SS	21				
308.0 / 4.4	Crushed rock, some sand and gravel (FILL)					308		
		7	SS	17			330.9	
307.4 / 5.0	PEAT, fibrous Firm Dark brown Wet					307		
		8	SS	4			353.6	
		9	SS	4		306	411.7	
		10	SS	4			467.3	
304.9 / 7.5	Silty SAND Loose Dark brown to grey Wet					305		
304.5 / 7.9	SILTY CLAY and CLAYEY SILT, some sand Soft Grey Wet	11	SS	2		304		
303.8 / 8.6	CLAYEY SILT, some sand Very stiff Grey Wet	12	SS	20/0.10				
303.0 / 9.4	End of Borehole Sampler Refusal Auger Refusal							

Note:
Water level in open borehole at a depth of 0.9 m (Elev. 311.5 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120

UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No 07-16

 METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 LOCATION N 4962141.9; E 215965.0 ORIGINATED BY P.A.H.

DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.

DATUM Geodetic DATE October 24, 2007 CHECKED BY L.C.C.

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.0	GROUND SURFACE						
0.0	Sand and gravel (FILL) Loose Brown to grey-brown Moist to wet	1	AS	-	∇ (Elev. 311.2)		
		2	SS	8			
310.5 / 1.5	Gravelly sand, trace silt, containing cobbles and organic matter (FILL) Very loose to loose Grey to black Wet	3	SS	4			26 67 (7)
		4	SS	8			
308.9 / 3.1	PEAT, fibrous, Very soft to firm Dark brown Wet	5	SS	1			
		6	SS	1		529.4	OC = 84%
		7	SS	6		559.1	
306.9 / 5.1	Schist (BEDROCK), containing thin calcite bands Fresh Olive grey to green Medium strong	1	RC	REC 100%			RQD = 66%
		2	RC	REC 100%			RQD = 61%
303.9 / 8.1	End of Borehole Note: Water level in open borehole at a depth of 0.8 m (Elev. 311.2 m) upon completion of drilling.						

Elevation scale: 311, 310, 309, 308, 307, 306, 305, 304

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120

UNIT WEIGHT γ kN/m³

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

MIS-MTO 001 07-1111-0044.GPJ GAL-MISS.GDT 6/24/09 ACM/SAC

Golder Associates

RECORD OF BOREHOLE No 07-17

 METRIC

PROJECT 07-1111-0044			
W.P. 66-99-00	LOCATION N 4962144.4; E 215975.8	ORIGINATED BY P.A.H.	
DIST Eastern HWY 62	BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger	COMPILED BY S.L.	
DATUM Geodetic	DATE October 24, 2007	CHECKED BY L.C.C.	

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT / REMARKS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.2	GROUND SURFACE					
0.0	Sand and gravel (FILL), Loose, Grey to brown, Wet	1	SS	4		
310.7	Sand, some gravel, containing organic matter (FILL), Very loose, Grey, Wet	2	SS	2		
310.4 / 1.8	PEAT, fibrous, Soft, Dark brown, Wet	3	SS	1	OC = 28%	
		4	SS	1	402.3	
308.5 / 3.7	SAND and SILT, trace to some gravel, trace clay, Compact, Grey, Wet	5	SS	23		6 57 34 3
307.8 / 4.4	Schist (BEDROCK), containing calcite seams, Fresh, Olive grey to green, Medium strong	1	RC	REC 100%		RQD = 60%
		2	RC	REC 100%		RQD = 100%
304.9 / 7.3	End of Borehole					

Note:
Water level in piezometer at a depth of 0.8 m (Elev. 311.4 m) on Nov. 9, 2007, and at 0.6 m (Elev. 311.6 m) on June 18, 2009.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (312–305); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³.

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

Golder Associates

RECORD OF BOREHOLE No 07-18

 METRIC

PROJECT 07-1111-0044			
W.P. 66-99-00	LOCATION N 4962137.5; E 215974.2	ORIGINATED BY P.A.H.
DIST Eastern HWY 62	BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger	COMPILED BY S.L.
DATUM Geodetic	DATE October 25, 2007	CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
312.5	GROUND SURFACE							
0.0	ASPHALTIC CONCRETE							
0.1	Sand and gravel, containing organic matter (FILL) Compact Brown to grey to red-brown Moist to wet					312		
		1	SS	18	∇			
						311		
		2	SS	13				
310.5 / 2.0	PEAT, fibrous Soft Dark brown Wet					310		
		3	SS	3			310.6	
		4	SS	2			160.1	
309.0 / 3.7	SAND and GRAVEL Loose Grey Wet					309		
	End of Borehole Auger Refusal							

Note:
Water level in open borehole at a depth of 1.1 m (Elev. 311.4 m) upon completion of drilling.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³; STRAT PLOT

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

Golder Associates

RECORD OF BOREHOLE No 07-19

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4962589.9; E 215813.5 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE October 18 and 22, 2007 CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.4	GROUND SURFACE								
0.0	Sand and gravel, trace to some silt, containing cobbles (FILL) Very loose Grey brown to red brown Moist to wet	1	SS	3		313			
		2	SS	3	∇				
		3	SS	3		312			
311.4 2.0	PEAT, fibrous Soft Dark brown Wet	4	SS	2		311	381.3		OC = 64%
		5	SS	4/0.15					
310.2 3.2	Schist (BEDROCK) Slightly weathered Olive grey Medium strong	1	RC	REC 100%		310			RQD = 32%
		2	RC	REC 88%					RQD = 0%
309.0 4.4	Schist (BEDROCK), containing quartzite and pyrite intrusions Fresh Olive grey to green Medium strong	3	RC	REC 100%		309 308			RQD = 55%
307.6 5.9	End of Borehole Note: Water level in open borehole at a depth of 1.1 m (Elev. 312.3 m) upon completion of drilling.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120

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

Golder Associates

RECORD OF BOREHOLE No 07-20

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4962593.7; E 215824.9 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE October 22, 2007 CHECKED BY L.C.C.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.5	GROUND SURFACE						
0.0	ASPHALT	1	AS	-			
0.2	Sand and gravel (FILL) Brown				313		
	Gravelly sand, trace to some silt (FILL) Compact Red brown to grey brown Moist to wet	2	SS	14			28 62 (10)
311.8 / 1.7	Sand, some gravel, containing organic matter (FILL) Loose Brown and grey Wet	3	SS	5	312		
311.4 / 2.1	PEAT, amorphous Firm Brown Wet						
311.1 / 2.4	Sandy SILT, trace to some gravel, containing organic matter Compact Grey Wet	4	SS	12	311	~78	
310.3 / 3.2	Schist (BEDROCK), containing quartzite and pyrite intrusions Fresh Olive grey Medium strong	5	SS	>50			
		1	RC	REC 100%	310		RQD = 86%
		2	RC	REC 100%	309 / 308		RQD = 49%
307.5 / 6.0	End of Borehole						

Note:
Water level in piezometer at a depth of 1.2 m (Elev. 312.3 m) on Nov. 9, 2007, and at 1.1 m (Elev. 312.4 m) on June 18, 2009.

Column headers also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³.

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

RECORD OF BOREHOLE No 07-21

 METRIC

PROJECT 07-1111-0044
W.P. 66-99-00 LOCATION N 4962585.1; E 215824.3 ORIGINATED BY P.A.H.
DIST Eastern HWY 62 BOREHOLE TYPE CME-75, 108 mm I.D. Hollow Stem Auger COMPILED BY S.L.
DATUM Geodetic DATE October 25, 2007 CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.8	GROUND SURFACE						
0.0	ASPHALTIC CONCRETE						
	Sand and gravel (FILL) Red brown	1	AS	--			
0.3	Sand, some gravel, trace to some silt (FILL) Compact Grey brown Moist	2	SS	12	∇	313	20 71 (9)
312.3 / 1.5	Sand and gravel, containing cobbles (FILL) Very dense Grey-brown Wet	3	SS	>50		312	
311.6 / 2.1	End of Borehole Sampler Refusal Auger Refusal						

Note:
Water level in open borehole at a depth of 1.4 m (Elev. 312.4 m) upon completion of drilling.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 40 80 120; UNIT WEIGHT γ kN/m³

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

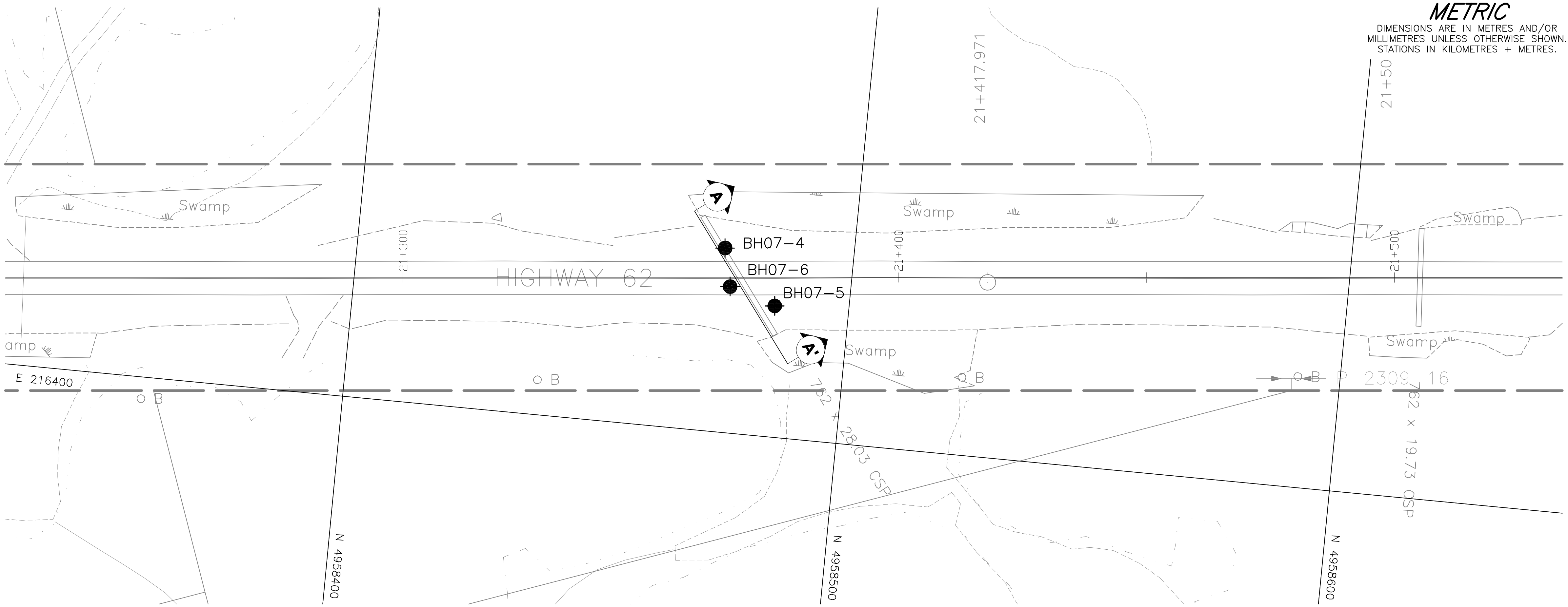
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 66-99-00

SHEET

HIGHWAY 62
Culvert at Station 21+369
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE
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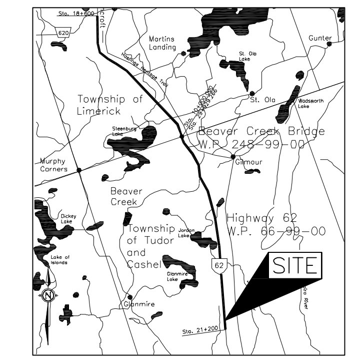
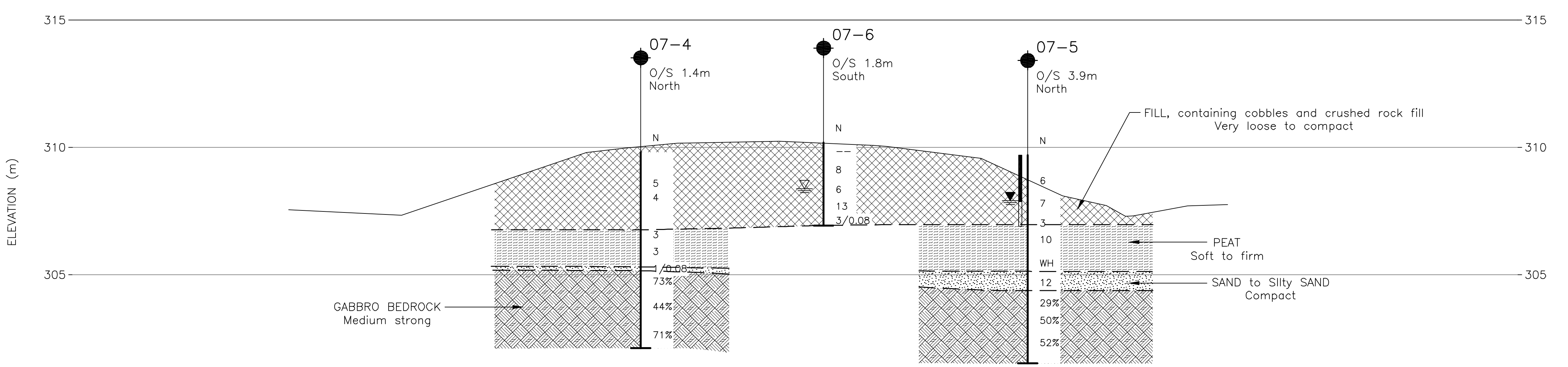


KEY PLAN
NOT TO SCALE



A-A'
1

Section A-A'
Culvert at Station 21+369

SCALE
VER 2 0 2 4 m

SCALE
HOR 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 9, 2007
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
07-4	309.8	4958474.0	216362.9
07-5	309.7	4958485.0	216373.5
07-6	310.2	4958475.7	216370.5

NOTES

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REFERENCE

Base plans provided in digital format from Genivar (Drawing File No "LIMERICK-PLAN.dwg" and "TUDOR-baseplan.dwg", received Nov. 14, 2007).

NO.	DATE	BY	REVISION

Geocres No. 31F-169

HWY.		PROJECT NO. 07-1111-0044	DIST.
SUBM'D. PAH	CHKD. SH	DATE: 10-Mar-2009	SITE:
DRAWN: DD	CHKD. SH	APPD. LCC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: June 24, 2009

MINISTRY OF TRANSPORTATION, ONTARIO

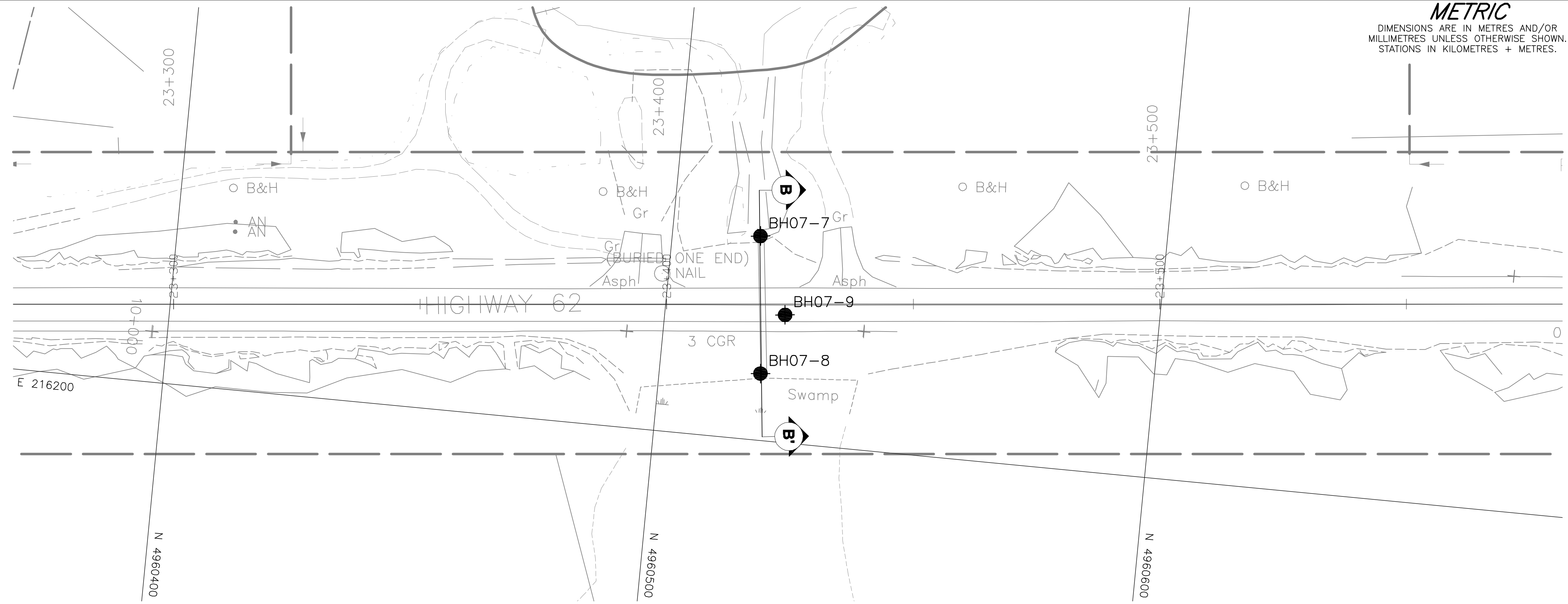
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 66-99-00

SHEET

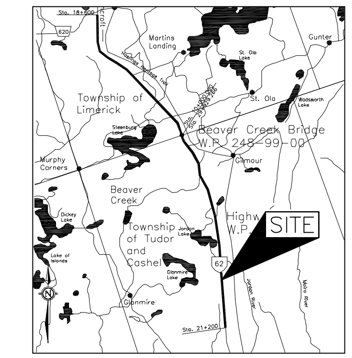
HIGHWAY 62
Culvert at Station 23+418
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

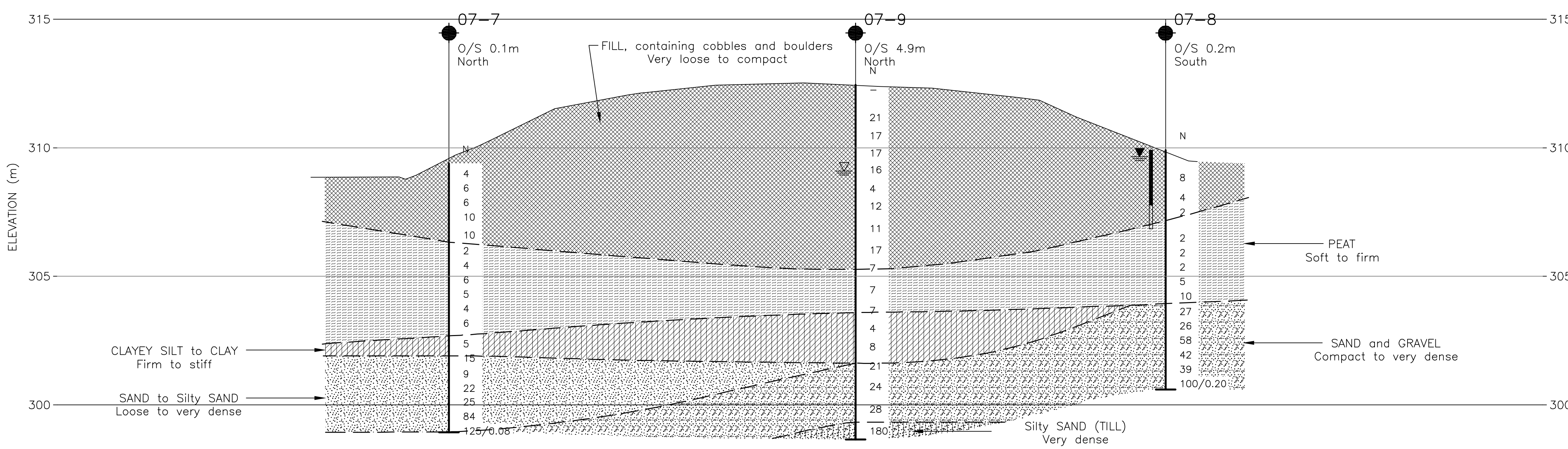


PLAN

SCALE 10 0 10 20 m



KEY PLAN
NOT TO SCALE



Section B-B'
Culvert at Station 23+418

SCALE
VER 2 0 2 4 m

SCALE
HOR 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 9, 2007
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
07-7	309.4	4960517.8	216158.7
07-8	309.9	4960520.5	216186.5
07-9	312.4	4960524.3	216174.2

NOTES

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REFERENCE

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NO.	DATE	BY	REVISION

Geocres No. 31F-169

HWY.		PROJECT NO. 07-1111-0044	DIST.
SUBM'D. PAH	CHKD. SH	DATE: 10-Mar-2009	SITE:
DRAWN: DD	CHKD. SH	APPD. LCC	DWG. 2

PLOT DATE: June 24, 2009

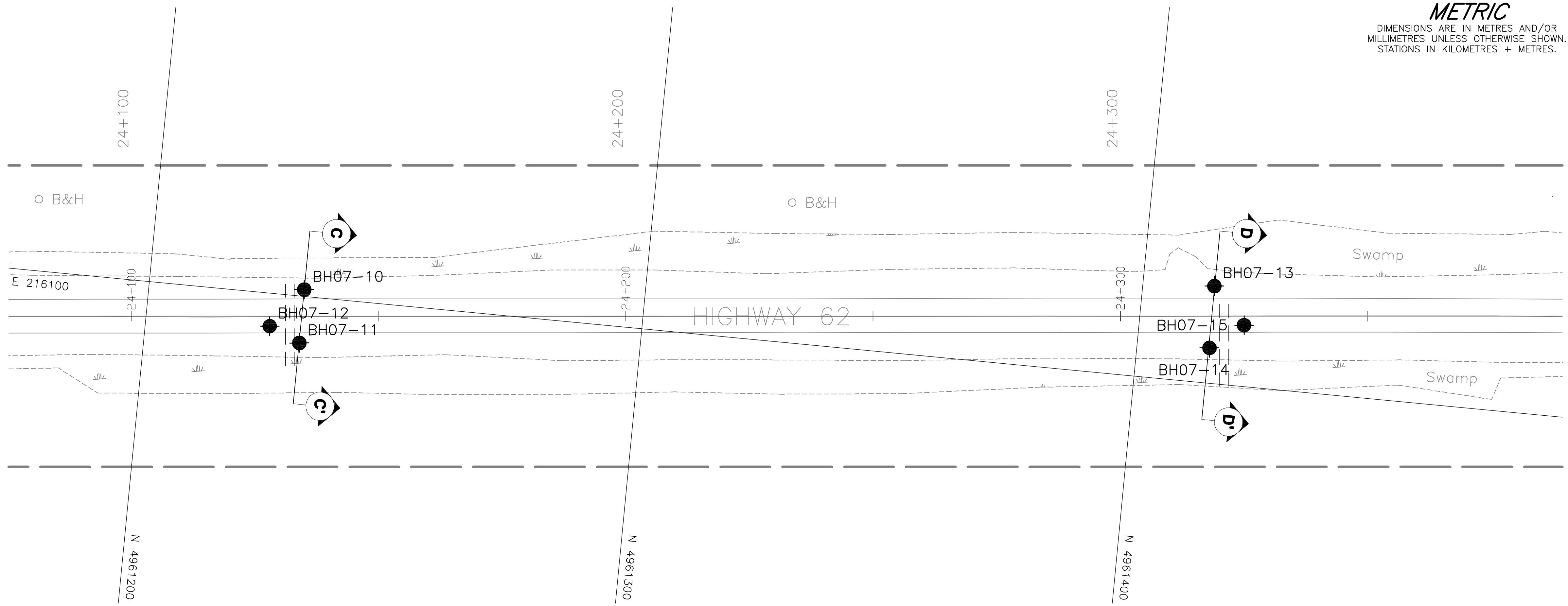
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 66-99-00

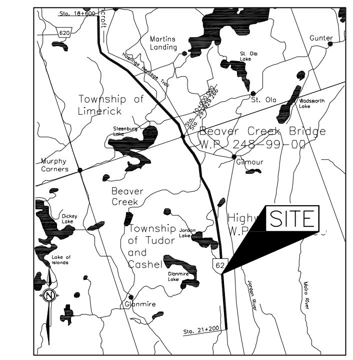
SHEET

HIGHWAY 62
Culverts at Station 24+124/24+126 and 24+320/24+322
BOREHOLE LOCATIONS AND SOIL STRATA

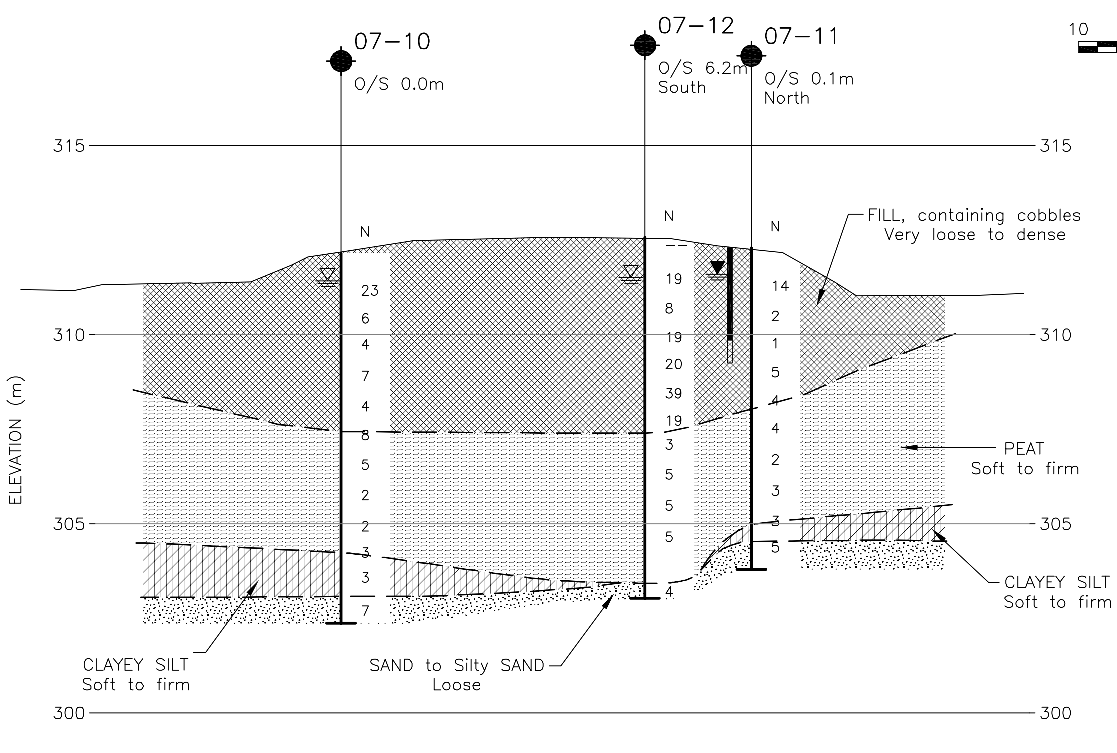
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



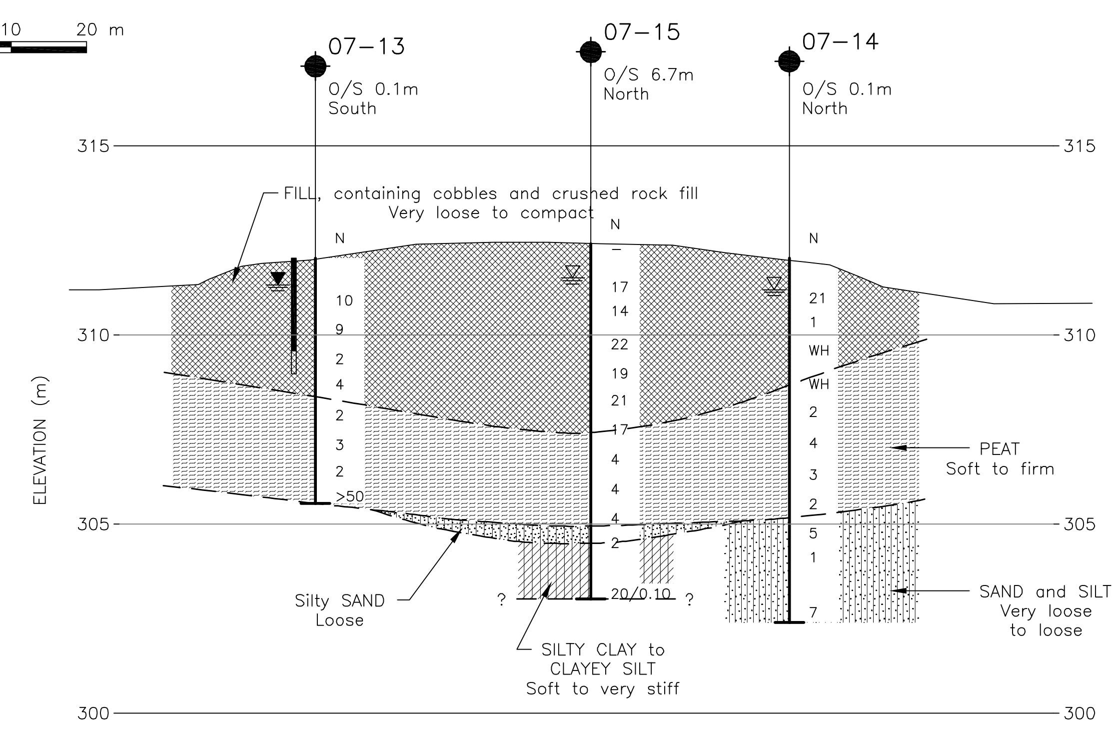
PLAN



KEY PLAN
NOT TO SCALE



Section C–C'
Culverts at Station 24+124/24+126



Section D–D'
Culverts at Station 24+320/24+322

SCALE
VER 2 0 2 4 m

SCALE
HOR 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 9, 2007
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
07-10	312.2	4961231.3	216098.6
07-11	312.3	4961231.4	216109.5
07-12	312.6	4961225.1	216106.7
07-13	312.0	4961414.4	216080.3
07-14	312.0	4961414.6	216092.9
07-15	312.4	4961421.2	216087.6

NOTES

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REFERENCE

Base plans provided in digital format from Genivar (Drawing File No "LIMERICK-PLAN.dwg" and "TUDOR-baseplan.dwg", received Nov. 14, 2007).

NO.	DATE	BY	REVISION

Geocres No. 31F-169

HWY.	PROJECT NO. 07-1111-0044		DIST.
SUBM'D. PAH	CHKD. SH	DATE: 11-Mar-2009	SITE:
DRAWN: DD	CHKD. SH	APPD. LCC	DWG. 3

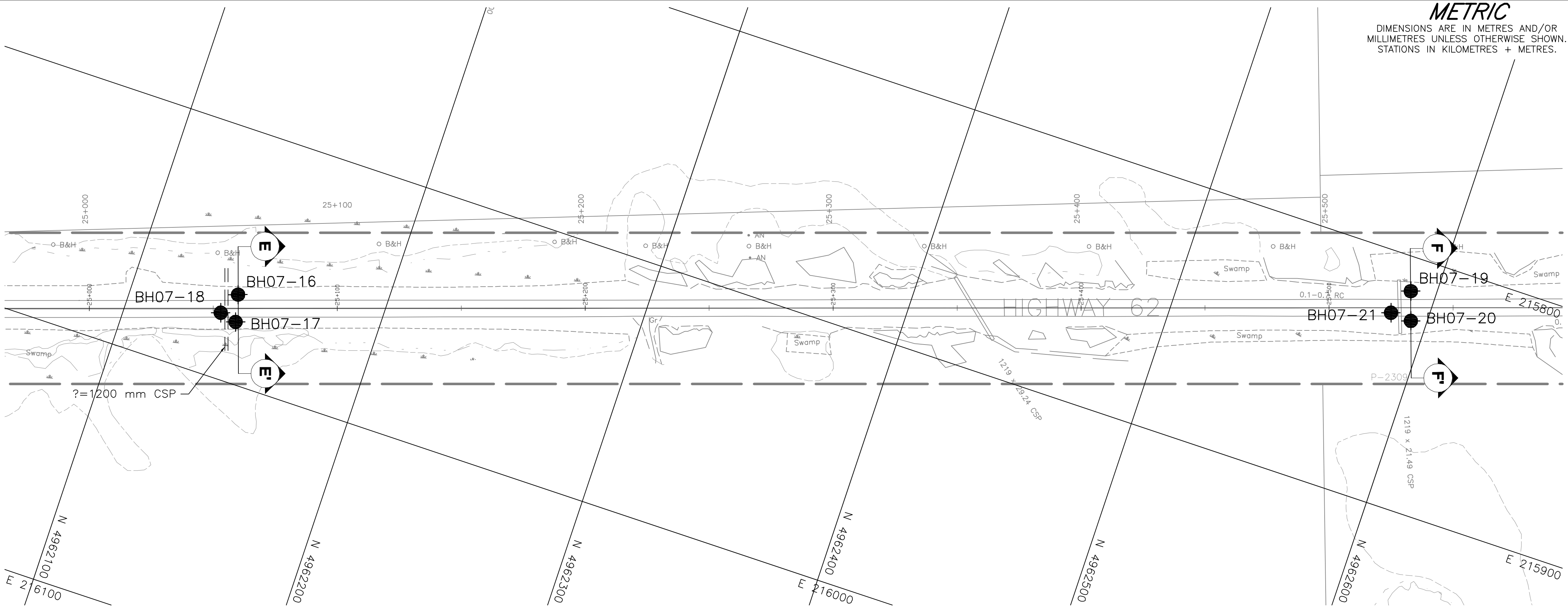
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 66–99–00

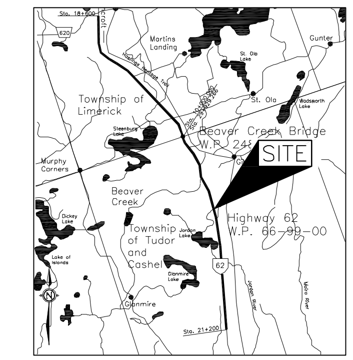
SHEET

HIGHWAY 62
Culvert at Station 25+057 and 25+529
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

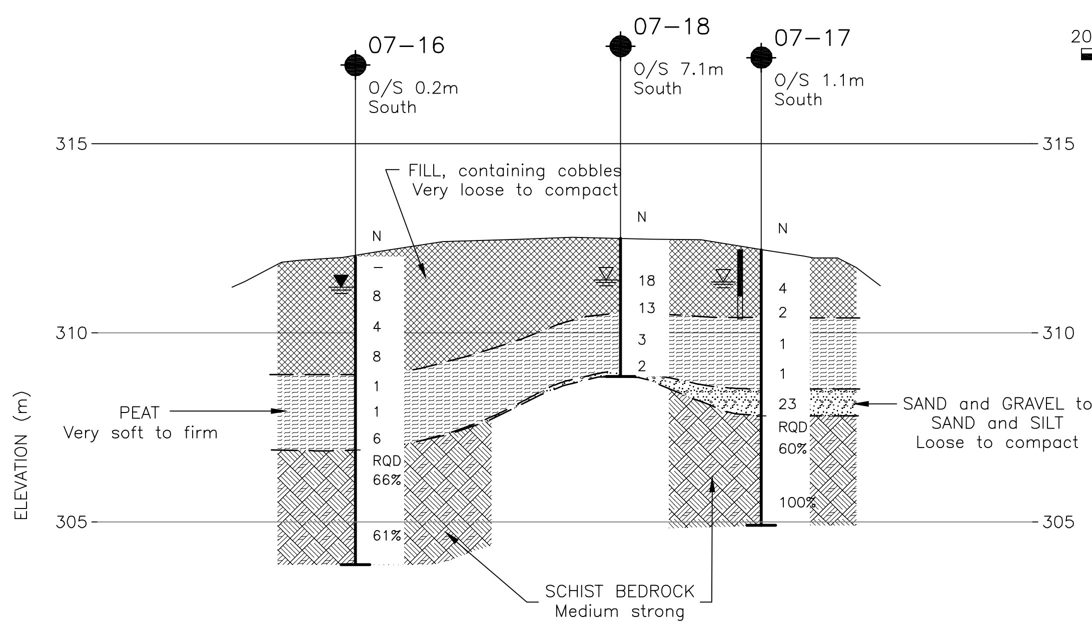


PLAN

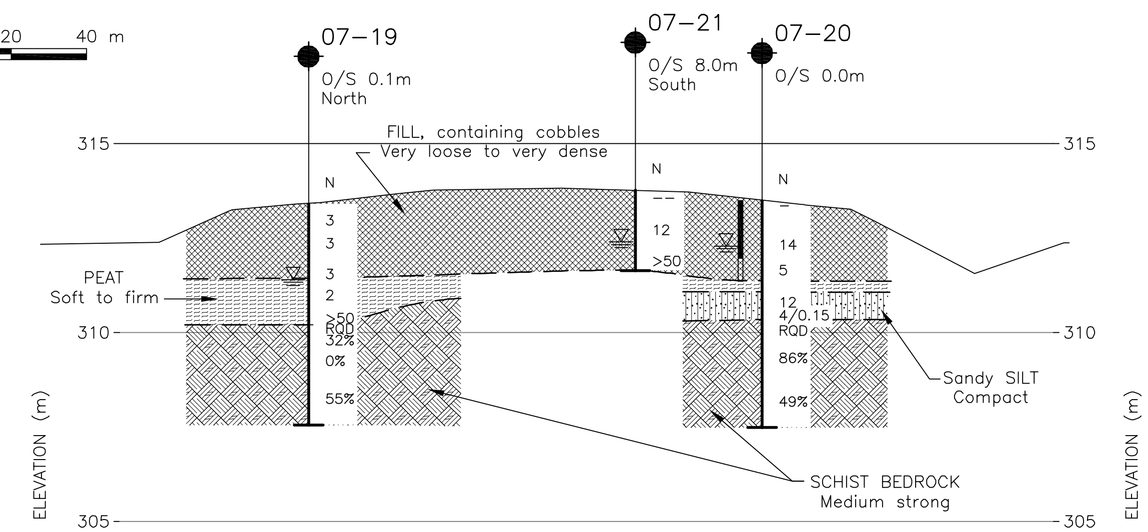


KEY PLAN
NOT TO SCALE

SCALE
20 0 20 40 m



Section E–E'
Culvert at Station 25+057



Section F–F'
Culvert at Station 25+529

SCALE
VER 2 0 2 4 m

SCALE
HOR 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 9, 2007
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES NORTHING	EASTING
07–16	312.0	4962141.9	215965.0
07–17	312.2	4962144.4	215975.8
07–18	312.5	4962137.5	215974.2
07–19	313.4	4962589.9	215813.5
07–20	313.5	4962593.7	215824.9
07–21	313.8	4962585.1	215824.3

NOTES

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REFERENCE

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NO.	DATE	BY	REVISION

Geocres No. 31F–169

HWY.		PROJECT NO. 07–1111–0044	DIST.
SUBM'D. PAH	CHKD. SH	DATE: 10–Mar–2009	SITE:
DRAWN: DD	CHKD. SH	APPD. LCC	DWG. 4

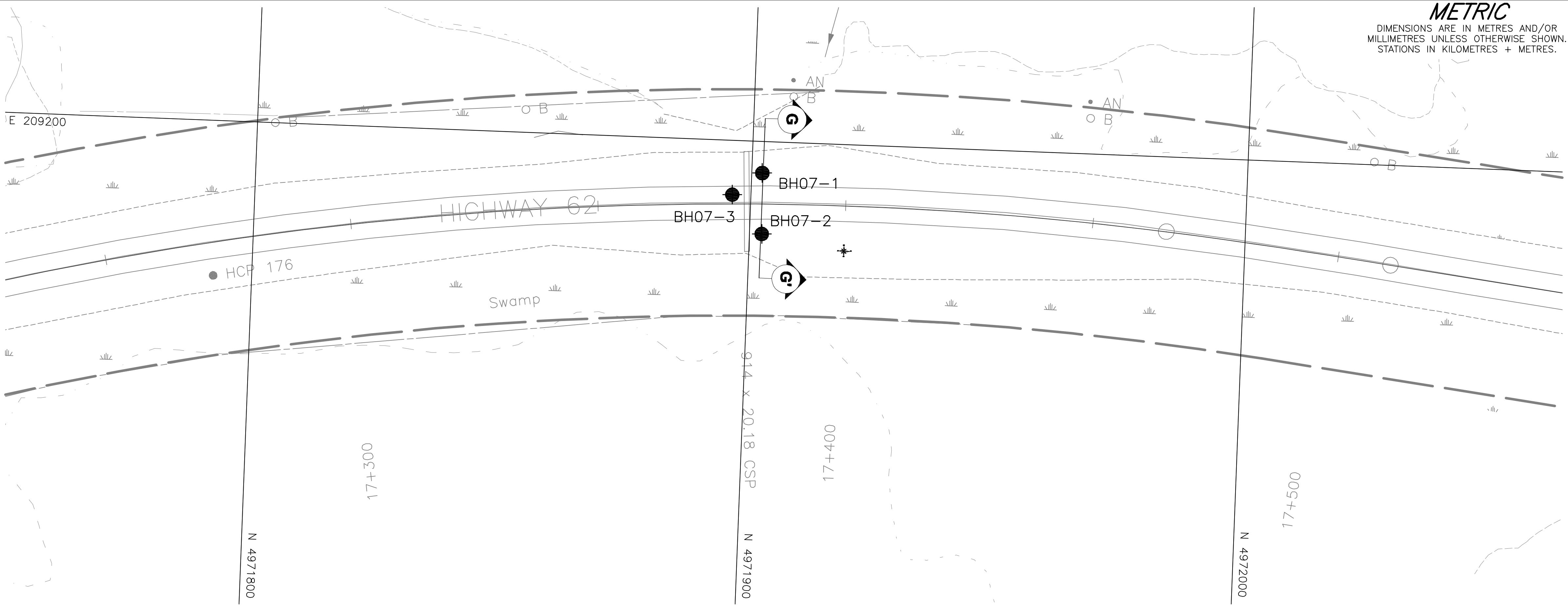
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 66-99-00

SHEET

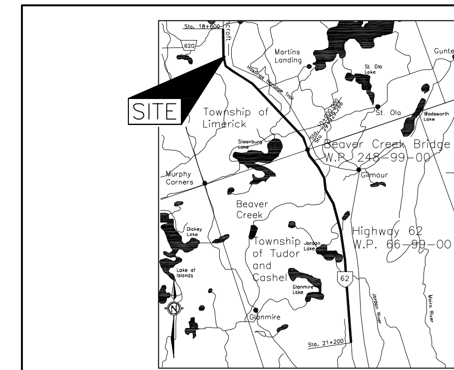
HIGHWAY 62
Culvert at Station 17+379
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

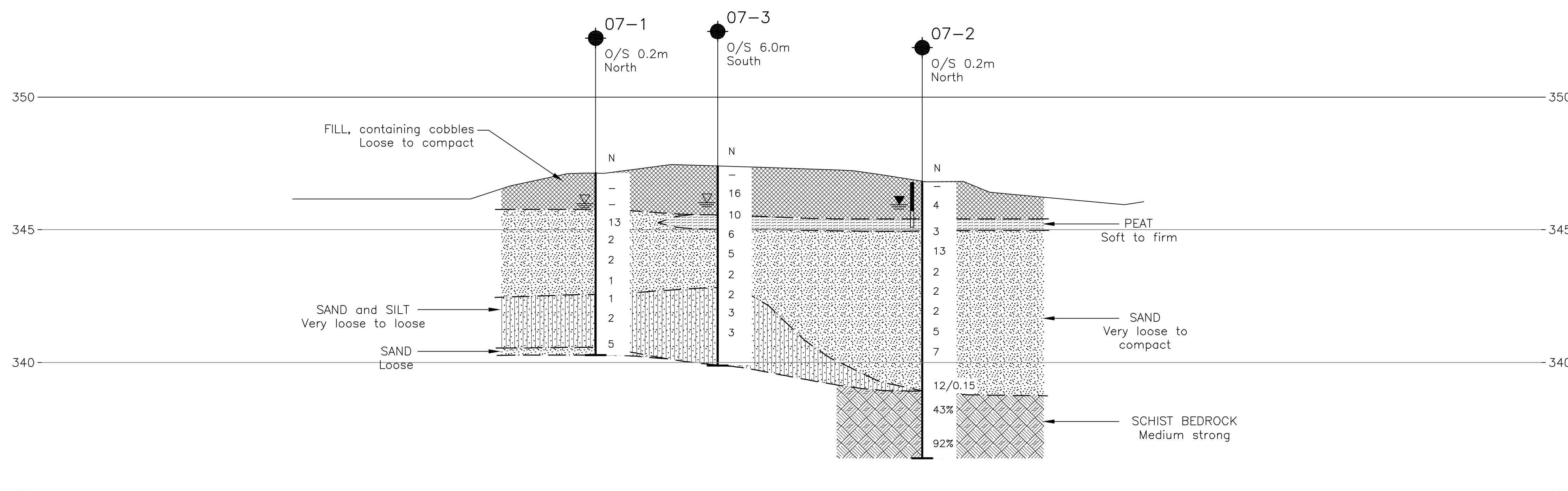


PLAN

SCALE
10 0 10 20 m



KEY PLAN
NOT TO SCALE



Section G-G'
Culvert at Station 17+379

SCALE
VER 2 0 2 4 m

SCALE
HOR 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 9, 2007
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
07-1	347.1	4971902.1	209206.4
07-2	346.8	4971902.5	209218.7
07-3	347.4	4971896.3	209211.0

NOTES

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REFERENCE

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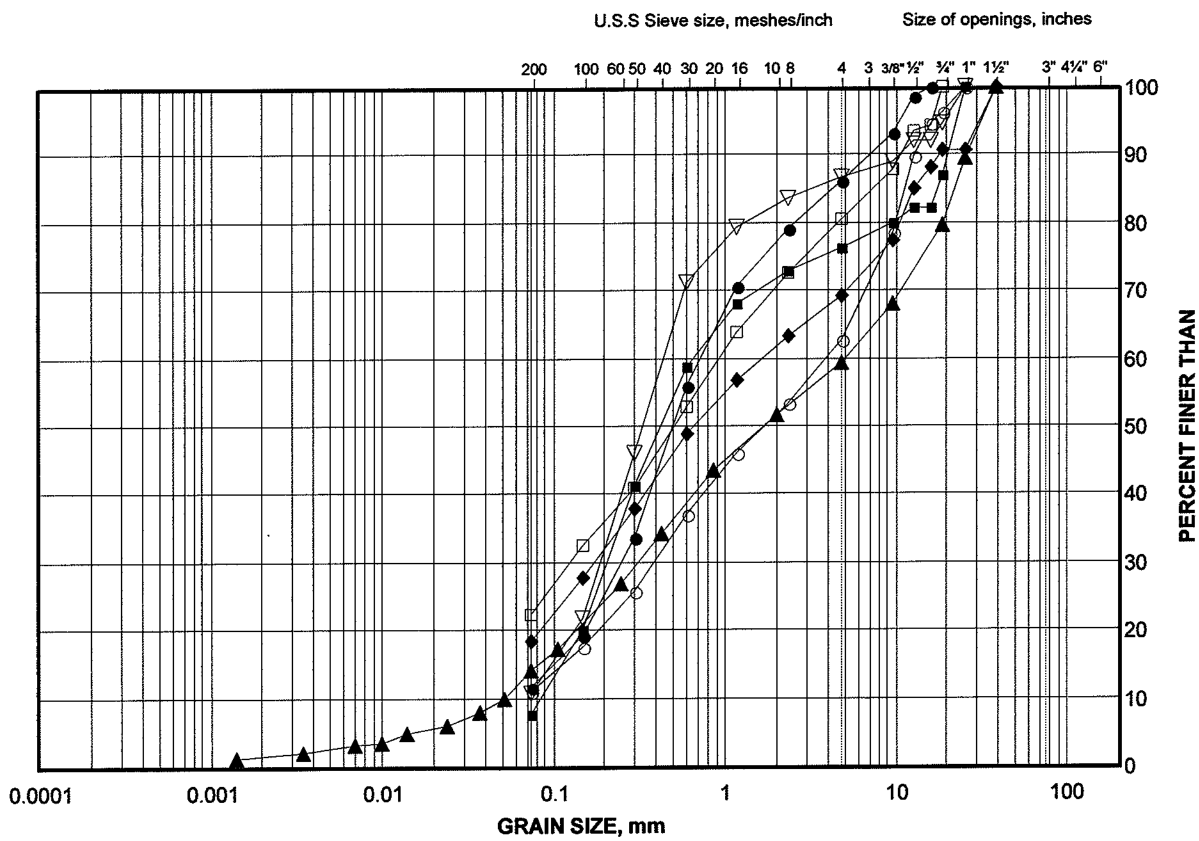
NO.	DATE	BY	REVISION

Geocres No. 31F-169

HWY.		PROJECT NO. 07-1111-0044	DIST.
SUBM'D. PAH	CHKD. SH	DATE: 10-Mar-2009	SITE:
DRAWN: DD	CHKD. SH	APPD. LCC	DWG. 5

GRAIN SIZE DISTRIBUTION TEST RESULTS Sand and Gravel to Silty Sand Fill	FIGURE 1A



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-8	1	1.0
■	07-7	2	0.9
◆	07-5	2	1.8
▲	07-4	2	1.8
▽	07-2	2	1.0
○	07-6	3	1.8
□	07-9	4	2.6

Project Number: 07-1111-0044

Checked By: [signature]

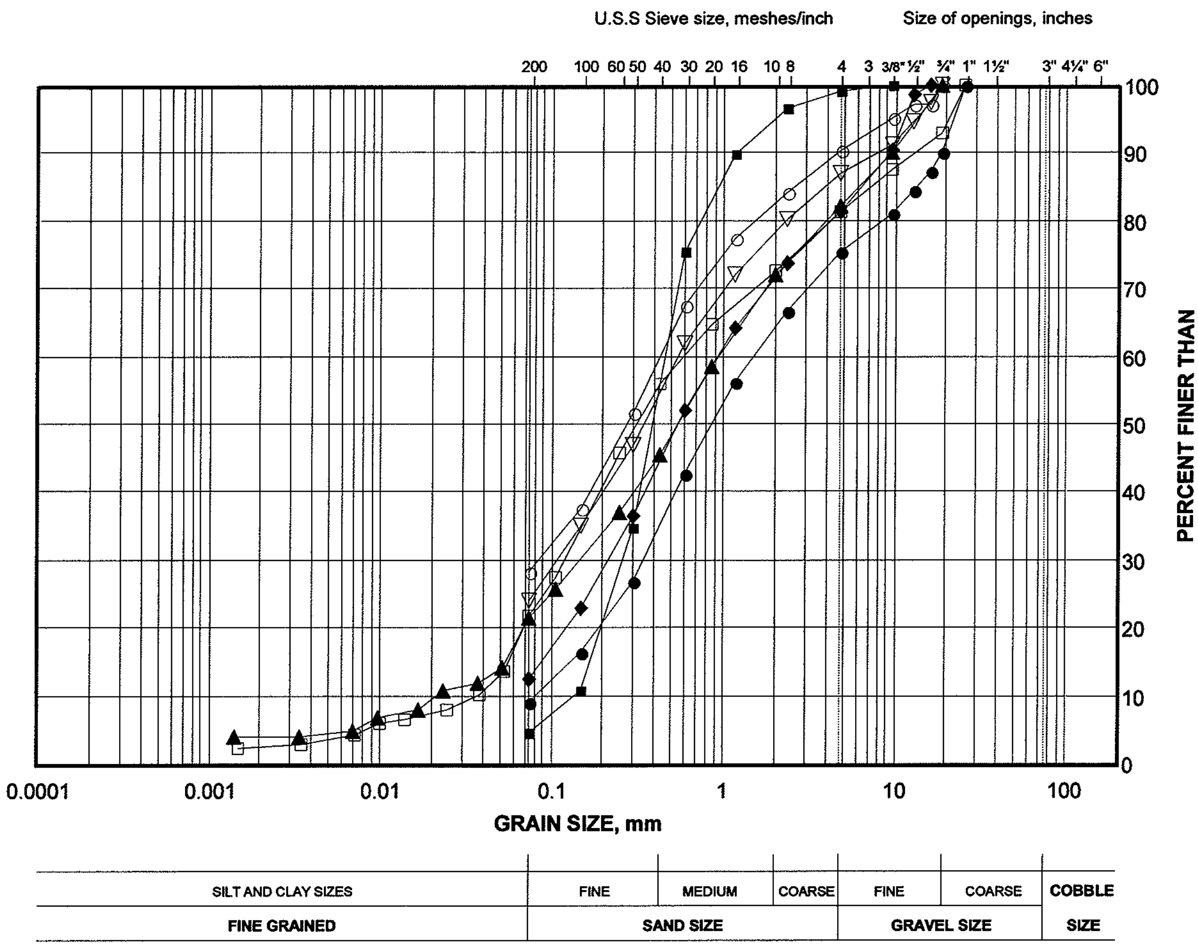
Golder Associates

Date: 04-Mar-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Silty Sand Fill

FIGURE 1B



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-13	1	1.1
■	07-14	2	1.8
◆	07-12	2	1.1
▲	07-10	2	1.8
▽	07-10	4	3.4
○	07-15	5	3.4
□	07-12	5	3.4

Project Number: 07-1111-0044

Checked By: [signature]

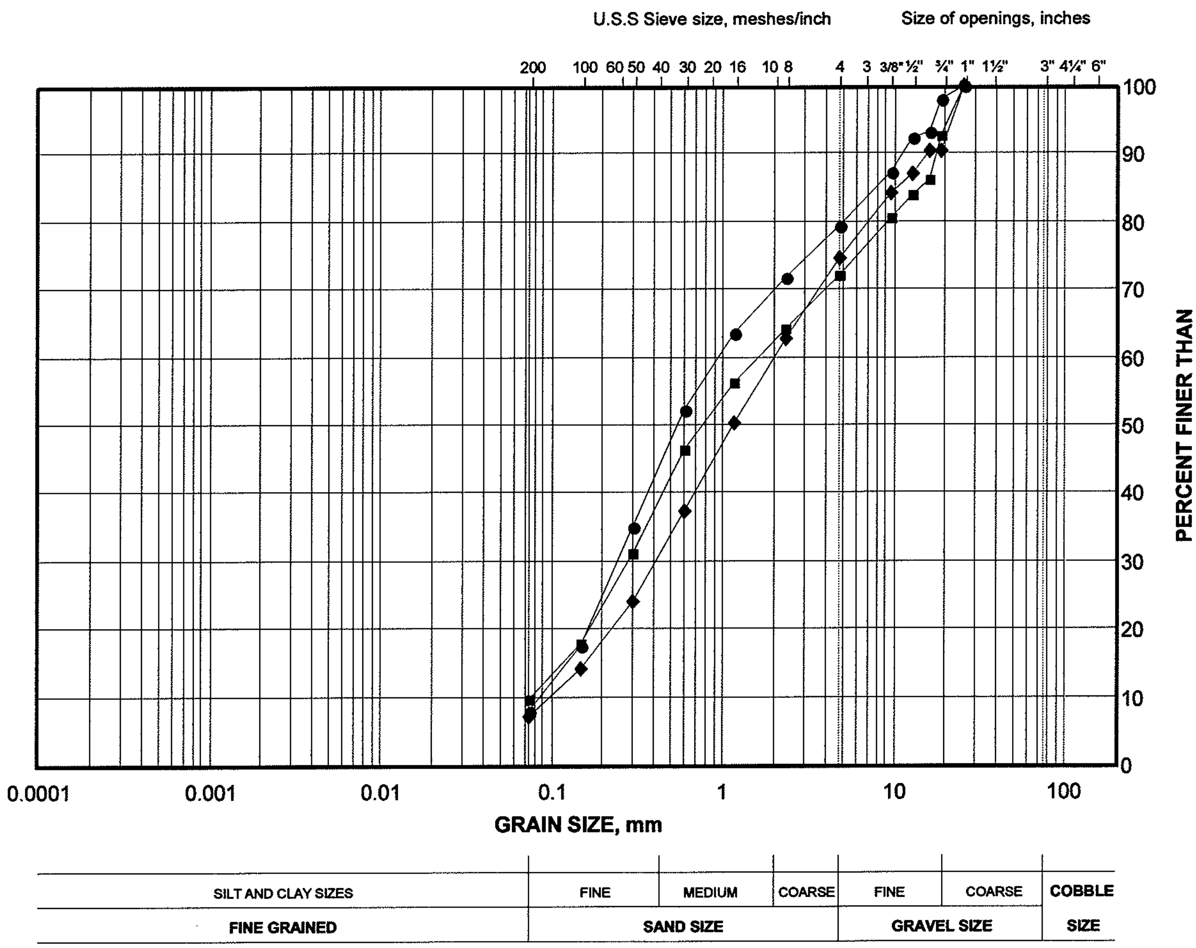
Golder Associates

Date: 04-Mar-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Silty Sand Fill

FIGURE 1C



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-21	2	1.1
■	07-20	2	1.2
◆	07-16	3	1.8

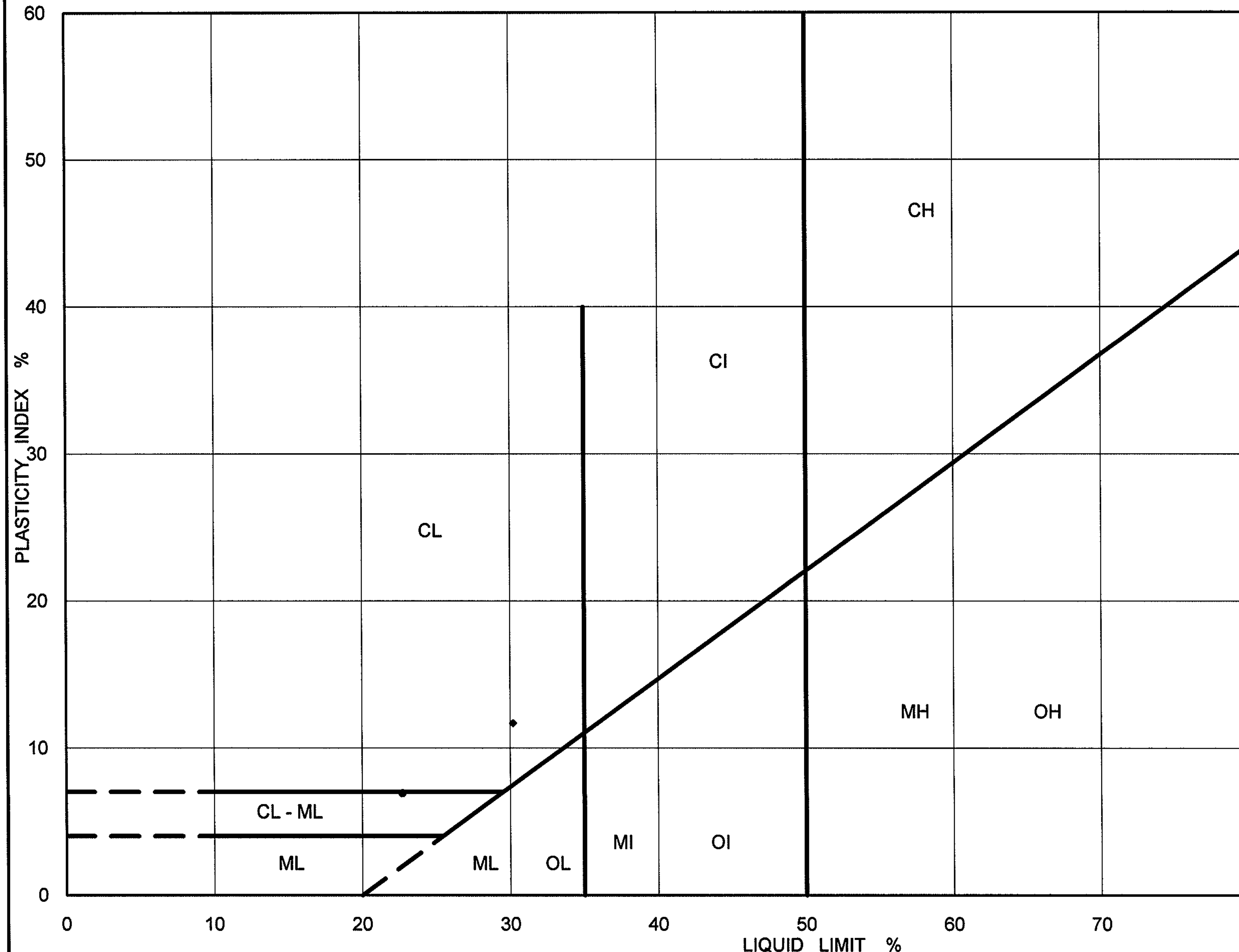
Project Number: 07-1111-0044

Checked By: [signature]

Golder Associates

Date: 04-Mar-09

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
07-9	13B	●
07-15	11B	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Clay

Figure No. 2

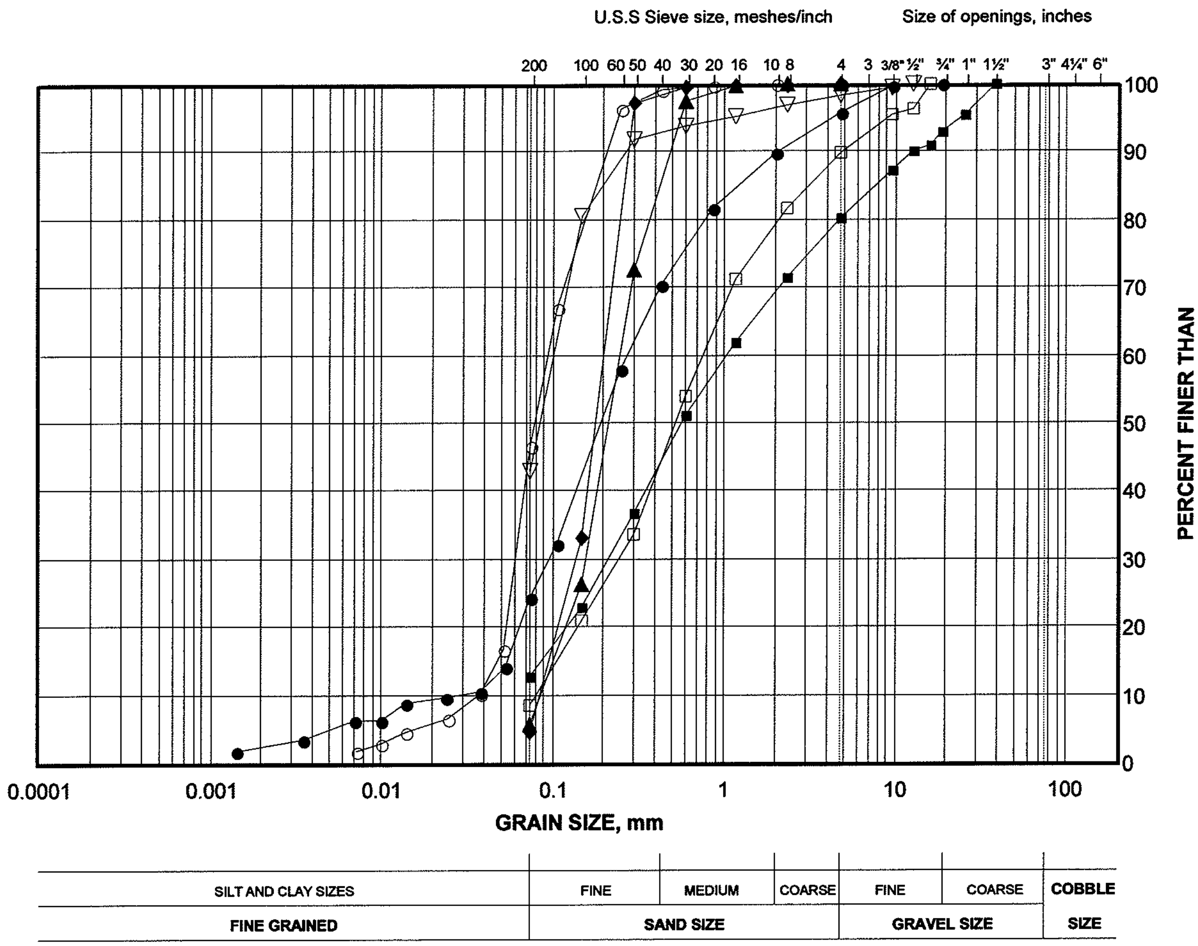
Project No. 07-1111-0044

Checked By:

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Sand and Silt

FIGURE 3A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-7	15	8.8
■	07-7	17	10.1
◆	07-2	5	3.4
▲	07-1	5	3.4
▽	07-3	8	5.6
○	07-1	8	5.6
□	07-2	9	6.4

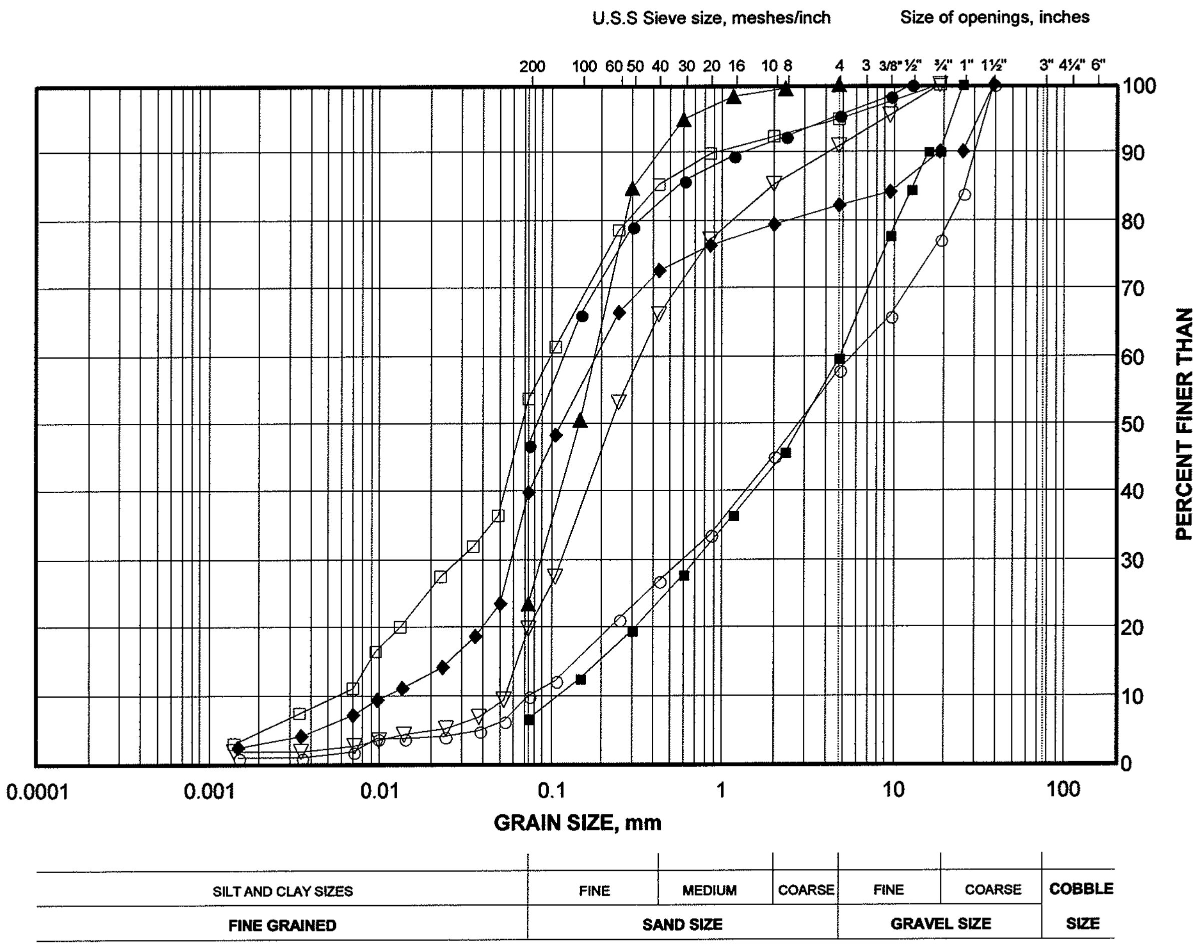
Project Number: 07-1111-0044

Checked By: [signature]

Golder Associates

Date: 04-Mar-09

GRAIN SIZE DISTRIBUTION TEST RESULTS Sand and Gravel to Sand and Silt	FIGURE 3B



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-14	10	7.9
■	07-8	10	7.0
◆	07-14	11	9.4
▲	07-12	12	9.4
▽	07-10	12	9.5
○	07-9	16	11.7
□	07-14	9	7.2

Project Number: 07-1111-0044

Checked By: [signature]

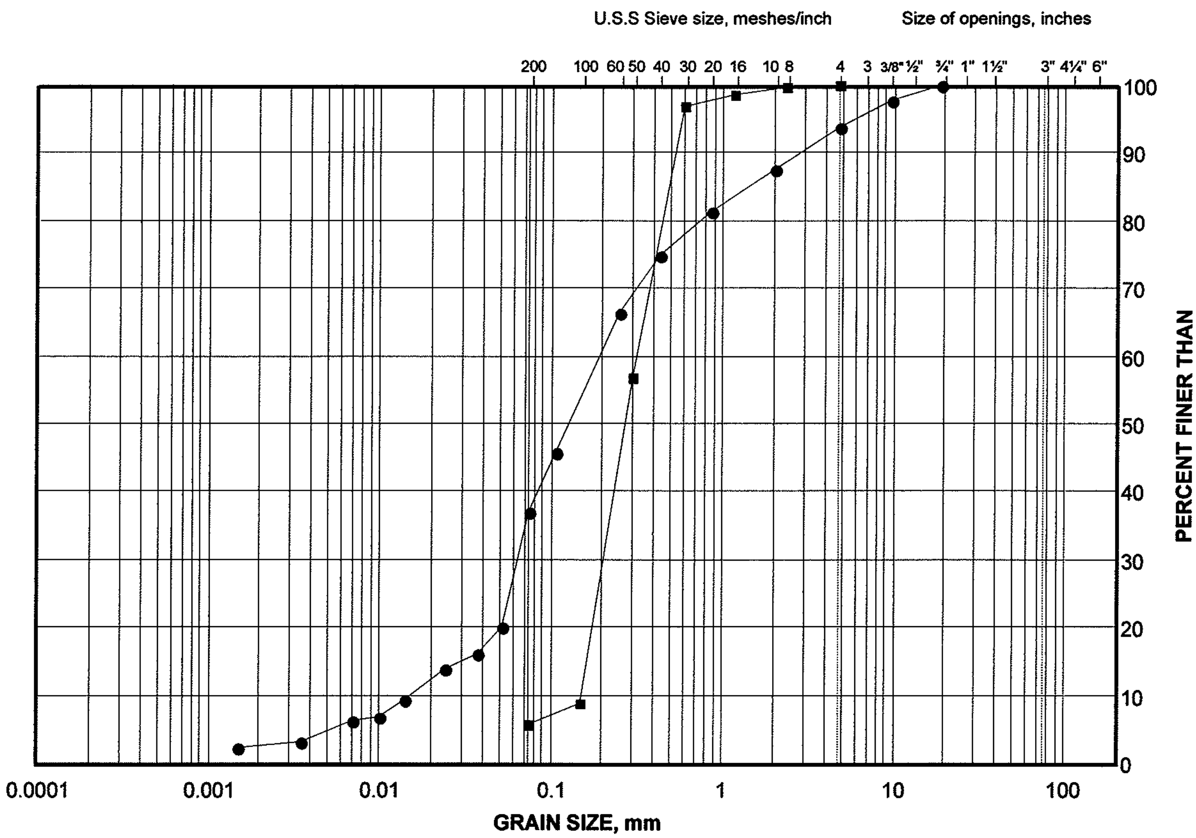
Golder Associates

Date: 04-Mar-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Sand and Silt

FIGURE 3C



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-17	5	4.1
■	07-22	7	4.9

Project Number: 07-1111-0044

Checked By: [signature]

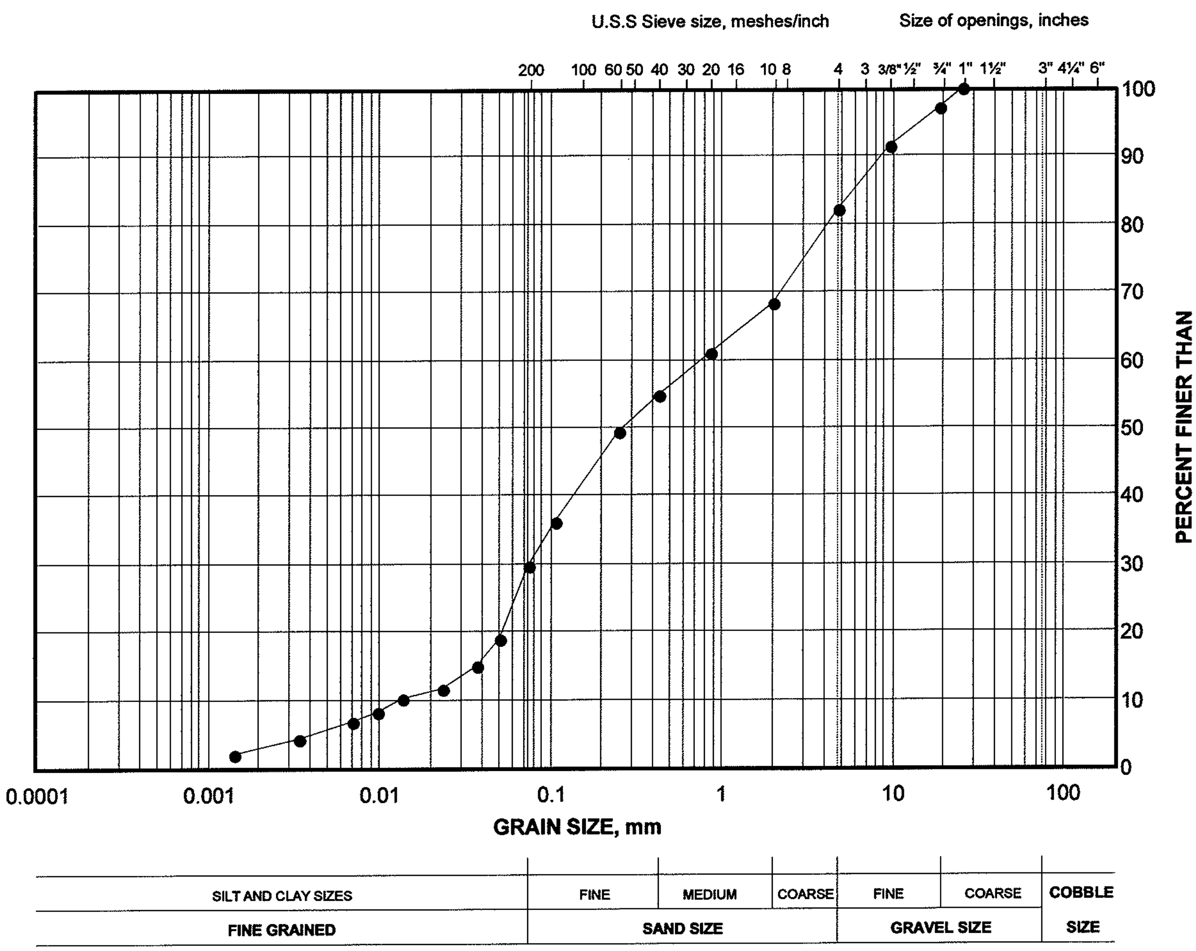
Golder Associates

Date: 04-Mar-09

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Sand Till

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	07-9	18	13.4

Project Number: 07-1111-0044

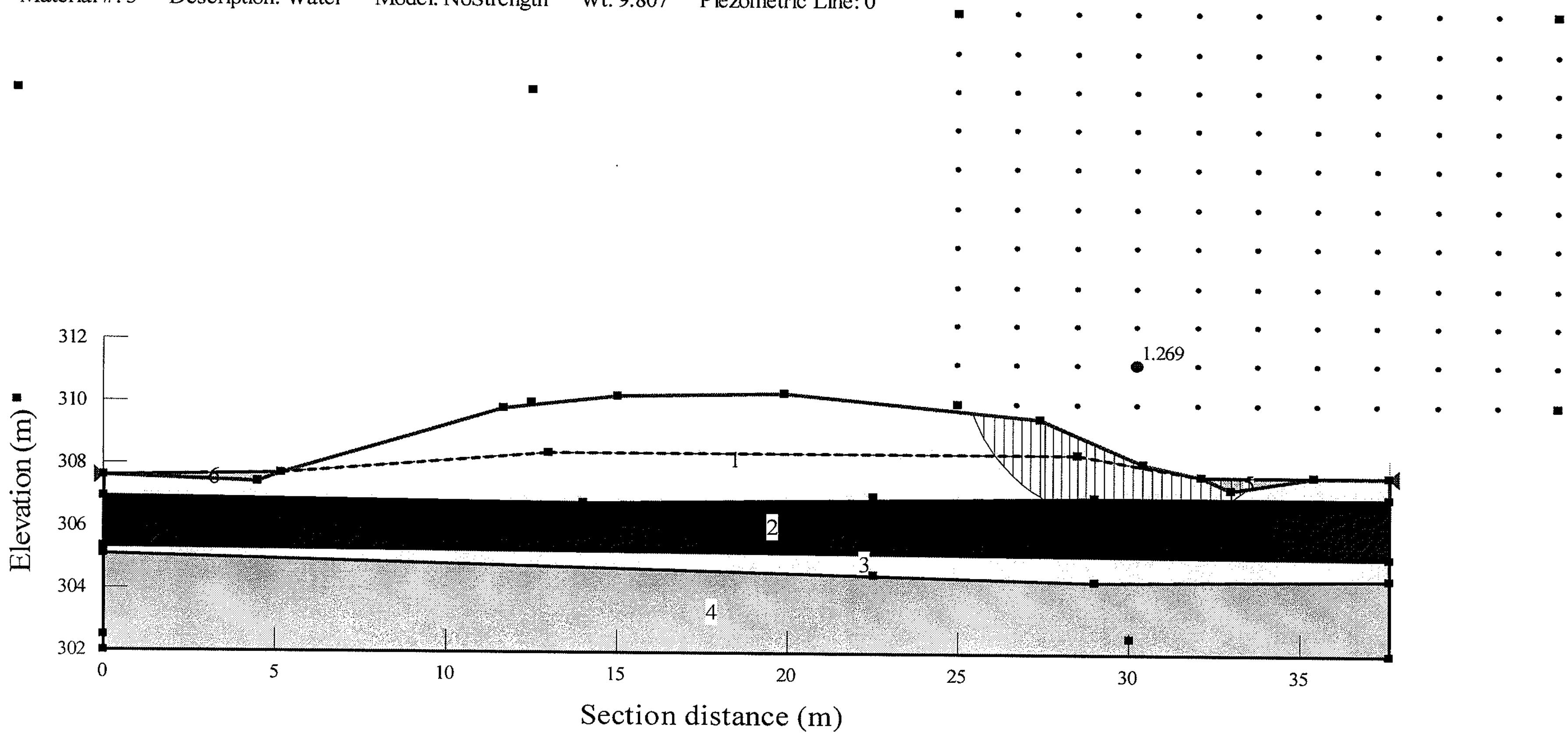
Checked By: [signature]

Golder Associates

Date: 04-Mar-09

STATIC GLOBAL STABILITY EXISTING EAST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT CULVERT AT STATION 21+369	FIGURE 5

Material #: 1 Description: Sandy Fill Model: MohrCoulomb Wt: 20 Cohesion: 0 Phi: 29 Piezometric Line: 1
Material #: 2 Description: Peat Model: MohrCoulomb Wt: 11.8 Cohesion: 0 Phi: 27 Piezometric Line: 1
Material #: 3 Description: Compact Sand to silty Sand Model: MohrCoulomb Wt: 19 Cohesion: 32 Phi: 0 Piezometric Line: 1
Material #: 4 Description: Bedrock Model: Bedrock Piezometric Line: 1
Material #: 5 Description: Water Model: NoStrength Wt: 9.807 Piezometric Line: 0



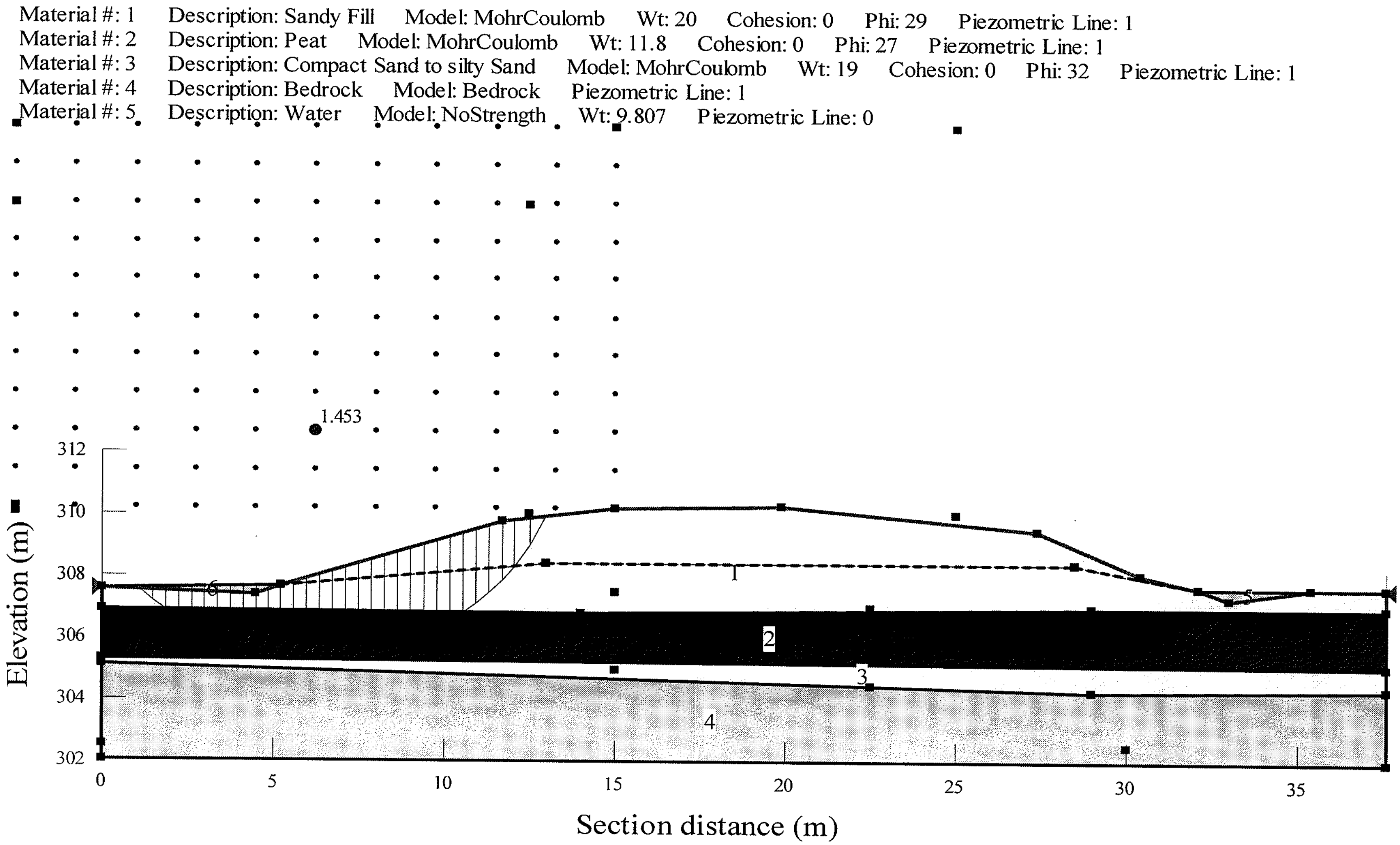
Date: March 2009
Project: 07-1111-0044-2

Golder Associates

Drawn: SH
Checked: LCC

STATIC GLOBAL STABILITY
EXISTING WEST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT
CULVERT AT STATION 21+369

FIGURE 6

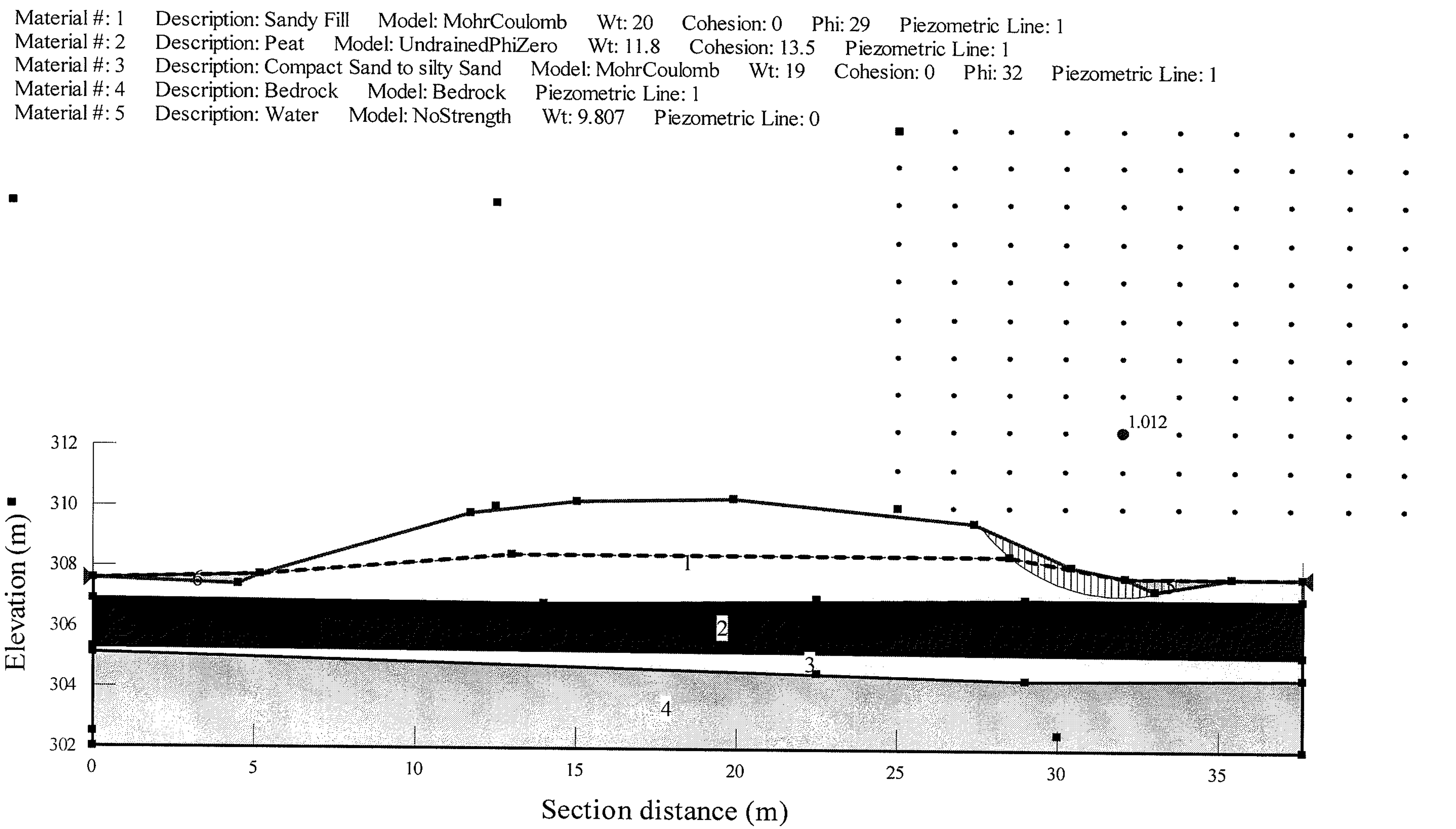


Date: March 2009
Project: 07-1111-0044-2

Golder Associates

Drawn: SH
Checked: LCC

SEISMIC GLOBAL STABILITY EXISTING EAST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT CULVERT AT STATION 21+369	FIGURE 7



Date: March 2009
Project: 07-1111-0044-2

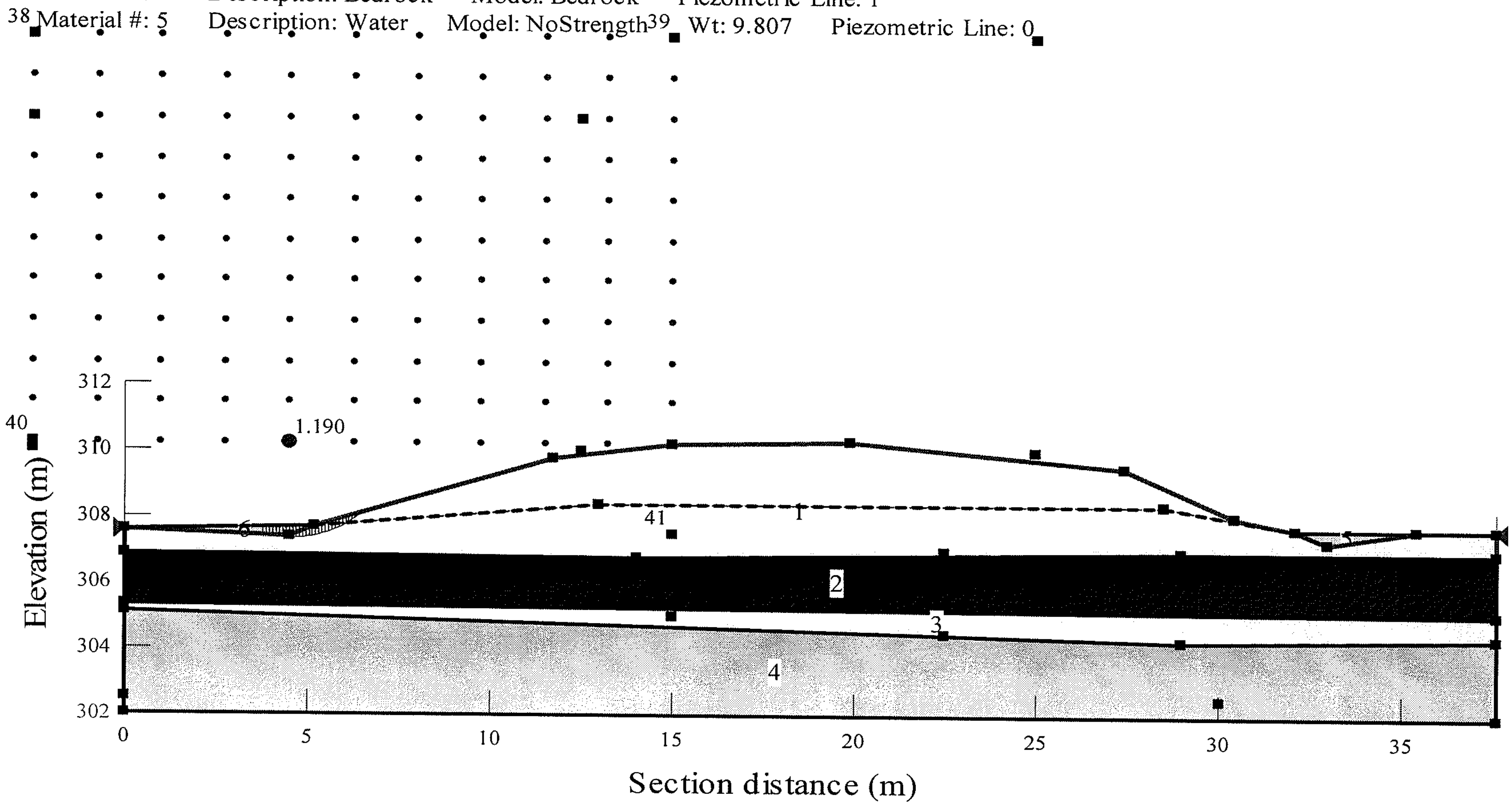
Golder Associates

Drawn: SH
Checked: LCC

SEISMIC GLOBAL STABILITY
EXISTING WEST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT
CULVERT AT STATION 21+369

FIGURE 8

Material #: 1 Description: Sandy Fill Model: MohrCoulomb Wt: 20 Cohesion: 0 Phi: 29 Piezometric Line: 1
Material #: 2 Description: Peat Model: UndrainedPhiZero Wt: 11.8 Cohesion: 13.5 Piezometric Line: 1
Material #: 3 Description: Compact Sand to silty Sand Model: MohrCoulomb Wt: 19 Cohesion: 0 Phi: 32 Piezometric Line: 1
Material #: 4 Description: Bedrock Model: Bedrock Piezometric Line: 1
Material #: 5 Description: Water Model: NoStrength Wt: 9.807 Piezometric Line: 0



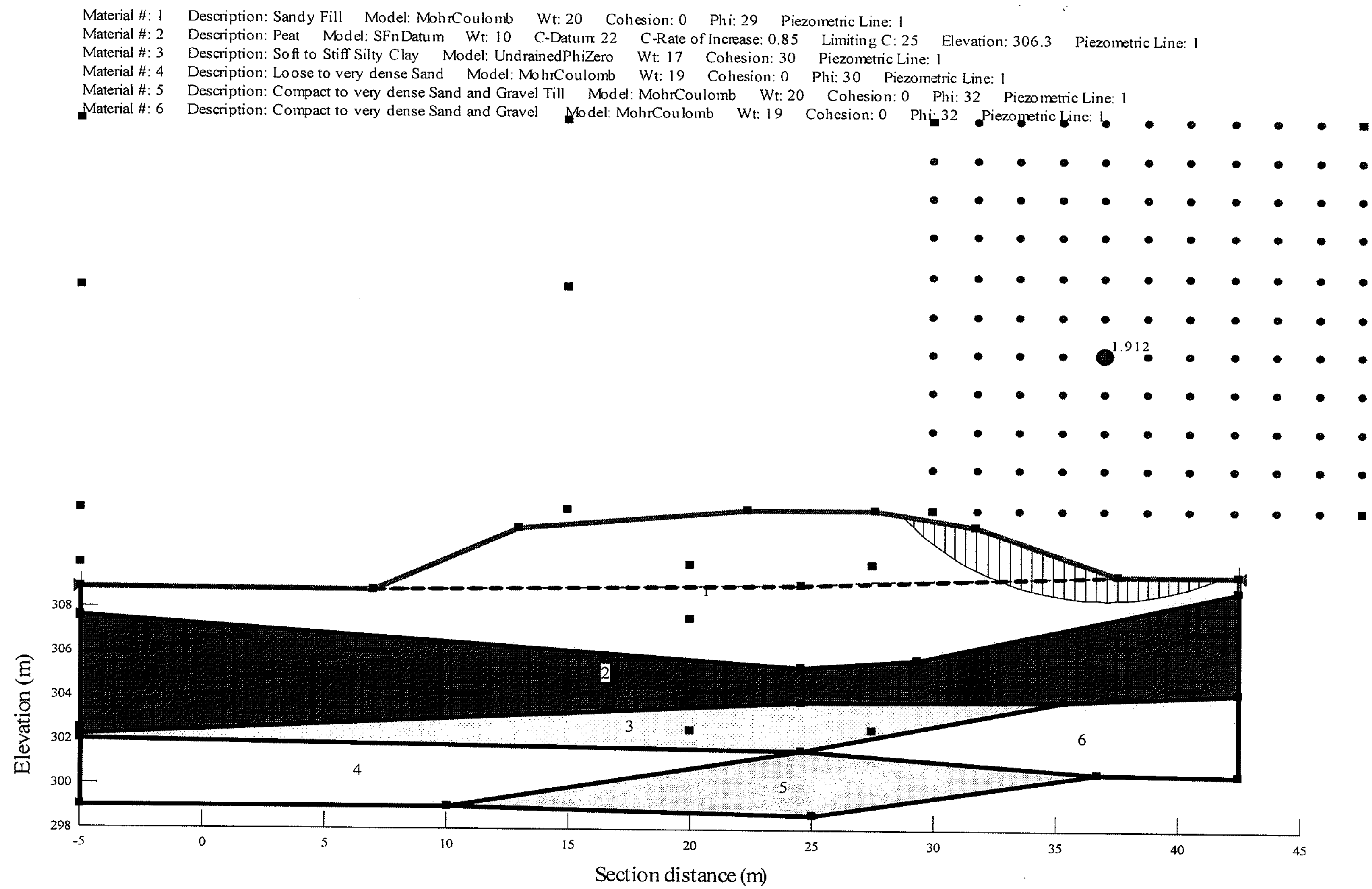
Date: March 2009
Project: 07-1111-0044-2

Golder Associates

Drawn: SH
Checked: LCC

STATIC GLOBAL STABILITY
EXISTING EAST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT
CULVERT AT STATION 23+418

FIGURE 9



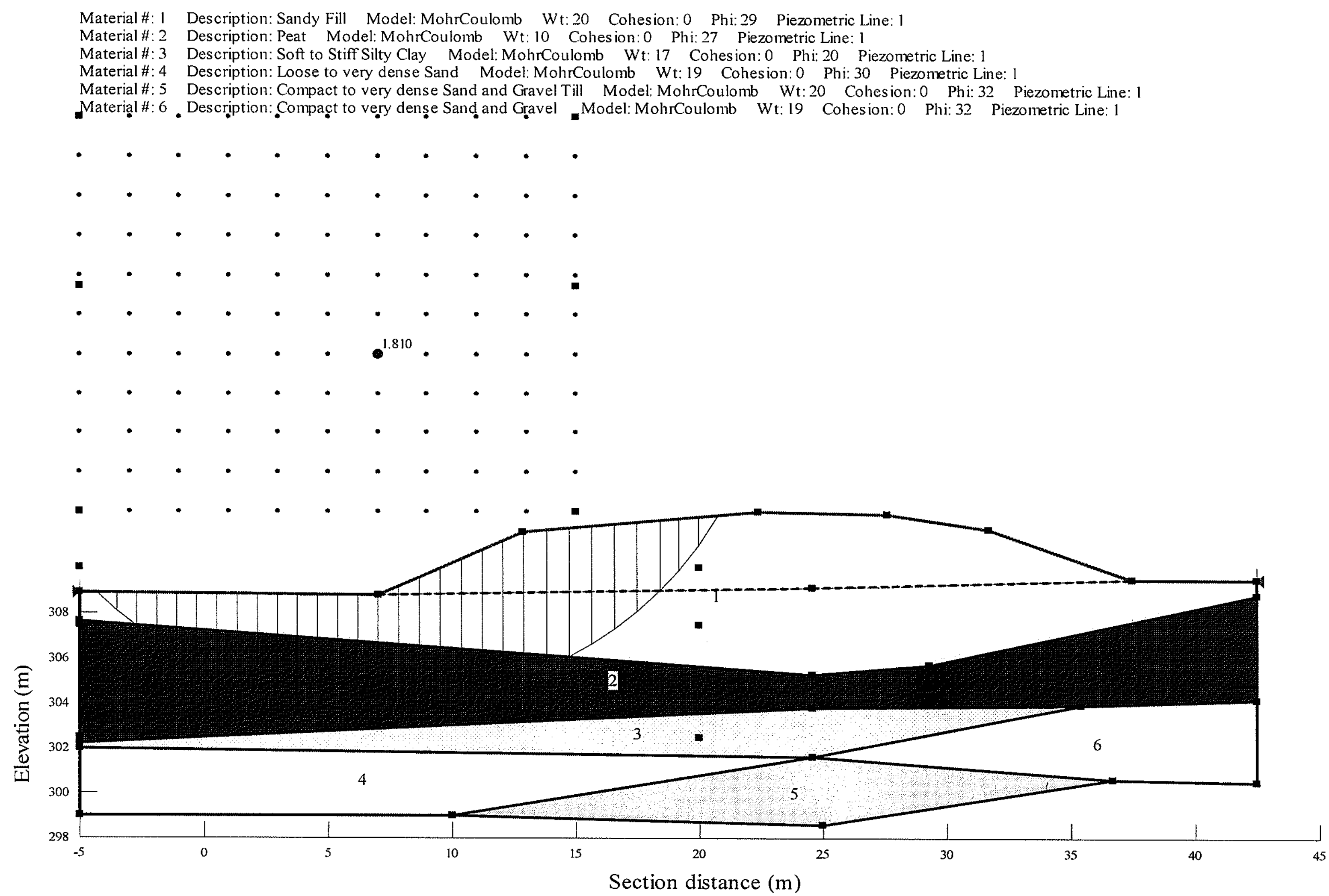
Date: March 2009
Project: 07-1111-0044-2

Golder Associates

Drawn: SH
Checked: LCC

STATIC GLOBAL STABILITY
EXISTING WEST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT
CULVERT AT STATION 23+418

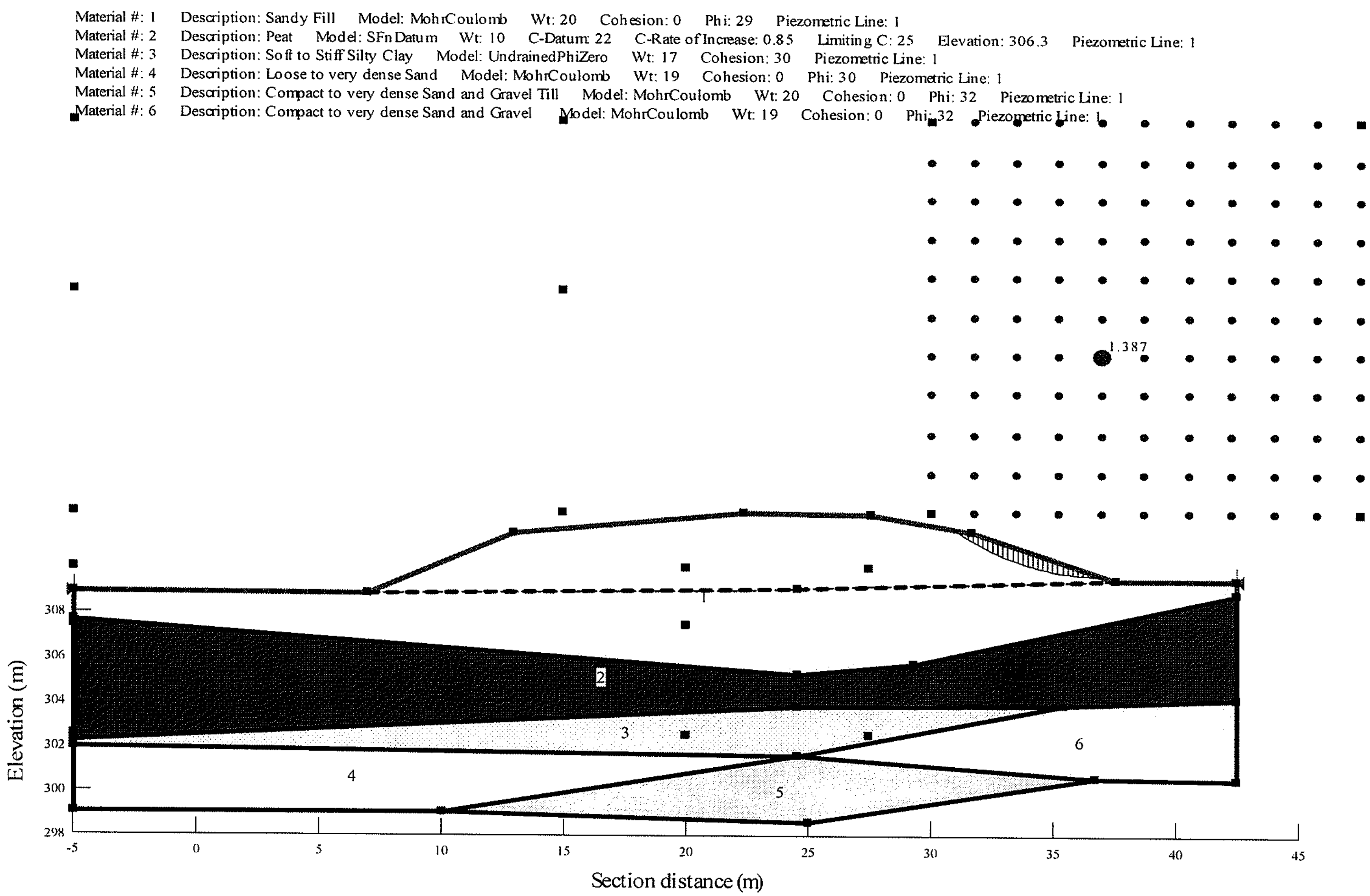
FIGURE 10



Date:	March 2009		Drawn:	SH
Project:	07-1111-0044-2	**Golder Associates**	Checked:	LCC

SEISMIC GLOBAL STABILITY
EXISTING EAST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT
CULVERT AT STATION 23+418

FIGURE 11



Date: March 2009
Project: 07-1111-0044-2

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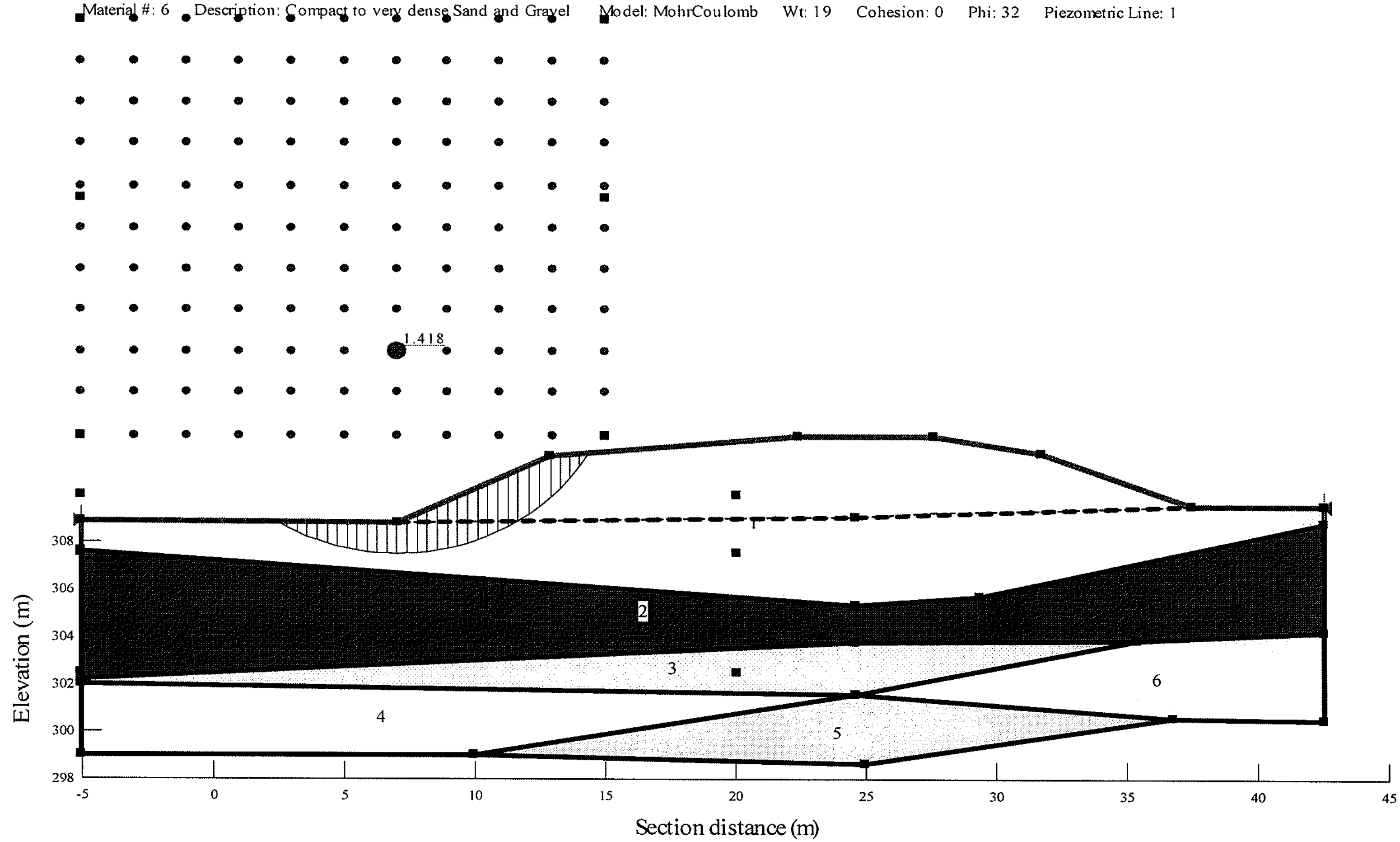
SEISMIC GLOBAL STABILITY
EXISTING WEST SIDE SLOPE OF HIGHWAY 62 EMBANKMENT
CULVERT AT STATION 23+418

FIGURE 12

Material #: 1 Description: Sandy Fill Model: MohrCoulomb Wt: 20 Cohesion: 0 Phi: 29 Piezometric Line: 1
Material #: 2 Description: Peat Model: MohrCoulomb Wt: 10 Cohesion: 0 Phi: 27 Piezometric Line: 1
Material #: 3 Description: Soft to Stiff Silty Clay Model: MohrCoulomb Wt: 17 Cohesion: 0 Phi: 20 Piezometric Line: 1
Material #: 4 Description: Loose to very dense Sand Model: MohrCoulomb Wt: 19 Cohesion: 0 Phi: 30 Piezometric Line: 1
Material #: 5 Description: Compact to very dense Sand and Gravel Till Model: MohrCoulomb Wt: 20 Cohesion: 0 Phi: 32 Piezometric Line: 1
Material #: 6 Description: Compact to very dense Sand and Gravel Model: MohrCoulomb Wt: 19 Cohesion: 0 Phi: 32 Piezometric Line: 1



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APPENDIX A

ENGINEERING PROPERTIES OF PEAT

A ENGINEERING PROPERTIES OF PEAT

Peat can be grouped into two categories according to the deposit structure: amorphous peat, and fibrous peat. The structural arrangement or texture of peat has a significant influence on its engineering properties. Amorphous peat, which contains particles of colloidal size with pore water absorbed around the particle surface, exhibits behaviour similar to clay soils. Fibrous peat has an open structure, with interstices filled with a secondary structural arrangement of fine fibrous material (Dhowian and Edil, 1980); due to the irregular shape of the particles, fibrous peat is generally quite permeable, resulting in short periods for completion of primary consolidation settlement.

A.1 Shear Strength

The undrained shear strength of peat can be estimated using a ratio of the normalized strength (i.e., ratio of undrained shear strength to effective stress) of the soil. Edil and Wang (2000) indicated no direct dependency of normalized undrained shear strength on organic content of the peat and recommended a range of 0.5 to 0.7 of the normalized strength ratio, with an average value of 0.59. The shear strength of the peat is normally low; however, the shear strength can increase considerably upon completion of primary consolidation under applied loading. Edil and Dhowian (1981) reported a drained angle of friction of 50 degrees for amorphous peat and a range of drained angles of friction from 53 to 57 degrees for fibrous peat; Landva (1983) indicated a lower range from 27 to 32 degrees of drained friction angles for the peat under normal pressures ranging from 3 kPa to 50 kPa.

A.2 Consolidation Parameters

MacFarlane et al. (1969) developed empirical correlations for the measured water content and organic content of peat with physical and consolidation properties for the peat, such as bulk density, void ratio, compression index (C_c) and coefficient of compressibility (a_v). Various ranges for the coefficient of consolidation (C_v) of the peat with different ranges of normal effective stress are indicated in the literature. For this project, coefficients of consolidation for the peat have been estimated based on monitored settlement data presented by Lea and Brawner (1963) and Samson and Rochelle (1972) for highway embankment construction in areas with similar subsurface conditions to these culvert sites.

At completion of the primary consolidation settlement, all excess pore pressure has dissipated and the soil begins secondary (creep) compression due to restructuring of soil particles; secondary (creep) compression continues at a much slower rate than primary consolidation. The amount of secondary compression is proportional to the secondary compression index ($C\alpha$) and preconstruction void ratio and thickness of the soils.

APPENDIX B

GLOBAL STABILITY OF
EXISTING HIGHWAY 62 EMBANKMENT
AT CULVERT LOCATIONS

B Stability of Existing Highway 62 Embankment at Culvert Locations

B.1 General

B.1.1 Static Global Stability

Static global slope stability analyses have been carried out for the existing embankment configurations at the culvert sites, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., to assess the stability of the existing embankments at the culvert sites.

For all static stability analyses, the factor of safety of numerous potential failure surfaces was computed to establish the minimum factor of safety of the existing embankment configuration. A target factor of safety of 1.3 against deep-seated, global failure has used to assess the existing embankment slope configurations under static conditions.

B.1.2 Stability Under Seismic Loading

The potential instability under seismic (earthquake) loading has also been assessed, with a minimum factor of safety of 1.0 required for the existing embankment configuration. These analyses have been carried out using a simple "pseudo-static" model, in which a horizontal force is applied to the failure mass. This horizontal force is proportional to the weight of the failure mass and has been determined using a "seismic coefficient" which is a simplified representation of the complex dynamic forces acting within the slope during an earthquake event.

Based on experience from other projects in which peat layers greater than 3 m in thickness were present overlying mineral soil deposits, the site-specific peak accelerations of ground motion and seismic response spectra tend to decrease significantly as the seismic waves propagated through the peat layer(s); in these cases, the peat layers tended to dampen the short-period response during seismic shaking. A zonal acceleration ratio of 0.1 has been used in the seismic stability analyses, as per the *Canadian Highway Bridge Design Code (CHBDC)* for the Bancroft area. For the "pseudo-static" slope stability analyses presented herein, the seismic coefficient has been taken as 50 per cent of the zonal acceleration ratio as described in *CHBDC* and its *Commentary*.

B.1.3 Engineering Properties of Peat and Mineral Soils

Effective stress parameters were employed in the static stability analyses for the existing embankment configurations, assuming long-term drained conditions for the soils. Due to the rapid nature of seismic loading, the clay and peat soils are expected to behave in an undrained manner during seismic events, and undrained shear strength parameters for these types of soils have been used in the seismic analyses; drained conditions and parameters have been used for free-draining soils in the seismic analyses.

For the analysis of the peat deposits at the culvert sites, the undrained shear strengths used in the analyses have been estimated to be 50 per cent of the present effective stress beneath the toes of the existing embankment, and the drained friction angle has been assumed to be 27 degrees (i.e., at the lower bound of the empirical values as discussed in Appendix A – Engineering Properties of Peat).

The effective stress parameters (effective friction angle and cohesion) for mineral soils were estimated from empirical correlations using the results of Standard Penetration Tests and Atterberg limits, in conjunction with engineering judgement from experience in similar soil conditions.

B.2 Embankment at Station 21+369

The parameters used for the global stability analysis of the existing 3 m high embankment at Station 21+369 are summarized in the following table:

SoilType	*Bulk Unit Weight*	*Effective Angle of Friction*	*Undrained Shear Strength*
Existing embankment fill	20 kN/m^3	29 º	-
Peat	11.8 kN/m^3	27 º	13.5 kPa
Compact sand to silty sand	19 kN/m^3	32 º	-

The results of the static stability analyses using these parameters indicate that the existing embankment configuration (west side oriented at 3H:1V, and east side oriented at 2.3H:1V to 3.3H:1V) has factors of safety of approximately 1.2 to 1.4 against shallow slope instability. For “global” failure surfaces passing through the deeper portions of the fibrous peat, the factor of safety against slope instability increases to at least 1.3 (see Figure 5 and 6). Therefore, the existing embankment configuration at the culvert site at Station 21+369 is considered to satisfy the minimum factor of safety for static global stability, provided that no grade raise or embankment widening is carried out as part of the Highway 62 rehabilitation works.

The “pseudo-static” seismic analyses demonstrate factors of safety against shallow-seated slope instability are about 1.0 at the east side slope and greater than 1.0 at the west side slope (see Figures 7 and 8). Therefore, the existing embankment configuration at the culvert site at Station 21+369 is considered to satisfy the minimum factor of safety requirements for stability during seismic events.

B.3 Embankment at Station 23+418

The parameters used for the analysis of the existing 3.7 m high embankment at Station 23+418, with the “worst-case” soil conditions, are summarized in the following table:

Soil Type	*Bulk Unit Weight*	*Effective Angle of Friction*	*Undrained Shear Strength*
Existing embankment fill	20 kN/m^3	29 º	-
Peat	10 kN/m^3	27 º	22 to 25 kPa
Soft to stiff clayey silt to clay	17 kN/m^3	20 º	30 kPa
Loose to very dense sand to silty sand	19 kN/m^3	30 º	-
Compact to very dense sand and gravel	20 kN/m^3	32 º	-
Very dense silty sand till	19 kN/m^3	32 º	-

The results of the static slope stability analyses using these parameters indicate that the existing embankment configuration (west side slope oriented at 2H:1V, and east side slope oriented at 2.5H:1V) has a factor of safety of approximately 1.6 against shallow-seated slope instability for a failure surface passing through the sand and gravel to sand fill. For failure surfaces passing through the deeper portions of the fibrous peat deposit, the factor of safety against static slope instability increases to greater than 1.8 (see Figures 9 and 10). Therefore, the existing embankment configuration at the culvert site at Station 23+418 is considered to satisfy the minimum factor of safety for static global stability, provided that no grade raise or embankment widening is carried out as part of the Highway 62 rehabilitation works.

The "pseudo-static" seismic analyses demonstrate factors of safety against shallow-seated slope instability are greater than 1.3 at both the west and east side slopes (see Figures 11 and 12). Therefore, the existing embankment configuration at the culvert site at Station 23+418 is considered to satisfy the minimum factor of safety requirements for stability during seismic events.

B.4 Embankment at Remaining Culvert Sites

The existing embankments at the remaining culvert sites (at Stations 24+124/24+126, 24+320/24+322, 25+057, 25+529 and 17+379) are less than 2 m in height, with side slopes oriented at 2H:1V or flatter. There was no observed evidence of instability of the existing embankment at these sites at the time of the field investigation, and generalized slope stability analyses confirm that the factor of safety against static global instability is greater than 1.3 for both the west and east embankment side slopes at these five culvert sites.

APPENDIX C

ESTIMATE OF SETTLEMENT
UNDER EXISTING HIGHWAY 62 EMBANKMENT
AT CULVERT LOCATIONS

C Estimate of Settlement Under Existing Embankment at Culvert Locations

Settlement analyses were carried out to estimate the total magnitude and duration of the post-construction settlement of the founding soils below the existing embankments at the seven culvert sites, based on the estimated empirical engineering properties of the peat as discussed in Appendix A, and the following empirical correlations of consolidation properties for the relatively thin clayey silt to clay deposits (where present):

$$\sigma_p' = s_u / 0.27$$
$$C_c = 0.009\,(w_L - 10)$$
$$C_c = 0.5 \text{ x } GS\,(PI / 100)$$
$$C_c = 0.75\,(e_o - 0.5)$$
$$C_r = C_c / 10$$

where σ_p' is the preconsolidation pressure (kPa);
C_c is the compression index;
C_r is the recompression index;
s_u is the undrained shear strength (kPa);
w_L is the plastic limit (%);
PI is the plasticity index (%); and
e_o is the preconstruction void ratio of the clayey silt to clay deposits.

The coefficients of consolidation (C_v) of the clayey silt to clay soils were estimated based on the empirical correlation with the plastic limit of the deposit (NAVFAC DM 7.1, 1982). The following table summarizes the estimated physical and consolidation parameters of the site soils used in the analysis of the magnitude of settlement that occurred following the original embankment construction:

Soil Unit	***Bulk Unit Weight (kN/m^3)***	***e_o***	***C_r***	***C_c***	***Secondary Compression Index, Cα***	***C_v (m^2/year)***
Embankment Fill	20	–	–	–	–	–
Peat	10 to 12	4 to 12	0.08	1.8 to 5.0	0.11 to 0.30	30
Soft to Stiff Clayey Silt to Clay	17.5	1.05 to 4.15	0.04	0.33 to 1.84	0.013 to 0.074	0.5 to 6.3

C.1 Primary Consolidation Settlement

It is estimated that the preconsolidation pressure (P_c') of the peat and the soft to stiff clayey silt to clay deposits (where present) was exceeded after the original construction of the Highway 62 embankment at the seven culvert sites. The following table summarizes the estimated duration that was required for completion of primary consolidation settlement, together with the estimated magnitude of primary consolidation settlement that occurred under the existing embankment loading at the seven culvert sites:

Culvert Station	*Peat*		*Clayey Silt to Clay*	
	Duration	*Magnitude*	*Duration*	*Magnitude*
21+369	2 months	1.1 m to 1.2 m	N/A	N/A
23+418	3 to 12 months	1.2 m to 5.0 m	2 to 30 months	25-150 mm
24+124/24+126	3 to 4 months	3.1 m to 4.4 m	2 to 6 months	30-75 mm
24+320/24+322	3 to 4 months	3.4 m to 4.9 m	8 months	75-100 mm
25+057	2 to 3 months	1.2 m to 2.1 m	N/A	N/A
25+529	1 month	0.4 m to 1.7 m	N/A	N/A
17+379	1 month	0.3 m	N/A	N/A

N/A – Not applicable as clayey silt to clay deposits not present at culvert site.

According to available information, the existing culverts and Highway 62 embankment were probably constructed in 1965. Based on this assumed construction date, the primary consolidation settlement in the peat would have been completed in 1966, and the primary consolidation settlement in any clayey silt to clay soils would have been completed by 1968. The estimated magnitudes of primary consolidation settlement of the peat are corroborated by the peat surface elevations below the highway embankment and shoulders, as shown on the subsurface profiles on Drawings 1 to 5.

C.2 Secondary Creep Settlement

Although the primary consolidation settlement for the existing embankment configuration at the seven culvert sites has been completed, secondary "creep" settlements are continuing within the peat and some of the clayey silt to clay layers (where present).

The secondary settlement of soil can be estimated using the following formula:

$$S_{secondary} = \frac{C_c}{1+e_0}\frac{C_\alpha}{C_c}L_0 \log\frac{t}{t_p}$$

where

C_c = compression index of soil;
C_α = secondary compression index of soil;
e_o = preconstruction void ratio under original effective stress;
L_o = preconstruction thickness of compressible layer with void ratio e_o (m);
t = time after the end of construction; and
t_p = duration of primary consolidation.

The magnitude and duration of the continuing secondary creep settlement is difficult to predict precisely due to the skeleton structures and organic nature of the peat deposits. Mesri et al. (1997) indicated ratios of $C\alpha/Cc$ equal to 0.06 for peat and 0.04 for silty clay soils. Since the initial conditions, such as void ratio and thickness, of the peat and soils are not known with certainty, it has been assumed that the original thickness of the peat and cohesive soils is the sum of the current investigated thickness in each borehole plus the estimated consolidation settlement and partial secondary settlement of soils. The preconstruction void ratios of the soils were estimated according to the present physical properties of soils. Based on these assumptions, it has been estimated that the secondary creep settlement from the end of the primary consolidation settlement in 1968 to present (2009) would be on the order of 350 mm or less for the seven culvert sites.

It is understood that the design of new culverts is to take into account a 75-year service life. Assuming that no widening or grade raise is carried out as part of this rehabilitation or future highway works at the seven culvert sites, the estimated secondary creep settlement over the next 75 years is summarized in the following table:

Culvert Station	*Magnitude of Creep Settlement at Embankment Toe*	*Magnitude of Creep Settlement Under Embankment CL*	*Differential Creep Settlement Between Toe and CL*
21+369	30 mm	N/A	N/A
23+418	50 to 65 mm	70 mm	5 to 20 mm
24+124/24+126	60 to 70 mm	75 mm	5 to 15 mm
24+320/24+322	60 to 70 mm	80 mm	10 to 20 mm
25+057	30 to 40 mm	45 mm	5 to 15 mm
25+529	10 to 30 mm	N/A	N/A
17+379	0 to 5 mm	5 mm	0 to 5 mm

N/A – Peat was not encountered at these locations due to refusal at base of fill.

As shown in the above table, the ongoing “creep” settlement of the peat is expected to be on the order of 25 mm to 75 mm for the six culvert sites in Tudor and Cashel Township, and less than 25 mm at the northernmost culvert site in Limerick Township, over the next 75 years.

APPENDIX D

NON-STANDARD SPECIAL PROVISIONS

GROUNDWATER CONTROL – Item No.

Non-Standard Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of temporary dewatering systems to facilitate the installation of new transverse culverts located at Stations 21+369, 24+124/24+126, 24+320/24+322, and 25+057.

Excavations for culvert removal and replacement work will extend through the existing Highway 62 embankment fill, which consists of very loose to very dense sand and gravel to silty sand to crushed rock fill, and which is water bearing. Cohesionless soils submerged below the groundwater level will be subjected to conditions of unbalanced head and can boil, cave in or slough during temporary excavation work.

REFERENCES

OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS 518	Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering systems shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole logs included in the contract documents as a guide in determining requirements and in accordance with the conditions noted on the drawings.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation and maintenance of an adequate dewatering system to lower the groundwater level to at least 0.3 m below the proposed invert level for the new (replacement) culverts. The dewatering system shall allow for installation of the new (replacement) culverts without disturbance to the new culvert subgrade and backfill.

Water pumped from trenches shall be redirected into the watercourse downstream of the work area in a manner that is not injurious to public health or safety, to property, to the environment or to any part of the work already completed or under construction.

Operation

A continuous dewatering operation shall be provided to facilitate the installation of the new culvert at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their preconstruction conditions, unless specified otherwise.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

PROTECTION SYSTEMS - Item No.

Special Provision

The existing 1.8 m diameter corrugated steel pipe (CSP) culvert at Station 24+320/24+322 has been abandoned in place as part of a previous contract, by pumping grout into the culvert. This culvert is to remain in place (i.e., it is not planned to remove this culvert as part of the current contract). The presence of this existing culvert must be considered in the design and installation of the protection system(s) for the new culvert construction at this site.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

REMOVAL OF PIPE CULVERTS - Item No.

Special Provision

Scope Of Work

This special provision outlines the procedure to be used for backfilling following removal of the existing pipe culverts at Stations 21+369, 24+124/24+126, 25+057, and 25+529 of Tudor and Cashel Township, and Station 17+378 of Limerick Township.

Construction

If the culvert removal works are carried out without dewatering, such that the groundwater level in the excavation for the removal has not been lowered to below the base of the excavation, then the excavation shall be backfilled subaqueously using Ontario Provincial Standard Specification (OPSS) 1010 Granular B Type II fill, up to the underside of the bedding for the replacement culvert.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION