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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN REPORT CNR BRIDGE REHABILITATION AND WIDENING HIGHWAY 401, KINGSTON, ONTARIO G.W.P. 78-99-00

Submitted to:
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Geocres Number: 31C-202

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REPORT

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Table of Contents

LIST OF TABLES

LIST OF DRAWINGS

LIST OF FIGURES

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
CNR BRIDGE REHABILITATION/WIDENING
KINGSTON, ONTARIO
G.W.P. 78-99-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation associated with the Highway 401 expansion in Kingston, Ontario. The section of Highway 401 included in this assignment (G.W.P. 78-99-00) extends from Montreal Street to about 1.8 kilometres east of the Canadian National Railway (CNR) structure.

Foundation investigation services are required for the following components:

- CNR bridge rehabilitation/widening;
- Highway 401 embankment widening – Cataraqui wetlands;
- Montreal Street Underpass replacement;
- Overhead signs (total of 2); and,
- Noise Barrier Wall.

This report addresses the CNR bridge rehabilitation/widening component (W.P. 4015-06-01), Geocres Number 31C-202.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2008. The work was carried out in accordance with Golder's Quality Control Plan dated November 2008.

2.0 SITE DESCRIPTION

The existing Highway 401 structure included in this assignment (G.W.P. 78-99-00) carries Highway 401 over the CNR line (Mile 171.10 of the Kingston Subdivision) and is located about 310 m east of the Montreal Street interchange in Kingston, Ontario.

Through this area, Highway 401 is a four lane divided highway with a rural cross-section. The highway profile grade over the CNR bridge structure varies from west to east from about elevation 87.8 to 86.3 m (i.e., grade declining eastward). The existing bridge, which was constructed in 1954, consists of a three span cast-in-place concrete girder structure on concrete abutments and pier. Information provided by MTO at the proposal stage indicated that the west abutment and west pier are founded on spread footings on bedrock, and that the east abutment and pier are founded on piles driven to bedrock. This information is consistent with information shown on Department of Highways Bridge Office drawings (dated April 1953, originally numbered D3349-1 though D3349-11) which were obtained by MRC and provided to Golder.

The CNR Kingston Subdivision crosses beneath the Highway 401 structure with top of rail at an elevation of about 78 m. The railway has two tracks at this crossing, with space for a third track on the west side.

No GEOCRES information is available for this structure.

To the west, adjacent to the bridge structure, rock outcrops exist that are up to about 9 m high relative to the existing bridge deck. To the east, the existing approach embankments are up to about 10 m high relative to the surrounding natural ground surface and have approximately 1.5 horizontal to 1 vertical (1.5H:1V) side slopes. No signs of embankment instability were observed.

The highway profile at the approaches does not seem to indicate significant differential settlement of the roadway relative to the bridge, although the maintenance history at this location is not currently known.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the proposed CNR bridge structure and approach embankment locations between June 8 and September 16, 2009, at which time ten (10) boreholes (numbered B1 to B10, inclusive) were advanced at the locations shown on Drawings 1 and 2. The borehole locations were selected as follows:

Approach Embankments:

- One borehole (numbered B5) located at the west approach, about 20 m west of the existing westbound lane abutment, extending through the overburden and then cored 3.3 m into bedrock; and,
- One borehole (numbered B10) located at the east approach embankment; about 20 m east of the existing westbound lane abutment, extending through the existing rock fill embankment plus a depth equivalent to the existing embankment height (i.e., 10 m) below the base of the rock fill.

Abutments:

- Two boreholes (numbered B1 and B6) located at the west abutment, one on each side of the existing abutment, extending through the road base and rock fills and then cored 3 m into bedrock; and,
- Two boreholes (numbered B4 and B9) located at the east abutment, one on each side of the existing abutment, extending through the embankment rock fills, native silty clay and till, and then cored 2.6 and 3.1 m into bedrock, respectively.

Piers:

- Two boreholes (numbered B2 and B7) located west of the west pier, one on each side of the existing pier, extending through the overburden and then cored 3.6 and 3.1 m into bedrock, respectively; and,
- Two boreholes (numbered B3 and B8) located at the east pier location, one on each side of the existing pier, extending through the overburden and then cored 3.7 and 2.8 m into bedrock, respectively.

The boreholes were located within 5 m of the proposed bridge foundation locations, with the exception of B7 which was put down 5.7 m west of the west pier centreline.

The three boreholes advanced for the east abutment and the east approach embankment (i.e., boreholes B4, B9, and B10) were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers on a truck-mounted drill rig (boreholes located on highway), supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths ranging from 21.8 to 25.9 m below the existing roadway surface.

The remaining boreholes were advanced using portable/manual drilling equipment supplied and operated by OGS Drilling Services of Appleton, Ontario. The boreholes were advanced to depths ranging from 5.7 to 17.9 m below the existing ground surface.

Soil samples were obtained nearly continuously during the portable drilling, and at intervals of 0.75 m to 1.5 m of depth using the truck-mounted drill rig, using a 50 mm outer diameter (O.D.) split-spoon sampler. Where possible, the split spoon was advanced in accordance with Standard Penetration Test (SPT) ASTM D1586 procedures. At boreholes B1, B5, and B6, where access constraints did not permit use of a tripod above the hole, a one-third weight hammer was used and blow counts were adjusted accordingly to correlate with SPT values. In-situ vane testing (N vane) was carried out within the cohesive deposits where possible. Relatively undisturbed, 75 mm diameter thin-walled Shelby tube (ASTM D1587) samples of the cohesive soils were retrieved using a fixed piston sampler where possible.

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of the work.

The field work was supervised throughout by members of our technical staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The soil samples were identified in the field, placed in appropriate containers, labelled, and transported to our Ottawa geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing, including grain size distribution, water content, and Atterberg limit testing. Laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

In addition, laboratory point load index and unconfined compressive strength testing was carried out on selected samples of the bedrock core, and laboratory oedometer consolidation testing (ASTM D2435) was carried out on two specimens of the Shelby tube samples of the silty clay deposit from boreholes B4 and B8 at Golder's Mississauga geotechnical laboratory.

The borehole locations and ground surface elevations were determined by Golder personnel at the site using a Trimble R8 GPS unit. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole No.	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
B1	Proposed eastbound lane, west abutment	4904268.6	307188.1	87.6
B2	Proposed eastbound lane, west pier	4904273.3	307205.4	80.4
B3	Proposed eastbound lane, east pier	4904280.6	307230.6	77.3
B4	Existing eastbound lane, east abutment	4904295.2	307258.3	86.4
B5	Proposed westbound lane, west approach embankment	4904302.4	307197.7	87.8
B6	Proposed westbound lane, west abutment	4904304.2	307204.2	87.5
B7	Proposed westbound lane, west pier	4904318.2	307227.7	79.7
B8	Proposed westbound lane, east pier	4904323.6	307254.3	78.5

Borehole No.	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
B9	Existing westbound lane, east abutment	4904324.4	307274.6	86.0
B10	Existing westbound lane, east approach embankment	4904329.3	307288.8	85.6

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the southern portion of the physiographic region known as the Napanee Plain, and just west of the Leeds Knobs and Flats, as delineated in The Physiography of Southern Ontario[1].

The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[2] indicates that the bedrock within the Napanee Plain consists of grey limestone/dolostone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams. The limestone/dolostone is underlain by arkosic sandstone of the Shadow Lake Formation.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay. Well records indicate that the average depth to bedrock within the Napanee Plain is approximately 2 m. However, in many areas bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the northern and southern portion of the Plain, and within and adjacent to river valleys throughout the Plain.

The Leeds Knobs and Flats are characterized by knobs of Precambrian rock (i.e., Limestone Plain) surrounded by clay flats (i.e., Clay Plain). The clay is grey in colour, and very weakly calcareous.

In particular, the study area lies within the western limits of the Cataraqui River. The Cataraqui River is characterized by a number of lakes joined by the river. This river flows southerly towards Kingston.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock, and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are given on the attached Record of Borehole sheets and on Figures 1 to 11.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and in-situ testing 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.

The borehole locations and ground surface elevations are shown on Drawing 1.

In summary, the subsurface conditions encountered consist of up to about 13.7 m of fill material at the boreholes located at the proposed east approach embankment and east abutment locations, and up to about 7.3 m of fill material at the other borehole locations. The fill is generally underlain by up to about 10.7 m of silty clay, with the exception of boreholes B1, B5, and B6, which are located on the western portion of the site. Up to 7.1 m of limestone rock slabs were encountered beneath the silty clay in the boreholes advanced at the proposed west pier locations. The fill and/or silty clay is generally underlain by a thin silty sand matrix till deposit. Bedrock was encountered at depths of 2.4 to 3.7 m below existing ground surface at boreholes located at the proposed west

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[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.

[2] Map 2544, Ministry of Northern Development and Mines, 1991.

approach embankment and west abutment locations (i.e., elevations of 83.8 to 85.4 m), and at depths of 11.8 to 23.3 m below existing ground surface at the other borehole locations (i.e., elevations of 63.1 to 66.3 m).

A more detailed description of the subsurface conditions encountered in the boreholes carried out at the site of the proposed bridge structure is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 to 4.

4.2.1 Pavement Structure / Embankment and Grade Fill

The pavement structure was penetrated by borehole B4 on the south shoulder of the eastbound lanes of Highway 401 and at boreholes B9 and B10 located on the north shoulder of the westbound lanes. The pavement structure ranges from approximately 0.9 to 1.5 m in thickness and is generally comprised of 0.3 to 0.4 m of asphaltic concrete (i.e., asphalt), overlying crushed stone and sand and gravel base materials.

Beneath the pavement structure at boreholes B4, B9 and B10, and at ground surface at the other borehole locations, embankment and/or grade fill was encountered. The fill was fully penetrated at each borehole location and varies in thickness and in composition throughout the site. At the east approach embankment and east abutment locations (i.e., boreholes B4, B9, and B10), the embankment fill beneath the pavement structure ranges in thickness from 9.2 to 12.2 m. At the other borehole locations, the embankment and/or grade fill ranges in thickness from 2.4 to 5.3 m. The variable fill thickness reflects, in part, the differing native ground surface elevations at the borehole locations which vary from about elevation 96.0 m at the west approach (where the road is in cut) to about elevation 75.0 m beneath the east approach embankment fills. Portions of the fill may have been placed in association with the initial construction of the CNR tracks while the remainder was likely associated with construction of the existing Highway 401.

The embankment and grade fill material generally consists of variable amounts of rock fill, sand and gravel, silty sand, and sand fill. Layers of silty clay and silt fill were encountered at boreholes B2, B3, B4, and B7. Cobbles and boulders were also inferred to be present within the fill. Diamond drilling techniques were required to penetrate the rock fill, cobbles, boulders, and/or limestone slabs in five of the boreholes advanced using portable/manual drilling equipment.

The results of grain size distribution testing carried out on samples of the embankment and grade fill are provided on Figures 1 to 3. The results have been sorted/reported according to the fill material type in accordance with the descriptions on the Record of Borehole sheets, rather than according to the specific gradation of each sample, recognizing that there are natural variations in the material from the generalized descriptions on the borehole records. The results also do not reflect the cobble, boulder, or coarse gravel contents of the material, since the samples were retrieved using a 50 mm diameter sampler.

Standard Penetration Test ‘N’ values for the embankment fill ranging from 3 to greater than 50 blows per 0.3 m of penetration indicate that the material ranges in consistency from very loose to very dense, although the soil is generally loose to compact, with the higher ‘N’ values likely reflecting the presence of cobbles and boulders, rather than the state of packing of the soil matrix.

The measured water content of samples of predominantly granular fill ranges from approximately 3 to 13 percent. The measured water content of three samples of the more silty/clayey fill from boreholes B4 and B7 ranged from about 27 to 31 percent.

4.2.2 Silty Clay to Clay

On the east part of the site, the embankment fill at boreholes B2 to B4, and B7 to B10, is underlain by a deposit of sensitive silty clay to clay.

The silty clay was fully penetrated at each borehole location and varies in thickness from about 1.0 to 10.7 m, though the deposit is thicker to the east of the CNR tracks (i.e., 7.9 to 10.7 m thick). On the west side of the CNR tracks, where the clay was encountered only at boreholes B2 and B7, the silty clay is 2.2 and 1.0 m thick, respectively.

The results of grain size distribution testing carried out on samples of the silty clay are provided on Figure 4.

The upper portion of the silty clay at boreholes put down east of the CNR tracks and the full thickness of silty clay at boreholes put down west of the CNR tracks has been weathered to a grey brown colour. Where encountered, the thickness of the weathered crust ranges from 1.0 to 6.3 m. Standard Penetration Tests carried out within the weathered silty clay gave ‘N’ values ranging from 4 to 72 blows per 0.3 m of penetration, however most readings ranged from 8 to 30 blows per 0.3 m of penetration, indicating a generally stiff to very stiff consistency.

Unweathered (i.e., grey in colour) silty clay was encountered at the boreholes east of the tracks (i.e., boreholes B3, B4, and B8 to B10, inclusive) below the upper weathered silty clay. This unweathered silty clay ranges from about 1.2 to 6.3 m in thickness (i.e., extends to elevations of between 64.2 to 66.9 m) and contains occasional silty sand and sand seams. The measured SPT “N” values within this deposit ranged between ‘weight of hammer’ and about 5 blows per 0.3 m of penetration. In situ vane testing in this material measured undrained shear strengths ranging from about 20 to greater than 80 kilopascals. These results indicate a generally soft to stiff consistency.

The results of Atterberg limit testing carried out on fourteen samples of the silty clay are shown on Figure 5 and indicate plasticity index values generally ranging from 16 to 36 percent and liquid limit values ranging from 35 to 63 percent, reflecting intermediate to high plasticity (i.e., silty clay to clay). The measured water content of the unweathered grey silty clay ranges from approximately 31 to 58 percent, which is generally close to the measured liquid limit. The measured water content of the weathered silty clay ranges from approximately 18 to 42 percent, and is generally below the measured liquid limit. In one case, the measured water content of the weathered silty clay was at the measured liquid limit.

Oedometer consolidation testing was carried out on specimens from two thin-walled Shelby tube samples of the silty clay. The results of that testing are provided on Figures 6 and 7 and are summarized in the table below. The results indicate that the silty clay at the east pier location (borehole B8, put down at the toe of the embankment) is close to normally consolidated, having an overconsolidation ratio of about 1.1, which would be expected given its loading history and location relative to the existing embankment. Results of consolidation testing of the silty clay at the east abutment (borehole B4, put down at the top of the east approach embankment) indicate that the sample is linear elastic in its response to increased loading, with a constant void ratio (e) to load (p) rate of -0.00107/kPa for loads up to 600 kPa.

Borehole/ Sample No.	Sample Depth / Elev. (m)	Unit Weight (kN/m^3)	σ_P' (kPa)	σ_{VO}' (kPa)	$\sigma_P' - \sigma_{VO}'$ (kPa)	C_c	C_r	e_o	OCR	C_v (cm^2/s)
B4 / 16 [(1)]	19.3 / 67.1	19.3	n/a	260	n/a	0.29	0.033	0.85	n/a	0.014
B8 / 12	10.6 / 67.9	18.4	130	117	13	0.47	0.053	1.07	1.1	0.0038

(1) Sample may have been disturbed.

Notes:

σ_P' - Apparent preconsolidation pressure

σ_{VO}' - Computed existing vertical effective stress

C_c - Compression index

C_r - Recompression index

e_o - Initial void ratio

OCR - Overconsolidation ratio

C_v - Coefficient of consolidation

4.2.3 Silty Sand Till

At boreholes located east of the CNR tracks (i.e., boreholes B3, B4, and B8 to B10, inclusive), the fill and/or silty clay are underlain by till. The till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand with traces of clay.

The surface of the till varies from about elevation 64.2 to 66.9 m. The till was fully penetrated at boreholes B3, B4, B8, and B9 and varied in thickness from 0.1 to 1.4 m, extending down to elevations varying from 63.1 to 65.5 m. The till at borehole B10 was not fully penetrated, but proven for a thickness of 0.8 m (i.e., extending down to elevation 63.9 m).

Results of grain size distribution testing carried out on three samples of the till (Figure 8) confirm that the till matrix generally consists of a silty sand with variable amounts of gravel and typically trace amounts of clay. These samples were retrieved using a 50 mm diameter sampler and therefore the results do not reflect the coarse gravel, cobble and boulder content of the deposit.

Standard Penetration Test ‘N’ values for this material ranging from 22 to greater than 65 blows per 0.3 m of penetration indicate a compact to very dense state of packing, although the higher ‘N’ values could reflect the presence of coarse gravel, cobbles and boulders, rather than the state of packing of the soil matrix.

The measured water content of the till ranges from approximately 11 to 12 percent.

4.2.4 Limestone Rock Slabs

Limestone rock slabs were encountered beneath weathered silty clay at the two boreholes (B2 and B7) advanced for the pier west of the CNR tracks. Diamond drilling techniques were required to penetrate the rock slabs. Numerous voids or loose soil infilled seams were encountered during the diamond drilling, as shown on the Record of Borehole sheets. Furthermore, a layer of cobbles, boulders and silty clay was encountered beneath the rock slabs at borehole B7.

The limestone rock slabs were fully penetrated at boreholes B2 and B7 and varied in thickness from 7.0 to 7.1 metres, extending down to elevation 66.2 m and 66.3 m, respectively.

4.2.5 Refusal and Bedrock

Bedrock was encountered beneath the embankment fill, silty clay, glacial till and/or limestone slabs, and cored for about 3 m depth, at boreholes B1 through B9. At borehole B10, which was advanced at the location of the east approach embankment, refusal to augering was encountered at about elevation 63.9 m. Refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

The following table summarizes the bedrock surface depths and elevations as encountered at the nine borehole locations where bedrock was cored.

Location	Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Approach – North	B5	87.8	2.4	85.4
West Abutment – South	B1	87.6	3.3	84.3
West Abutment – North	B6	87.5	3.7	83.8
West Pier – South	B2	80.4	14.3	66.2
West Pier – North	B7	79.7	13.4	66.3
East Pier – South	B3	77.3	11.8	65.5
East Pier – North	B8	78.5	13.6	64.9
East Abutment – South	B4	86.4	23.3	63.1
East Abutment – North	B9	86.0	21.5	64.5

At the location of the west abutment – north (westbound widening), available data suggests that the bedrock surface continues to drop towards the east from borehole B6.

The bedrock encountered in boreholes B1 to B7 consists of grey, greenish grey and reddish grey interbedded limestone and dolomitic limestone. The bedrock is fresh to weathered, medium strong and laminated to medium bedded. The Rock Quality Designation (RQD) values measured on recovered limestone and dolomitic limestone bedrock core samples were quite variable and ranged from 0 to 86 percent, indicating a very poor to excellent quality rock. However the RQD values were generally found to increase with depth. The discontinuities observed in the rock core are typically horizontal, associated with the bedding planes.

Laboratory point load index testing was carried out on nine selected specimens of limestone and dolomitic limestone core. Unconfined compressive strengths (UCS) interpreted from the point load index testing ranged widely from 14 to 199 MPa. Laboratory unconfined compressive strength testing carried out on an additional three samples of limestone indicate UCS values ranging from about 40 to 64 MPa. The results are summarized on Figure 9.

Beneath the limestone and/or dolomitic limestone at boreholes B3 and B4, and the bedrock encountered in boreholes B8 and B9 is grey, red, reddish grey, and greenish grey arkosic sandstone of the Shadow Lakes Formation. The sandstone was encountered at the boreholes east of the tracks, at elevations ranging from 62.8 to 64.9 m. The bedrock is fresh to slightly weathered, medium strong and fine to coarse grained. The Rock Quality Designation (RQD) values measured on recovered sandstone bedrock core samples were quite variable and ranged from about 47 to 100 percent, indicating a fair to excellent quality rock.

Laboratory point load index testing carried out on six selected specimens and laboratory unconfined compressive strength testing carried out on one selected specimen of the sandstone indicate that the unconfined compressive strengths range widely from 49 to 248 MPa. Results are summarized on Figure 10.

Precambrian bedrock was encountered beneath the sandstone at borehole B8. The red, grey and black rock is fresh and medium strong. The Rock Quality Designation (RQD) value measured on one recovered sample of Precambrian bedrock core sample was 38 percent, indicating a poor quality rock. Laboratory point load index testing carried out on one selected specimen from the Precambrian bedrock core indicates an interpreted compressive strength of about 197 MPa (see Figure 11).

4.3 Groundwater Conditions

The groundwater levels in the piezometers in boreholes B2, and B3 were measured on September 29, 2009. The observed groundwater levels are summarized in the table below:

Borehole Number	Existing Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)
B2	80.4	4.3	76.1
B3	77.3	0.9	76.4

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This report was prepared by Ms. Erin O'Neill, P.Eng., under the direction of the Project Manager, Mr. Michael Snow, P.Eng., Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
CNR BRIDGE REHABILITATION/WIDENING
KINGSTON, ONTARIO
G.W.P. 78-99-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed rehabilitation/widening of the bridge structure that carries Highway 401 over the CNR tracks between the Cataraqui wetlands to the east and the Montreal Street underpass to the west in Kingston, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site.

The interpretation and recommendations herein are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational 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.

The current plans for this project are to rehabilitate and widen the existing three-span cast-in-place concrete girder bridge structure from four to six lanes of traffic by adding an additional lane to the eastbound and to the westbound side of the bridge and widening the adjacent rock cut and embankment to the west and east, respectively. Provision for a third track on the west side of the existing CNR tracks is being considered by MTO in the design of the proposed rehabilitation/widening.

Information provided by MTO at the proposal stage and corroborated by Department of Highways drawings dated April 1953 indicates that the bridge is supported on spread footings to the west of the CNR tracks and piles driven to bedrock to the east, however no Geocres information or as-built drawings were available to confirm this. There is good agreement between the subsurface conditions encountered during the current and previous investigations, with the exception of the depth to the top of bedrock at the west pier. On the 1953 design drawings, the top of rock at the west pier is indicated to be at about 73 m elevation (equivalent to the top of the rock slabs identified in the current investigation). The current investigation indicates that the top of rock at the west pier is at about 7 m lower, at about elevation 66 m. As built design drawings indicate that the west pier is supported on spread footings at an elevation of about 73 m elevation, indicating that the footings are likely supported on rock slabs above the bedrock surface.

West of the bridge structure, the approach embankments are constructed within a rock cut. The existing east approach embankments are about 10 m high relative to the surrounding natural ground level. It is understood that current embankment heights are to be maintained.

Foundation engineering recommendations for the bridge foundations are provided in Sections 6.3 to 6.5.

6.2 Bridge Foundation Options

The following options have been considered for the foundations of the widened portions of the bridge:

- Shallow foundations (i.e., spread footings) bearing on bedrock.
- Deep foundations (cast-in-place concrete caissons, driven steel H-piles, drilled piles or micropiles) which derive their support from end-bearing on the bedrock surface at depth.

Geotechnical recommendations for the design of the foundations for the bridge abutments are presented in the following sections. Summary comparisons of the advantages, disadvantages, relative costs, and risks associated with each foundation option are presented as tables following the text of this report. Separate tables have been prepared for foundation elements at the west abutment, west pier, east pier and east abutment, in Tables 1 through 4, respectively.

Due to the varying subsurface conditions at the site, there are several geotechnically preferred options for the support of the bridge foundations. At the west abutment location, where bedrock is close to surface, spread footings are considered the preferred foundation option. At the west pier, where subsurface investigations indicate the presence of detached limestone slabs above bedrock, micropiles are the recommended foundation option. Finally, for the east pier and abutment where the overburden is thick, foundations supported on steel H-piles driven to bedrock are considered the preferred option. Concrete caissons are also feasible; however, they are not considered economical for foundations at the depths required at the east pier and abutment.

6.3 Shallow Foundations

The use of shallow foundations (spread footings) placed on or within relatively near-surface bedrock is considered appropriate for support of the west abutment, and is consistent with the foundation support for the existing structure. Support of the west pier on spread footings, as indicated on the 1953 drawings to be the current method of support, was considered as an option for support of the new pier footings, but is not the preferred solution given the presence of a 7.0 m thick zone of limestone slabs (which may have previously been mistaken as bedrock) overlying bedrock at a depth of 14 m below ground surface. Support of the east pier and abutments on spread footings is not considered feasible as the depth of firm bedrock bearing is some 12 to 22 m below grade.

6.3.1 West Abutment

Footings at the west abutment may be founded at or below elevation 82.5 m. Excavations in the order of 3 to 4 m below existing ground surface will be required to reach the bedrock surface. At the north side of the highway, borehole B6 was put down 5 m west of the west abutment. The bedrock surface near the abutment is expected to drop off quickly to the east and, as such, there is a risk that the top of bedrock is significantly deeper than anticipated based on the results of borehole B6. As such, the design for the west abutment footings should be flexible enough to allow for some variation in the bedrock surface elevation and placement of mass concrete or a working slab to raise the grade by up to 2 to 3 m to the founding level after exposing the bedrock and removing any loosened/fractured bedrock, if required.

If the rock surface exceeds the practical excavation depth at the north side of the west abutment, consideration may need to be given to the use of driven pile foundations at this location.

To mitigate the risk of sloping bedrock at the west abutment, we recommend that the contractor determine the bedrock elevations within the footing footprint (plus an additional 2 m to the east) prior to excavation and submit the data for review to the CA a minimum of 4 weeks in advance of footing construction to allow for modifications to the structural and foundation design, as necessary. A separate non-standard special provision (NSSP) has been prepared (see Appendix B) and should be included in the contract package.

6.3.2 West Pier

Consideration was given to founding footings at the west pier at the top of rock slabs at an elevation at or below 73 m, consistent with the reported founding depth of the existing west pier spread footings. However, due to the presence of voids or loose soil below this elevation, it is expected that, even if lightly loaded, differential settlement of the new piers relative to the existing piers would occur. A significantly reduced SLS bearing pressure (in the order of 250 kPa) would need to be used for footings placed on rock slabs and minimum excavation depths in the order of 6.5 to 7.5 m below existing ground surface (and adjacent to the CNR tracks) would be required to reach the rock slab surface. To mitigate the risk of settlement and bearing failure due to the voids and loose soil, a series of probe holes would need to be put down within the footprint of the footing at regular intervals and that, where voids are encountered, they would either be grouted with a low viscosity grout, or the depth of excavation would need to be extended. Given the uncertainties surrounding the bearing capacities and settlement behaviour of the rock slabs, shallow foundations are not considered the preferred foundation treatment at the west pier.

6.3.3 Excavation

Excavations within the predominantly granular grade fills can either be cut at relatively shallow slopes or, where site constraints limit the extent of excavations, at steeper or near-vertical slopes where appropriate excavation shoring is provided (as outlined in Section 6.10.2 Temporary Excavation and Shoring). Depending on the chosen founding level for the footings and the quality of near-surface bedrock, some rock excavation may also be required. Excavation could be carried out using drilling and hoe ramming techniques where relatively shallow depths of cut into the bedrock are required, however this procedure tends to result in a very uneven founding surface and significant over-excavation is likely. Line drilling and pre-shearing techniques provide better control over the configuration of the founding surface and would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

The contract documents should contain the MTO Special Provision SP902S01 – Excavation and Backfilling – which contains reference to the use of a Quality Verification Engineer to inspect the foundation area prior to footing construction. All footing excavations should be inspected prior to placing concrete to ensure that the base has been adequately cleaned and that the bedrock conditions as exposed at the founding level are consistent with the design assumptions. All loose or shattered rock within the footprint of the footings should be removed from the base of the excavation and replaced with concrete.

6.3.4 Limits States Factored Geotechnical Resistance and Reaction

Spread footings placed on sound bedrock may be designed for a factored geotechnical resistance at ultimate limit states (ULS) of 5 MPa. This value is for vertical concentric loads only. Spread footings placed on rock slabs should be designed for a factored geotechnical resistance at ultimate limit states (ULS) of no more than 350 kPa and a geotechnical resistance at serviceability limit states of no more than 250 kPa for total differential settlements of 25 mm and 19 mm, respectively. Effects of load eccentricity need to be taken into account as appropriate in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* using the curve for “cohesive soil or rock”. Serviceability limit states (SLS) conditions do not apply to footings placed on the sound limestone bedrock which is classified as non-yielding.

The factored geotechnical resistance value for sound bedrock given above assumes that the bedrock at and below the founding level has not been fractured, and that no adverse jointing is present below the footings.

Additional rock reinforcement in the form of rock bolts or dowels and/or protection in the form of shotcrete may be required before the footings are constructed in order to ensure the integrity of the rock mass.

Foundation elements for the west abutment should in all cases be located a minimum horizontal distance of 2 m from the existing fill slope.

6.3.5 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete footings and bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ, may be taken as 0.7 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, sliding resistance can be supplemented by doweling the footings into bedrock. The horizontal resistance of the dowels will be dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedded in concrete. The dowels should have a minimum embedded length of 1.0 m within the bedrock, and the structural strength of the dowel and the compressive strength of the grout should not be exceeded. A Non Standard Special Provision for dowels in rock should be included in the contract documents and a sample has been included in Appendix B of this report.

The geotechnical resistance to lateral loading on the foundations, as provided by the means of the geotechnical resistance or dowels, will be reduced wherever rock is not present below the founding level in the areas in front of the footings. This may for example be the case at the west abutment where the edge of the rock is in close proximity to the footings. As such we recommend that all foundation elements be located a minimum of 2 m horizontal distance from the edge of the rock.

6.3.6 Frost Protection

For spread footings founded on fresh limestone bedrock or mass concrete, frost protection cover is not required.

6.4 Pile Foundations

As noted in Section 6.1, piled foundations on limestone and/or sandstone bedrock are the preferred method of foundation support for the west pier, east pier and east abutment. At the west pier, where subsurface investigations indicate the presence of detached limestone slabs above bedrock, pre-drilled H-piles, drilled piles or micropiles are considered technically feasible foundation options for extending the foundations to the underlying bedrock. As built drawings indicate that the existing west pier is founded on spread footings at the level of the top of the "limestone slabs", rather than on bedrock. Because the new deep foundations for the west pier would be extended to the top of bedrock, well below the existing west pier foundations, it is considered prudent to use caution in advancing the new foundations. To reduce the potential negative impacts of construction of new deep foundations adjacent to the existing shallow west pier foundations, we recommend that micropiles be selected as the foundation option at the west pier. At the east pier and east abutment where the overburden is thick, foundations supported on steel H-piles driven to bedrock are considered the preferred option and are consistent with the existing foundation support. Given the proximity of bedrock to surface at the west abutment, piled foundations at this location are not considered a preferred option.

The following table summarizes the anticipated pile toe elevations and founding stratum for the east pier, east abutment and west pier based on the depth to bedrock encountered in the boreholes. We have assumed that the underside of the pipe cap at the east and west piers is at elevation 74.4 m and at 82.5 m at the east abutment, as shown on the latest GA drawing.

Location	Approximate Ground Surface Elevation (m)	Anticipated Pile Cap Elevation (m)	Anticipated Toe Elevation (Top of Rock) (m)	Anticipated Average Pile Length (m)	Founding Stratum
East Pier	77 – 77.5	74.4	65 - 66	9	Limestone or Sandstone Bedrock
East Abutment	82 – 82.5	82.5	63 - 65	18.5	Limestone or Sandstone Bedrock
West Pier	79	74.4	66 *top of rock slabs at 73	8.5	Limestone Bedrock

All pile installation/driving should be in accordance with Special Provision SP903S01. Installation of micropiles at the west pier should be carried out in accordance with a Non Standard Special Provision which can be provided by Golder as a part of the detailed design deliverable if this foundation option is chosen. At the east pier and abutment, driven piles will essentially be advanced to practical refusal on bedrock. Depending on the required pile capacities, drilled piles at the west pier will either be socketed nominally or embedded to a target depth into bedrock. As noted above, micropiles are the recommended foundation choice for the west pier. For all piling methods, a Non Standard Special Provision to alert the contractor of the presence of limestone slabs and of boulders/obstructions within the rock fill and glacial till should be included in the contract documents and a sample has been included in Appendix B of this report.

The pile termination or set criteria for driven piles will be highly dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, for piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile. The pile termination depth for drilled piles or micropiles will be based on the minimum required axial and lateral capacity.

If additional piles are installed to widen the existing abutment and piers, new piles may be in close proximity to the piles supporting the existing structures. If existing piles are offset from their intended location or alignment, the potential exists for conflicts when driving the new piles. Current construction practice generally limits the acceptable pile offset from design *at the surface* to 50 mm and the deviation from the design inclination to 2 percent. However, even for piles installed meeting these construction limits, the tip offset at depth may be greater and it is considered that, for piles less than 10 m in length such as at this site, the tip offset *at depth* may be as much as 5 percent of the pile length. As such, for new piles put down within the potential zone of interference with the existing abutment or pier piles (defined as a distance around the existing pile centre equal to 10 percent of the pile length), the installation operations shall be continuously monitored by the QVE and the contractor shall cease advance of the pile if the QVE indicates that the new pile may have come in contact with an existing pile. If contact between the new and existing piles is believed to exist, it may be necessary to extract or re-install piles. A Non Standard Special Provision for installation of piles adjacent to existing battered piles should be included in the contract documents and a sample has been included in Appendix B of this report.

For both driven and drilled piles, vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity of 100 mm/s is recommended adjacent to existing abutments and east pier. At the west pier, where there is a significant risk of damage to the existing shallow spread footing foundations, maximum peak particle velocities should be limited to 50 mm/s to minimize vibrations of foundations potentially supported on rock slabs. Piles put down furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the pile driving criteria for the remaining piles. A Non Standard Special Provision for vibration monitoring should be included in the contract documents and a sample has been included in Appendix B of this report.

6.4.1 Pile Options and Axial Geotechnical Resistance

6.4.1.1 *Steel H-Piles Driven or Installed in Pre-Drilled Holes*

As noted above, steel H-piles driven to refusal on bedrock is the preferred foundation option at the east pier and east abutment where the overburden is thick. At the west pier, a thick zone of rock slabs above the bedrock surface is expected to severely limit the successful driving of H-piles to bedrock and attempts to advance the piles through the rock slabs may induce significant vibrations which could be harmful to existing foundations. At this location, consideration could be given to installing H-piles in pre-drilled holes to advance through the rock slabs and achieve the desired depth; however, this too may induce harmful vibrations. The cost of installing pre-drilled H-piles will also be significantly higher than driving H-piles because specialized drilling systems, such as the “Symmetrix (ROTEX)” or dual rotary (Barber), would be required (see Section 6.4.1.2 below) to advance a hole of sufficient diameter to accommodate the H-pile. At the west abutment, where the depth to bedrock is shallow, driven steel H-piles are also not preferred.

Boreholes advanced at the east abutment and east pier indicate the potential for cobbles and boulders within the rock fill. To minimize the risk of damage to pile tips, vertically driven and battered piles should be equipped with suitable driving points (such as Titus standard rock bearing point or equivalent) to ensure seating of the piles on the bedrock. For driven steel H-piles at the east pier and abutments, the drawings should incorporate an appropriate note stating that the piles should be equipped with rock bearing points and should be driven to bedrock.

Even with suitable driving points, there is a risk of damage to piles and/or misalignment of the piles driven to bedrock. In the event that the piles are damaged and/or driven out of alignment when driven through the fill, it would be necessary to remove and re-drive the piles. Alternatively the pier or abutment design would have to be flexible enough to allow for the installation of extra piles within the footing area, if considered necessary during the installation.

If pre-drilled piles are chosen for the west pier, they should be advanced with a liner socketed a minimum of 0.6 m into bedrock to limit the inflow of soil and groundwater into the base of the hole. The installations must be inspected by qualified geotechnical personnel to ensure that the founding stratum has been reached and is consistent with the design assumptions and that the base has been properly cleaned and is dry prior to installation of the H-pile. Once the H-pile is installed, the annulus between the liner and H-pile can then be backfilled with grout.

For design of steel H-piles that are successfully advanced to found on the bedrock, the following factored axial resistances at Ultimate Limit States (ULS) may be assumed:

Pile Size	Factored ULS Resistance (kPa)
HP 310 x 110	2,000
HP 360 x 132	2,400
HP 360 x 152	2,750

The above values represent structural limitations for the piles rather than geotechnical limitations.

The geotechnical resistance at SLS for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS because the bedrock is considered to be an unyielding material. As such, ULS conditions will govern for this foundation type, providing the piles are successfully driven to bedrock.

6.4.1.2 *Drilled Pipe Piles*

At the west pier, to advance through the rock slabs to bedrock, consideration could be given to the use of drilled piles. The general procedure involves using smaller diameter (less than 324 mm in outer diameter) steel pipe piles advanced through the overburden soils and into the bedrock using down-the-hole hammer techniques. In general, the drilled pile system uses a four step process. The first step is to weld a non-salvageable ring (i.e., crown) to the end of a steel pipe pile that will be used to drill into the bedrock and allow rotation of the shoe without rotation of the steel pipe. The next step is to insert the pilot bit into the steel pipe pile, which locks into the crown by rotating clockwise. The next step involves drilling through the overburden and bedrock by rotating the lower part of the crown (called the driver) and the pilot bit while the upper part of the crown and the steel pipe casing does not rotate. The last step (after the steel pipe casing reaches the required bedrock socket depth) involves reversing the drill direction to unlock and retrieve the pilot bit, and leaving the steel pipe and non-salvageable crown in place. The steel pipe can then be filled with tremie concrete (if there is water inflow through the bedrock) and reinforcing steel added, if required.

The drilled pile excavations must be inspected by qualified geotechnical personnel to ensure that the founding stratum has been reached and is consistent with the design assumptions and that the base has been properly cleaned and is dry. In this regard, temporary liners (i.e. the steel pipe piles) will be required to limit inflow of soil and groundwater into the hole. The base of the hole should be flushed and any drilling debris removed to ensure adequate base capacity.

Drilled piles derive their axial resistance in part from end-bearing and in part from shaft friction. For this site, the majority of the resistance will be derived from base resistance. The factored axial geotechnical resistance at ULS that may be used for design of a single drilled pile are given in the table below:

Drilled Pile Size	Socket / Anchor Details	Factored Axial Geotechnical Resistance Bedrock
		ULS
300 mm diam. Drilled Pile (tremie concrete filled, 13 mm thick steel pipe)	Socketed a minimum 0.6 m into bedrock	2000 kN [(1)]
324 mm diam. Drilled Pile (tremie concrete filled, 13 mm thick steel pipe)	Socketed a minimum 0.6m into bedrock	2400 kN [(1)]

(1) Values are based on structural capacity of the pile and may need to be adjusted depending on final configuration, pipe steel grade, concrete strength, bedrock socket details, and reinforcing steel, if applicable.

For drilled piles founded in the bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

For larger diameter drilled piles (i.e., greater than about 324 mm diameter to up to 1200 mm), an installation method similar to the system described previously would be required to achieve adequate socketing for larger axial resistance capacities. It should be noted that larger diameter piles would likely induce more vibrations, become more difficult to excavate and require more specialized equipment (“Symmetrix” or Dual Rotary drilling rigs) to install. As a result, larger diameter drilled piles may be uneconomical.

6.4.1.3 *Micropiles*

As discussed in Section 6.4, subsurface investigations indicate the presence of detached limestone slabs above bedrock at the location of the west pier foundations. In these locations, individual limestone slabs which range in thickness from 0.1 m to 1.7 m are separated by voids or loose soil infilling. The zone of limestone slabs is about 7 m thick. At borehole B7, between the limestone slabs and the underlying bedrock, a layer of cobbles, boulders and silty clay was encountered. Inspection of the recovered core indicates that the slabs are comprised of limestone similar to that encountered at the west abutment, with individual slabs oriented with sub-horizontal bedding. The sub-horizontal bedding of the limestone slabs suggests that 7 m thick section may have been displaced as a block during glacial activities at the contact between the limestone bluffs to the west and the lowlands to the east. If bedding orientations had been variable, it would have suggested that the material was a talus deposit of blocks which had fallen from the bluffs to the west.

As built drawings indicate that the west piers are founded on spread footings at the level of the top of the “limestone slabs” (rather than on bedrock). We recommend that new foundations for the west pier be extended to the top of bedrock, to a depth which would extend below the existing west pier foundations. To minimize disturbance of the west pier foundations during construction of the adjacent new deep foundations, we recommend that micropiles be selected as the foundation option at the west pier.

Micropiles are small diameter (typically less than 300 mm) drilled and grouted replacement piles that are typically reinforced with high-capacity steel (typically threaded bars or reinforced steel) to resist a high proportion (or all) of the design load. Micropiles are often cased through overburden deposits and then socketed and grouted directly into bedrock at depth. The casing may or may not be removed, depending on the structural stiffness and axial/lateral capacities required. The special drilling and grouting methods used in micropile installation allow for high grout-to-ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. Theoretically micropiles have equal capacities in both tension and compression. Vertical micropiles may be limited in lateral capacity. The total pile length (i.e., embedment into the underlying bedrock) is determined by detailed pile design calculations.

The construction equipment/techniques and risks associates with micropiling are similar to those required for drilled piles or for pre-drilled holes for H-piles to advance through cobbles and boulders or rock slabs. However, because micropiles require a smaller hole than pre-drilled H-pile holes or drilled piles, installation is generally faster and vibration-induced movements on adjacent structures would likely be less. Also, less reinforcement (i.e., 2-#18 steel bars instead of a HP 310x110 steel section in a pre-drilled H-pile hole or a 13 mm thick steel pipe pile) is required.

Larger diameter micropiles have the advantage of requiring a shorter bond length to accommodate the required design loads and therefore will need less drilling into the bedrock (i.e., overall shorter length pile). Larger diameter micropiles are also stiffer and provide more buckling resistance over the free length of the pile. However, drilling with the larger diameter through rock slabs will be more time-consuming and poses a potentially greater risk of vibration, ground loss or settlement and movement of the existing structure. Smaller diameter micropiles may mitigate some of these risks, but would require a longer bond zone (i.e., overall longer length of pile) to achieve the same capacity.

Micropile Size	**Socket / Anchor Details**	**Factored Axial Geotechnical Resistance Bedrock**
		ULS
230 mm diameter casing, 2 - #18 central steel bars	Socketed into bedrock	1500 kN

Micropile Size	Socket / Anchor Details	Factored Axial Geotechnical Resistance Bedrock
		ULS
250 mm diameter casing, 2 - #18 central steel bars	Socketed into bedrock	2000 kN

For drilled micropiles founded in bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

The additional costs incurred for detailed geotechnical and structural design of the micropiles and micropile load tests would need to be considered. The detailed geotechnical design would include:

- liaison with the structural engineer on the initial pile section geometry and steel grade for structural considerations (i.e. lateral loads, proposed pile batter, required free length, etc.);
- assessment of the geotechnical capacity of a single micropile following the guidelines published in the FHWA “Micropile Design and Construction Guidelines Implementation Manual (June 2000);
- assessment of the lateral performance of a single micropile, development of p-y curves (using LPILE Plus v5.0) and reduction factors for group effects for use by MRC to carry out the assessment of capacity and performance of the full pile group;
- additional liaison with MRC during iterations, as required, to achieve efficient micropile group design;
- revision of existing MTO Special Provisions for micropiles developed recently by Golder for another MTO project; and
- preparation of a separate report/design memoranda specific to micropiled foundations.

A separate scope of work and cost estimate for detailed geotechnical design of micropiles can be provided if this foundation option is selected for the west pier.

6.4.2 Downdrag Load (Negative Skin Friction)

The widening of the existing east approach and placement of additional fill on the north and south shoulder east of the east abutment will raise the effective stress level in the silty clay deposit which underlies the site. It is understood that the maximum height of additional fill to be added at the north and south edge of the roadway is about 4 m. This increase in stress will lead to some compression of the silty clay deposit encountered in boreholes B4 and B9 put down at the east abutment. As discussed subsequently in Section 6.9.5 of this report, the magnitude of the resulting consolidation settlement of the embankment subgrade is estimated at about 40 to 60 mm, with additional long term settlement due to secondary compression (creep) and compression of the existing embankment fills.

The consolidation settlement is time-dependant and may not be complete by the end of the construction period. As such, some post-construction settlement of the silty clay deposit should be expected. The piles will be end-bearing on bedrock, and as such, even a small amount of settlement of the silty clay deposit relative to the pile will result in the development of negative skin friction. These negative skin friction or downdrag loads will need to be taken into account during design of the new piles supporting the east abutment.

In calculating the magnitude of the downdrag force, the methods described in the Canadian Foundation Engineering Manual were used. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in the analyses was assumed to be at the underside of the silty clay deposit.

Based on the above, for new piles supporting the east abutment (with the underside of the pilecap at elevation 82.5 m), the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the silty clay and overlying embankment fill is estimated to be about 900 kN. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.Given the geometry of the widening, no significant increase in load is expected at the pier foundation areas and, as such, downdrag loads are not anticipated at the east or west pier foundations.

It is not known whether downdrag loads were considered for the design of the existing piles; however, ground movements will occur within the existing embankment and will likely be large enough in magnitude to generate downdrag forces on existing piles located more than 10 m from the median on either side. As such, the effects of negative skin friction or downdrag loads on the existing east abutment piles close to the embankment widening should also be considered. Available information indicates that the existing east abutment piles are considerably shorter than the new piles (with the underside of the pilecap at elevation 74 m rather than 82.5 m) and, as such, the resulting downdrag loads on the existing east abutment piles will be less. The unfactored downdrag load acting on a single BP 12 x 12 x #53 pile over the length of the pile within the silty clay is estimated to be about 500 kN.

It is understood from design drawings that the piles supporting the existing bridge were installed in the 1950's and are equivalent in size to HP 310 x 79. We understand that, at the time, the design working load for these piles was 70 to 75 tons (about 650 kN). Assuming that the piles were driven to found upon the bedrock (as indicated in the drawings), the factored Ultimate Limit States resistance of the existing piles is estimated at 1,350 kN. This value represents a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions govern for this foundation type.

6.4.2.1 *Downdrag Mitigation*

If the predicted downdrag loads cannot be accommodated structurally, downdrag loads on new piles can be reduced through modifications to the pile geometry/frictional properties, or eliminated by significant reduction of the new loads and/or minimization of post-installation settlement of surrounding soils. For the existing bridge foundation piles, modifications to the pile geometry or frictional properties cannot be made and all induced settlements are "post-installation", so the options to mitigate downdrag on existing piles are limited to reduction of settlement by reduction in loading.

If downdrag loads on <u>new</u> piles are too high, the following means of reducing downdrag through geometric/frictional property modifications could be considered:

- reducing the new pile diameter (reducing the unit surface area of the pile would result in a corresponding reduction in the total downdrag load);
- changing the frictional properties of the soil/pile interface by casing the pile through the embankment fill (eliminating the contribution of the embankment fills to the total downdrag load would reduce downdrag by about 60 percent), however lateral resistance to the pile from the embankment fill would be affected;
- coating the piles with a friction reducer (e.g., bituminum). Case histories indicate that friction reducers can reduce the total downdrag loads by 80 to 90%, but that the application of the coating increases the cost per pile by 15 to 50% over the cost of uncoated piles and is not economical for a small job;

Alternatively, consideration could be given to the use of a heavier gauge pile, with higher structural capacity but similar surface area (i.e., HP 360 x 152 or HP 360 x 132).

Downdrag loads could also be eliminated using the following settlement mitigation strategies:

- preloading to minimize post-construction settlements to less than 10 mm. If the widened slopes are constructed and allowed to settle for a period of at least 4 to 6 months (estimated duration of primary consolidation) before installing the new piled foundations, settlement resulting from immediate elastic compression of the granular fills and primary consolidation settlement could occur prior to installation and downdrag on the new piles could be eliminated; or,
- Installation of lightweight fill (EPS) to reduce or eliminate applied loads due to the widening, thus eliminated downdrag on the piles (on both new and existing piles).

Additional information regarding the settlement mitigation options is provided in Section 6.9.5 of this report.

6.4.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. Alternatively, the resistance to lateral loading could be derived from the soil resistance in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.8.7.2 of the *Commentary to the CHBDC*.

The SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where:	n_h	Is the constant of horizontal subgrade reaction, as given below;
	z	Is the depth (m); and,
	B	Is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: kh Is the coefficient of horizontal subgrade reaction;

su Is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

The following values of n_h and s_u may be assumed in the structural analysis.

Location	Soil Type	n_h (MN/m^3)	s_u (kPa)
East Abutment	New Compacted Fill	6.6	-
	Existing Embankment Fill	4.8	-
	Weathered Crust	-	100
	Silty Clay	-	40
	Glacial Till	8	-
East Pier	New Compacted Fill	6.6	-
	Existing Embankment Fill	1.6	-
	Weathered Crust	-	100
	Silty Clay	-	20
	Glacial Till	8	-
West Pier	New Compacted Fill	6.6	-
	Existing Embankment Fill	6.0	-
	Weathered Crust	-	100
	Limestone Slabs	11	-

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

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using the procedures and parameters given above for evaluating the SLS response.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary to the CHBDC*.

The unfactored lateral passive resistance of a single pile in non-cohesive soils (new and existing fills) may be estimated by calculating passive earth pressure over an equivalent wall area having a depth from the ground surface equal to six times the pile diameter, and with a width equal to three times the pile diameter.

For individual piles in cohesive soils (i.e., silty clay) the ULS lateral resistance is assumed to vary linearly with a magnitude of $2S_u$ at the surface of the deposit and a magnitude of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap (where S_u is the undrained shear strength). Below a depth equal to 3 pile diameters, and to the bottom of the deposit, the lateral resistance is assumed to be constant at $9S_u$.

The ULS lateral passive resistance from the glacial till and rock slabs should be neglected since, in these non-cohesive soils, the CHBDC Commentary (Section C6.8.7.1) suggests that resistances only be considered within a depth equal to six diameters below the underside of the pile cap; these soils are below that depth.

The ULS lateral resistance of a pile group may be estimated as the sum of the individual resistances across the face of the group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the *CHBDC*, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

For *preliminary* design purposes, the ULS *geotechnical* resistance can also be estimated using the "Assessed Horizontal Passive Resistance Values for Various Pile Types" provided in the *Commentary to the CHBDC*. On that basis, a maximum lateral resistance of 125 kN at ULS (unfactored), and a maximum lateral resistance of 50 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles.

If micropiles are selected as the foundation option at the west pier, the lateral performance of a single micropile will need to be determined using a rigorous solution involving the development of p-y curves (using LPILE Plus v5.0) and reduction factors for group effects. This assessment can be started once the initial pile section geometry (e.g. diameter, steel grade, batter, applied lateral loads, required free length) is established by the structural designer. The structural designer would then use the lateral performance information to carry out the assessment of capacity and performance of the full pile group. A number of iterations will likely be required before a suitable foundation design and pile group geometry can be established.

6.4.4 Frost Protection

The pile caps should be provided with a minimum of 1.5 m of soil cover for frost protection. At the east pier, if the installation of temporary shoring (required for below grade pile caps) adversely impacts rail traffic operations, the depth of the pile cap could be reduced to less than 1.5 m below grade by either elevating the pile cap above grade or properly insulating it such that the minimum path distance for frost (i.e. the depth of footing below ground plus the lateral extent of insulation away from the pile cap) is equal to 1.5 m.

6.5 Caisson Foundations

Caissons founded on or socketed into the bedrock are considered a technically feasible method of support for the east and west piers and east bridge abutment. At the east pier, if the proximity of the adjacent rail tracks restricts installation of temporary shoring for pile caps, caissons may provide a suitable excavation-free alternative. Advance of the caissons to bedrock at the west pier will require extensive churn drilling due to the rock slabs and may induce vibrations of the existing bridge foundations. Such vibrations may induce excessive settlements, thus the use of caissons at the west pier may require underpinning of the existing foundations to control vibration-induced settlements. At the east abutment, the significant thickness of embankment fills and resulting depth to bedrock may render this option cost-prohibitive.

The following table summarizes the anticipated toe elevations and founding stratum at each location. We have assumed that the underside of the pile cap at the west and east piers will be at 1.5 m below existing ground surface for frost protection purposes and that the grade in front of the east abutment will be at about 3.0 m below pavement surface.

Location	Approximate Ground Surface Elevation (m)	Anticipated Pile Cap Elevation (m)	Anticipated Toe Elevation (Top of Rock) (m)	Anticipated Average Caisson Length (m)	Founding Stratum
West Pier	79	74.4	66 *top of rock slabs at 73	8.5	Limestone Bedrock
East Pier	77 – 77.5	74.4	65 - 66	9	Limestone or Sandstone Bedrock
East Abutment	82 – 82.5	82.5	63 - 65	18.5	Limestone or Sandstone Bedrock

The use of a temporary liner or permanent casing is recommended in order to advance the caissons through the overburden with minimal loss of ground. Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is socketed into the bedrock. It may therefore be more practical to socket the caissons into the rock a minimum of 300 mm, rather than found on the bedrock surface.

The bedrock at the site is moderately strong to strong. If socketing of the caissons into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling. Rock coring or churn drilling may be required at the piers and east abutment to advance through the rock fill and will likely be required to advance the casing through the limestone slabs encountered at the west pier. It is recommended that a Non Standard Special Provision be included in the Contract Documents to advise the Contractor of the presence of cobbles or boulders in the rock fill and that the bedrock is medium strong to strong and will require churn drilling. If this foundation option is adopted, a sample NSSP will be prepared for inclusion in Appendix B.

During caisson installation, vibration monitoring should be carried out to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity of 100 mm/s is recommended adjacent to existing east pier and abutment and 50 mm/s adjacent to the west pier. A Non Standard Special Provision for vibration monitoring, similar to that provided for driven piles in Appendix B, will be prepared for inclusion in the contract documents if this foundation options is adopted.

6.5.1 Axial Geotechnical Resistance

Caissons founded on the bedrock surface or socketed nominally (less than 1 m) into the bedrock should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 5 MPa for limestone and dolomitic limestone bedrock. At the north side of the east pier and at the east abutment, a factored geotechnical resistance at USL of 8 MPa should be used for sandstone bedrock. SLS resistances do not apply to caissons founded on or socketed in the bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.5.2 Downdrag Load (Negative Skin Friction)

The widening of the existing east approach and placement of additional fill on the north and south shoulder areas behind the east abutment will raise the effective stress level in the silty clay deposit (east abutment only), leading to some consolidation of the deposit. As discussed previously in Section 6.4.2 for piles, this condition will also result in downdrag forces on caissons. The unfactored downdrag load acting on a single 0.9 m or 1.5 m diameter caisson over the length of caisson within the silty clay and overlying embankment fill is estimated to be 2000 kN and 3400 kN, respectively (based on an underside of pile cap level at about elevation 82.5 m). The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described in Section 6.4.2 of this report with respect to steel H-piles.

Downdrag loads are not anticipated at the pier foundations, as no significant filling is proposed for the pier foundation areas.

6.5.3 Resistance to Lateral Loads

The resistance to lateral loading is derived from the soil in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.4.3.

6.5.4 Frost Protection

The pile (caisson) caps should be provided with a minimum of 1.5 m of soil cover for frost protection, or alternatively they can be suspended above grade or suitably insulated as per Section 6.4.4.

6.6 Feasibility of Integral and Semi-Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site is not considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35º; and abutment wall heights less than 6 m without a retained soil system.

As outlined in MTO's report BO-99-03, semi-integral abutment bridges are single or multiple span structures of less than 150 m in length with rigid foundations (spread footings) where the concrete deck is continuous with the approach slabs. Expansion joints are eliminated at the end of the deck and the superstructure is supported on movable bearings and is almost independent of the abutment. A control joint is provided at the end of the approach slab that is detailed to slide in between the wingwalls. Unlike integral abutment bridges, there is no limit on skew angle for semi-integral abutments provided that lateral restraint is incorporated in the bridge design to prevent rotation of the superstructure caused by eccentric lateral earth pressures in the horizontal plane acting on both ends of the superstructure and that the movement system at the end of the approach can accommodate deformations associated with skew.

The flexible pile-supported east abutment foundations meet MTO's foundation criteria for integral abutments, however the current overall skew of the bridge is such that integral abutments may not be possible. The use of semi-integral abutments would be feasible at the west abutment, where the ground conditions allow for support of the structure on rigid spread footings founded on bedrock.

We understand that lateral movement in the order of +/- 15 mm can be expected at the back of the deck diaphragms due to thermal expansion and contraction if integral or semi-integral abutments are used. In order to accommodate this movement, we recommend the installation of a minimum of 1.5 m width of compacted granular material behind the deck diaphragm where footings are adjacent to vertical rock faces (i.e., west abutment).

6.7 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

6.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the bridge abutments and pier will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment stems and retaining walls in accordance with CHBDC:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS 539.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 539. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the abutment stem (Case (a) in Figure C6.20 of the Commentary to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case (b) in Figure C6.20 of the Commentary to the CHBDC).

6.8.1 Static Lateral Earth Pressures for Design

- For the proposed and existing embankment fill materials (Case I), the following unfactored lateral earth pressure parameters may be used assuming the use of Select Subgrade material:

Soil Unit Weight:	$21\ kN/m^3$
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.33 0.50 3.0

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

	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	$22\ kN/m^3$	$21\ kN/m^3$
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.27 0.43 3.7	 0.27 0.43 3.7

- If the wall support and superstructure allow lateral yielding or where the abutments are expected to move away from the retained soils as the superstructure contracts due to decreases in ambient temperature, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
  - Rotation of approximately 0.002 about the base of a vertical wall:
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.
- If the abutment support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- Where the abutments are expected to move into the retained soils, such as at semi-integral abutments where the abutment expands due to increases in ambient temperature, passive earth pressures should be used in geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. The movement to allow passive pressures to develop within the backfill may be taken as:
  - Rotation of approximately 0.100 about the base of the vertical wall;
  - Rotation of approximately 0.020 about the top of a vertical wall;

- Horizontal translation of 0.05 times the height of the wall; or,
- A combination of the above.

Where movements are not great enough to mobilize full passive resistance, K_P may be determined in accordance with Figure C6.16 of the *CHBDC Commentary* based on the amount of displacement.

As discussed below in Section 6.7.5 of this report, the use of expanded polystyrene (EPS) light weight embankment fill is an option to mitigate the potential roadway settlements at the east abutment due to compression of the underlying silty clay deposit. The low unit weight and relatively high mechanical strength characteristics of the EPS blocks (in comparison to soil) will alter the design lateral earth pressures. For design purposes, the EPS should be assumed to have a unit weight of 1 kN/m3; this low unit weight should be considered in the calculation of the vertical stress level in the underlying granular backfill, and thus the horizontal lateral pressure applied to the abutment wall. Furthermore, because the EPS blocks would hold a vertical face without support, the lateral earth pressure applied by the EPS itself could be quite minor, resulting only from the resistance to lateral expansion of the material under vertical loading (i.e., from the 'Poisson' effect), which is generally small and difficult to quantify (and highly dependent on how tightly fitting the EPS blocks are placed against the abutment). It is therefore considered that the lateral earth pressures from the EPS can be neglected.

6.8.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the *CHBDC*. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.1 of the CHBDC, the site is located in Seismic Zone 2. The site-specific zonal acceleration ratio for Kingston is 0.1. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.1.

- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, kh, used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., kh = 0.15). For structures which allow lateral yielding), kh is taken as 0.5 times the zonal acceleration ratio (i.e., kh = 0.05). The vertical seismic coefficient, k_v, used in the calculation is assumed to range from -0.5 to 0.5 times the horizontal seismic coefficient, k_h.

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design.

Seismic Active Pressure Coefficients, K_{AE}

	Case I	Case II	
		Granular 'A'	Granular 'B' Type II
Yielding wall	0.34	0.28	0.28
Non-yielding wall	0.46(1)	0.38(1)	0.38(1)

[1] The CHBDC seismic K_{AE} values reported above include the effect of wall friction ($\delta = \phi'/2$) and are less than the static values of K_0 for the very low zonal acceleration ratio for this site. As such, for non-yielding walls only static earth pressures need to be considered for this low seismicity (A=0.1) location.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.10. This corresponds to displacements of up to approximately 25 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K \gamma d + (K_{AE} - K) \gamma (H-d)$$

Where:

$\sigma_h(d)$ Is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K Is either the static active earth pressure coefficient, K_a, or the static at-rest earth pressure coefficient (K_0);

K_{AE} Is the seismic active earth pressure coefficient;

γ Is the unit weight of the backfill soil (kN/m^3), as given previously;

d Is the depth below the top of the wall (m); and,

H Is the total height of the wall (m).

- The following dynamic increment of passive pressure coefficients (K_{PE}-K_P) for the two backfill cases (Case I and Case II) may be used in design of the bridge deck ends for semi-integral abutments. These coefficients represent the maximum value of (K_{PE}-K_P) obtained using the k_h and three value of k_v as described above and assuming that movements are sufficient to mobilize full passive resistance.

Seismic Passive Pressure Coefficients, (K_{PE}-K_P)

	Case I	Case II	
		Granular 'A'	Granular 'B' Type II
Yielding wall	0.1	0.1	0.1
Non-yielding wall	0.3	0.3	0.3

- Where seismic movements are not great enough to mobilize full passive resistance, K_P may be determined in accordance with Figure C6.16 of the CHBDC Commentary based on the amount of displacement. The dynamic increment of passive pressure (K_{PE}-K_P) should be reduced to the same extent as K_P.

- The earthquake-induced dynamic passive lateral pressure distribution, which is to be subtracted from the static passive earth pressure distribution, is a linear distribution with maximum pressure at the base of the wall and minimum pressure at its top (i.e., a triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_P \gamma d - (K_{PE} - K_P) \gamma (H - d)$$

Where:

$\sigma_h(d)$	Is the total (static plus seismic) pressure distribution at depth d, (kPa);
K_P	Is the static passive earth pressure coefficient, K_P,;
$K_{PE} - K_P$	Is the dynamic increment of passive earth pressure coefficient;
γ	Is the unit weight of the backfill soil (kN/m^3), as given previously;
d	Is the depth below the top of the wall (m); and,
H	Is the total height of the wall (m).

It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

6.9 Approach Design and Construction

The existing CNR bridge structure and highway sections to the east and west are to be widened to the north and south as part of the Highway 401 expansion in this area. West of the CNR overpass, widening will require an additional 10 m of permanent cut into the existing limestone rock cut slopes. East of the overpass, the existing 10 m high approach embankments will be widened to the north and south by some 3 to 5 metres, at a slope of about 1.25 Horizontal to 1 Vertical (consistent with the existing side slopes). We have assumed that the new east and west piers will be maintained at about the elevation of the existing piers (i.e., adjacent to the railway track) and that the grade between the piers and abutments will be sloped with rock fill at about 2H:1V to 3H:1V, in line with the existing foreslopes. Filling over the existing east embankment slopes will therefore be required and some filling may be required at the west abutment. Overall, the new abutment and/or retaining walls will be up to about 3 m high.

Wing walls will also be provided extending from the ends of the new abutment walls. The filling behind those walls will widen the embankment footprint in this area (to the junction between the new embankment side-slopes and fore-slopes).

Cast-in-place concrete walls founded on bedrock (at the west abutment) or piled foundations (at the east abutment) may be considered for the new retaining walls and wing walls.

6.9.1 Permanent Cut Slopes at West Approach

For permanent cut slopes through the bedrock, such as those required for the approach to the west abutment, the overall slope to the cut face may be formed vertical to near vertical (i.e., 1 horizontal to 12 vertical) and constructed in accordance with MTO's Special Provision 206S03 for Rock Faces. The use of carefully controlled drill and blast excavation techniques will be required in order to ensure a neat excavation line and to minimize face instabilities and long-term maintenance problems. Alternatively, the rock faces could be excavated mechanically using large hydraulic rock breakers as was done for the section of widening along Highway 401 to the west of Montreal Street, although there may be some restrictions given the height of the rock cuts. Line drilling of the rock face prior to mechanical excavation could be used to produce a neat face with minimal overbreak. Regardless of the method of excavation, mechanical scaling will be required to remove loose rock on the face which may be created due to the blocky nature of the rock mass and the presence of joint sets sub-parallel to the cut face. It is also likely that there will be some overbreak associated with the rock faces due to the joint sets that strike sub-parallel or obliquely to the faces.

The main mechanisms for instability on the existing rock faces are ravelling of loose surficial blocks of rock and the creation of overhangs which can eventually result in undercut blocks of rock falling. The rock falls along the section of highway west of the approach to the CNR bridge are mainly the result of poor blasting practices during original construction which have damaged (fractured) the rock face and ongoing weathering processes, predominantly ice jacking due to freeze thaw cycles in the winter months which tends to loosen blocks on the face. When the loosened blocks eventually fall, they sometimes create an overhang above. Eventually the overhanging blocks may also fall due to further weathering. Where trees are present above the crests of the rock cuts, the roots of these trees can grow inside the joints in the rock mass forcing the joints open and in some cases eventually creating unstable blocks.

To minimize the risk of undercutting the toe of rock cuts, we recommend that a 0.5 m offset be maintained between the base of the rock cut and the outside edge of the ditch. Alternatively, if space limitations preclude this offset, consideration could be given to lining the ditch with shotcrete.

6.9.2 Blasting Considerations

The use of controlled blasting techniques in accordance with OPSS 120 may be used for mass excavation for road widening along the west approach to the CNR bridge. Given the proximity of the existing and new bridge structure, a separate non-standard special provision should be included to highlight the need to minimize damage to the rock face, overbreak and fly rock adjacent to the existing and new structure. A sample non-special standard provision has been prepared and is included in Appendix B.

Above and beyond OPSS 120, the Special Provision includes requirements for:

- Submission of a separate perimeter wall control blast design by the blasting contractor or their blast consultant in accordance with OPSS 120 detailing the proposed blast methodology for perimeter wall control blasting within 50 m of new and existing bridge foundations;
- Separate trial blasts using perimeter wall control blast procedures prior to blasting within 50 m of new and existing structures; and,
- Acceptance of the perimeter wall control blasting methodology by the Contract Administrator following demonstration that the blast design is adequate to minimize damage to the rock face, overbreak and fly rock.

Inspection of the rock cut face immediately after blasting should be carried out by qualified geotechnical personnel retained by the contract administrator in order to assess where scaling I loosened rock removal should be carried out adjacent to the footings and where additional rock bolting may be required. The rock bolts, if required, should be 25 mm diameter, galvanized, fully grouted deformed bars, generally 3 m in length.

6.9.3 Subgrade Preparation and Embankment Construction at East Approach

East of the CNR overpass, the existing 10 m high approach embankments will be widened to the north and south by some 3 to 5 metres, at a slope of about 1.25 Horizontal to 1 Vertical (consistent with the existing side slopes).

Based on the borehole results, the subgrade soils within the expected depths of excavation will consist of existing embankment fill materials at each of the foundation elements, underlain by stiff weathered silty clay at the east pier and limestone bedrock at the west abutment. The area of new filling within 20 m of the east abutment is expected to be underlain by a thick deposit of silty clay, the upper 6 m of which is a stiff weathered crust (s_u >100 kPa) and the lower 1 to 4 m is firm to stiff (s_u >40 kPa). Beneath the silty clay is a thin veneer of till over bedrock consisting of limestone and arkosic sandstone. Any surficial topsoil, peat, organic matter and softened / loosened soils should be stripped from within the limits of the new approach embankment filling, including the existing embankment sideslope and the new footprint. All subgrade soils should be proof-rolled prior to fill placement.

Construction of the embankment above the prepared subgrade may be carried out using clean granular or rock fill (in accordance with OPSS 212), depending on material availability. At this site, we understand that rock fill generated from the widening of Highway 401 west of the CNR overpass will most likely be used as embankment fill. From a geotechnical/foundations perspective, this material is preferred for construction of the embankment widening as it will provide better compatibility with the existing embankment fill materials.

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010. It is likely that the new rock fill can be placed adjacent to the existing embankment fill without the need for special grading or separation layers between the new and existing materials. If the new rock fill proves to be significantly coarser than the existing rock fill or the pavement structure, the filter compatibility of the two materials will need to be assessed to limit the potential for migration of soil particles into voids of adjacent layers. If the materials are sufficiently dissimilar, there is potential for migration of finer particles which could result in settlement/sinkholes propagating to the ground surface and the surface of the rock fill layer will need to be carefully graded and “chinked” or a separation layer placed, before placing any granular fill for the pavement structure.

Rock fill embankments should be constructed and compacted in accordance with SP206S03. Earth fill or SSM embankments should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material’s standard Proctor maximum dry density in accordance with OPSS 539. The final lift prior to placement of the granular subbase or base courses should be compacted to 100 percent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

For semi-integral abutments, rock fill should not be placed within the active wedge zone. Rock fill contains numerous voids into which finer material can migrate due to water action and/or repeated loading. To limit potential settlement resulting from the migration of finer material, a filter material is required at the transition between the rock fill and the abutment backfill or other embankment fill. Granular B Type II (OPSS 1010) meets the criteria for filtration and drainage and therefore could be used as backfill to the abutment to transition between rock fill and earth fill embankment.

The permanent slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical (2H:1V) if constructed of granular fill or 1.25H:1V if constructed in rock fill. For soil slopes greater than 8 m in height and rock fill slopes greater than 10 m in height, MTO requires a 2 m wide mid-slope bench for maintenance purposes.

To reduce surface water erosion on the embankment, side slopes constructed of earth fill should either be protected with large diameter rock fill (rip-rap) or seeded and pegged with sod over topsoil

6.9.4 Approach Embankment and Bridge Retaining Wall Stability

With appropriate subgrade preparation and proper placement of granular or rock fill, the up to 10 m high east approach embankment with side slopes maintained at 1.25 horizontal to 1 vertical, founded on the native stiff weathered silty clay, will have a factor of safety greater than 1.3 against deep-seated slope instability.

Similarly, the proposed abutment retaining wall and adjacent wing walls, up to 3 m in height, founded on piled foundations, engineered fill or rock (after removal of the existing fill materials) at the top of the 2H:1V to 3H:1V abutment slope, will have a factor of safety greater than 1.3 against deep seated slope instability. Local stability may be improved with the use of geogrid reinforcements.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls and embankment side slopes will have factors of safety of greater than 1.1 against deep-seated slope instability based on a pseudo-static horizontal acceleration, kh, of 0.05g. The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. This sloughing would not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes or by placing rip rap.

The slope stability analyses for the above embankment and retaining wall configurations were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Rock Fill or Granular Embankment Fill	21	40 º	2(apparent cohesion)
Existing Embankment Fill	20	40 º	2(apparent cohesion)
Existing Fill along Railway	20	30 º	
Weathered Silty Clay	19.7		100
Silty Clay	17.4		40 - 25
Till	22	Impenetrable by failure surface	
Bedrock	24	Impenetrable by failure surface	

6.9.5 Approach Embankment Settlement

Settlement of the approach embankment extensions adjacent to the abutments will occur due to compression of the new embankment fill itself, as well as compression of the existing fill materials, and of the underlying silty clay at the east approach embankment. Settlements due to compression of the underlying glacial till should be negligible in magnitude.

Provided that the new embankment fill material consists of Select Subgrade Material or clean earth fill or rock fill, the settlement within the new and existing travelled lanes due to compression of the new embankment fill itself is expected to be less than about 25 mm. The use of granular fills for the new embankment construction would reduce the magnitude of *post-construction* settlement (likely to less than half that value), since the majority of the settlement of granular fills will occur during construction.

At the east approach embankment, the embankment fill materials will be (partially) underlain by the existing embankment fill materials that form the current foreslopes. These existing fill materials can generally be left in place beneath the embankment widening provided some modest settlement (i.e., less than 25 mm) of the subgrade can be tolerated. However the subgrade surface should be proof rolled and compacted to at least 95 percent of the standard Proctor maximum dry density. Additional subgrade settlements, in the order of 40 to 60 mm, can be expected due to consolidation of the 4 to 5 m of firm to stiff silty clay which is present below the embankment fills at the east abutment. The subgrade settlements resulting from compression of the silty clay are expected to occur within 4 to 6 months such that the post-construction settlements of the embankment surface would not be expected to noticeably exceed the compression of the embankment fill itself (i.e. 25 mm).

Given the proximity of bedrock to surface, the relative density of the existing fill and the absence of cohesive soils, subgrade settlements of the west embankment should be negligible.

The above settlements would occur predominantly within the area of new widening and, to a lesser extent, within existing portions of the roadway more than 10 m from the median. The settlements would be entirely differential relative to the existing east abutment structure (which would be supported on deep foundations on bedrock). These settlement values exceed the usual values acceptable by MTO for the approached to bridges, as shown in the following table:

Distance from Abutment	Tolerable Settlement
0 to 30 m	10 to 25 mm
30 to 70 m	25 to 50 mm
70 to 170 m	50 to 100 mm
Greater than 170 m	100 to 200 mm

These settlements will also result in downdrag loads on the new and existing east abutment piles.

The following mitigation options to result settlement at the east abutment have been considered:

1) Allow settlements to occur and periodically pad and overlay to correct the profile;
2) Pre-load the widening to minimize post-construction settlement
3) Use lightweight fill materials to construct portions of the widening nearest to the abutment;
4) Subexcavate the silty clay to remove the settlement-sensitive material; and,
5) Lowering of the profile grade.

The first three options above are considered feasible and recommendations are provided in the following sections. Subexcavation of the silty clay is not considered feasible considering the thickness and depth of the deposit and its location beneath the existing embankment. Lowering of the profile grade is also not considered feasible due to geometric constraints and existing site features. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the feasible options is provided in Table 5. If downdrag loads on the existing piles are considered tolerable, preloading of the east approach before the installation of the new east abutment piles is considered to be the preferred option of mitigating settlement in the immediate area of the bridge abutments and eliminating downdrag on the new piles. If the downdrag loads on the outer existing east abutment piles exceed structural limits, EPS fill is considered to be the preferred option of mitigating settlement in the immediate area of the bridge abutments and thus downdrag loads on the existing piles.

Option 1 – **Allow settlement to occur**. This option would involve allowing the roadway to settle and to accept the short-term potential impacts of the expected settlements on the roadway performance. It would be planned to pad and overlay the roadway periodically to reinstate the roadway profile. This option is considered to be technically feasible but is probably not appropriate considering the high volumes of traffic along this section of the highway. Both new and existing piles would need to be designed to accommodate the induced downdrag loads.

Option 2 – Pre-Loading. For this option, the footprint of the widening would be preloaded with fill to the final embankment height and allowed to settle in advance of the new piles being installed and the new lanes being paved and put in-service. As described above, it will require some 4 to 6 months for the excess pore water pressure to dissipate and the settlements to occur. Pre-loading could eliminate downdrag loads on new piles (if they were installed through the fill following completion of the pre-loading), but would not reduce the impacts of downdrag on the existing east abutment piles. As such, if downdrag loads on the existing piles resulting from the widening of the embankment are too high; pre-loading should only be carried out away from the existing piles. Also, it may not be feasible to fully preload the area immediately adjacent to the abutment without constructing some form of flexible retained soil system which would need to be used and disposed of at the completion of the preload time.

If the required preload times discussed above are not available, consideration could be given to surcharging or installation of wick drains. However, given the height of the embankment, the depth of the compressible silty clay layer, and the available space for surcharging, these options are not preferred.

Option 3 – Light weight Fill. Light weight fill materials (such as expanded polystyrene EPS 'geofoam' fill) could be used for portions of the embankment widening to reduce the increase in load in the compressible clay deposit such that the embankment subgrade settlements would be within acceptable tolerances. This option is particularly relevant in minimizing downdrag loads on existing east abutment piles, where preloading does not eliminate settlement (and thus downdrag loads) acting on these piles. Other light weight fill materials have been considered (i.e. blast furnace slag, tire derived aggregate or cellular/foamed concrete); however, it is considered that the unit weights of these materials are not sufficiently low to achieve the necessary reductions in final stress level.

To achieve the necessary reduction of stress increase and meet the previously described tolerable settlements at the existing east abutment piles, the following EPS thicknesses would be required:

East Approach Embankment Widening (eastbound and westbound lanes):

- Abutment to 10 m back: 2.4 m thick; and
- 10 m to 20 m back: 1.2 m thick.

A Non-Standard Special Provision for the supply and installation of EPS fill should be included in the contract documents and a sample has been provided in Appendix B of this report.

In general, the widened section of the roadway containing EPS fill would comprise, from bottom to top:

- a 0.3 metre thick layer of OPSS Granular A as a levelling pad beneath the EPS Geofoam, covered with up to 100 mm of mortar sand.
- up to 2.4 m thick Geofoam block(s) (e.g. EPS22 in accordance with ASTM D6817-02), having a compressive strength at 5% strain of at least 115 kilopascals.
- a cover of polyethylene sheeting on the outside surface of the EPS and guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision). The EPS is potentially soluble in hydrocarbons.

- a 125 mm thick protective concrete slab on the surface of the EPS Geofoam blocks at pavement subgrade level to distribute wheel loads and protect the Geofoam from overstressing, which could lead to rutting of the pavement surface.
- A minimum of 800 mm pavement granular thickness (granular base plus subbase) to limit the potential for premature icing of the roadway due to the insulating properties of the Geofoam.

The EPS blocks should extend for the full width of widening, from the existing edge of granular to within 1 m of the widened side slope.

6.10 Design and Construction Considerations

6.10.1 Excavations

The excavations for the construction of abutments or cast-in-place retaining wall foundations will extend through existing and/or new fill materials at all abutment and pier locations, potentially into weathered crust at the east and west piers, and potentially into bedrock at the west abutment. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

The soils are classified as Type 3 soils according to the OHSA and therefore temporary excavations could be made with unsupported cut with side slopes no steeper than 1 horizontal to 1 vertical. Temporary excavations for footing construction extending through the bedrock (e.g. west abutment) may be completed using near vertical sides. For pier footing construction, temporary excavation support may be required due to rail track protection requirements and space restrictions. Based on the GA provided and an assumed base of pile cap elevation of 74.4 m, excavations at the east and west piers could extend up to 5 m below the existing grade in places, and 3.5 m below track level.

Roadway protection, installed parallel to the highway alignment and located within the existing embankment foreslopes, will likely be required to accommodate the construction staging. Track protection will also likely be required to accommodate construction of the new east and west pier foundations. Both should be included in the tender documents, as per current MTO specifications.

6.10.2 Temporary Excavation Shoring

Temporary roadway protection will likely be required to accommodate the construction staging at the east and west abutments. The roadway protection should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

Temporary track protection will be required for construction of the pile caps at the east and west piers. Due to the proximity of the rail tracks (particularly at the east pier), there is likely insufficient distance to satisfy the 1 horizontal to 1 vertical side slopes requirement for temporary excavations, and therefore a support system will be required to minimize the movement of the railroad track and to maintain the stability of the existing highway embankment. The support system at the east pier would be located about 2.5 m or less from the outside rail of the nearest track to the existing piers at the toe of the existing highway embankment. At the west pier, the location of the track protection is more flexible, as there is a 6.8 m offset between the existing piers and the closest railway track.

Based on previous experience with excavations adjacent to railways, it is recommended that the lateral movement of the temporary shoring system adjacent to the CNR tracks meet Performance Level 1b as specified in OPSS 539. The shoring design should also meet the requirements set out in the American Railway Engineering and Maintenance of Way (AREMA) Manual for Railway Engineering.

The shoring method(s) chosen to support the excavation sides for the east and west pier pile caps must take into account: the soil stratigraphy, the groundwater conditions, the methods adopted to manage the groundwater, the permissible ground movements associated with the excavation and construction of the shoring system, and the potential impacts on adjacent structures. In general, there are three basic shoring methods that are commonly used in this area:

- Steel soldier piles and timber lagging;
- Driven steel sheet piles; and,
- Continuous concrete (secant or diaphragm) walls, though much less commonly used than the other two systems.

These three options are listed in order of generally increasing stiffness and ability to resist ground movements. Soldier piles and lagging are suitable where the objective is to maintain an essentially vertical excavation wall and the movements above and behind the wall need only be sufficiently limited so that relatively flexible features (such as roadways) will not be adversely affected. Where the deflections need to be more strictly limited, such as where heavily loaded foundations lie within the zone of influence of the shoring, continuous concrete shoring can be required. Sheet piling provides an intermediate level of stiffness.

For all of the above systems, some form of lateral support to the shoring is required for excavation depths greater than about 3 m. Lateral restraint can be provided by interior struts connected to either side of the excavation (if not too distant) or by means of rakers to either piles and/or footings within the excavation. Lateral support could also be provided by means of tie-backs consisting of either soil anchors or grouted bedrock anchors, or by means of tie-rods to deadman anchors.

At the east pier, where the existing outside rail of the CNR track is about 2.8 to 2.9 m from the edge of the existing piers, excavations are expected to extend though variable fill deposits (including some rock fill) overlying stiff to very stiff weathered silty clay at the base of the excavation at an elevation of 74.3 to 74.8 m. While soldier pile and timber lagging is technically feasible to install at the east pier, the system is not considered sufficiently stiff to maintain horizontal displacements at less than 10 mm as specified in MTO’s OPSS 539 and as required by the railway. A continuous concrete shoring system would also be technically feasible to install, but would be very costly and likely unnecessary for this project. At this location steel sheet piling, comprised on an internally braced box for each pier site, is considered most suitable and is the recommended temporary shoring protection system for construction of the pile caps. Note: Sheet piles may be unable to penetrate boulders in the near-surface rock fill at some locations and provision should be made for some excavation of the fill material. The contractor should be alerted to this issue. An NSSP could be included in the contract to address this issue and a sample has been included in Appendix B for reference. Because the existing piers are supported on piles, it is not anticipated that temporary excavation adjacent to the pile caps will compromise their ability to support the existing piers.

At the west pier, where the existing piers are about 6.8 from the edge of the existing CNR track, excavations are expected to extend though variable fill deposits (predominantly rock fill, especially closer to the existing toe of slope) with the base of the excavation within or just above a thin layer of stiff to very stiff weathered silty clay. Beneath the weathered crust are limestone slabs which were encountered at 73.1 to 73.4 m elevation at boreholes put down west of the existing west piers. The top of the rock slabs is expected to drop off in elevation to the east. It is not considered feasible to drive sheet piles into the limestone slabs beneath the weathered crust and the prevalence of rock fill at the west pier will make advance of sheet piles difficult. Given the subsurface conditions and the increased offset between the west pier foundations and the existing track, it may be more feasible to install soldier pile and lagging rather than sheet piling as soldier piles can be more readily advanced though obstructions in the fill and or limestone slabs and will provide an acceptable level of stiffness. Above the water table, the temporary excavation for the west piers may be cut at 1 horizontal to 1 vertical.

Roadway protection could conceivably consist of either steel sheet piling or soldier piles and lagging.

To the expected depths of excavation, it is not expected that basal heaving or basal instability will be a concern.

For the rail track protection, the potential for interference with the existing pier foundations (or the rail track protection from the original construction) and the bridge structure overhead will need to be evaluated.

6.10.2.1 *Lateral Earth Pressures for Shoring Design*

All temporary shoring should be designed to resist for lateral earth pressures resulting from the weight of the restrained earth and other dead and surcharge loads (including rail and construction traffic, equipment, or stockpiled materials). The earth pressure distribution used for shoring design is dependent upon the rigidity of the specific wall design and on the nature of the lateral support provided. The method of lateral restraint should be selected by the contractor since it potentially impacts on the construction logistics as well as the construction schedule.

At the west pier, the depth of shoring will be limited by the presence of limestone slabs and it is considered that lateral support be provided by means of internal bracing or rakers. At the east pier, additional lateral support may be realized from passive toe restraint in the underlying weathered silty clay crust. For roadway protection, lateral restraint could be provided by means of either rakers supported on footings or piles within the excavation or using tie-backs grouted into the soil or bedrock or fixed to dead-men behind the shoring. Cantilevering of the shoring might also be feasible, provided the retained height is no more than about 3 m.

Assuming the excavations are made predominantly in granular fill materials, strutted shoring walls should be designed to resist a rectangular earth pressure distribution. The unfactored rectangular earth pressure distribution (p in kN/m^2; constant with depth), can be calculated as follows:

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

where: K_a = 0.36 for level ground behind the excavation wall; K_a must be adjusted if there is sloping ground behind the excavation wall;

γ = soil unit weight, as given in the following table;

H = total height of the excavation;

q = surcharge at ground surface to account for construction traffic, equipment, or stock piled material;

U =hydrostatic pressure; and,
p_s =horizontal unit pressure on the wall due to the strip load (rail) loading.

Shoring walls that are not laterally supported, or that are supported using soil anchors or rakers, should be designed to resist a triangular earth pressure distribution. The unfactored triangular earth pressure distribution (P in kN/m^2; increasing with depth), can be calculated as follows:

$$P = K_a [(\gamma (H-h_w) + (\gamma-\gamma_w)h_w + q)] + p_s$$

where: K_a = 0.36 for level ground behind the excavation wall; K_a must be adjusted if there is sloping ground behind the excavation wall;
γ = soil unit weight, as given in the following table;
γ_w = 9.81 kN/m^3, unit weight of water;
H = total height of the excavation;
h_w = the height of the groundwater level above the base of the excavation;
q = surcharge at ground surface to account for construction traffic, equipment, or stock piled material; and,
p_s =horizontal unit pressure on the wall due to the strip load (rail) loading.

For the east and west pier excavations, the horizontal component of the rail loading will need to be added to the above earth pressures. As set out in the American Railway Engineering and Maintenance of Way (AREMA) Manual for Railway Engineering (Section 20.3.2.2), the following expression may be used to compute the intensity of horizontal unit pressure on the wall due to a uniform strip load parallel to the shoring system:

$$p_s = (2q/\pi) \cdot (\beta + sin\beta sin^2\alpha - sin\beta cos^2\alpha)$$

where: p_s is the intensity of horizontal unit pressure on the wall at any given point due to a continuous strip of surcharge load parallel to the shoring system;
q is the magnitude of the strip load per unit length (the train load distributed over a 2.4 m wide railway tie);
α is the angle between the vertical excavation and the midpoint of the strip load (in radians); and,
β is the angle between the near and far edges of the strip load (in radians).

The above expression for strip loading is a modified Boussinesq solution developed by Scott (1963) based on empirical data. This equation is valid for rigid systems such as the proposed track protection systems at the east and west piers. The lateral pressures computed are roughly double the values which would be obtained by elastic equations.

Alternatively, very narrow strip surcharge loads may be considered as line loads. The intensity of lateral pressure at depth from a line load may be computed based on the semi-empirical formulas presented in Section 20.3.2.3 of AREMA. These formulas, based on the work by Terzaghi and outlined in NAVFAC (1982) also assume an unyielding rigid wall.

The following table provides soil unit weights to be used in the above lateral earth pressure equations. For soldier pile and lagging installations, it is expected that the excavation will be fully drained, and bulk unit weights with no hydrostatic pressures should be used. If an interlocking sheet pile wall is adopted and dewatering of the surficial fill is not required, the shoring walls should be designed using effective unit weights ($\gamma - \gamma_w$) below the water table and should include a triangular water pressure distribution (U), with the design groundwater level taken at a depth of 1 m below the ground surface. However, the adequacy of all shoring designs should be checked for a water level at the ground surface (in the event of a failure of the dewatering system during a period of high groundwater) and for the fully drained case (in the event of a period of low groundwater or prolonged dewatering).

Soil Unit	Bulk Unit Weight (γ)
Surficial Fills	20 kN/m^3
Weathered Silty Clay Crust	19.7 kN/m^3

Passive toe restraint to the protection system should be determined using a triangular pressure distribution. The coefficient of passive lateral earth pressure, K_p, and the bulk unit weight, γ, for the soil in front of the piles may be taken as follows:

Soil Unit	K_p	Bulk Unit Weight (γ)
Fine Rock Fill	3.7	21
Weathered Silty Clay (approx. elev. 74.5 to 72m at east pier)	2.8	19.7
Lower Silty Clay (below elev. 72 m)	2.9	17.4

6.10.2.2 *Vibration Monitoring During Installation of Temporary Shoring Protection*

Vibration monitoring should be carried out during pier pile installation for the structure widening, to ensure that vibration levels at the existing piers and abutments are maintained below tolerable levels. As outlined in Section 6.4, a maximum peak particle velocity (PPV) of 100 mm/s is recommended at the existing abutments and east pier. At the west pier, where there is a significant risk of damage to the existing shallow spread footing foundations, maximum peak particle velocities should be limited to 50 mm/s to minimize vibrations of foundations potentially supported on rock slabs. A Non Standard Special Provision for vibration monitoring should be included in the contract documents and a sample has been included in Appendix B of this report.

6.10.3 Groundwater and Surface Water Control

The groundwater level is considered to be in the range of elevation 76 to 77 m. Given that the abutment foundation/pile cap elevations at the east and west abutments are well above the measured groundwater level, only a modest amount of groundwater/perched water flow is expected at these locations and we anticipate that groundwater inflow can be adequately controlled through the use of pumping from properly filtered sumps in the excavations. Excavations for the east and west pier foundations are expected to extend below the groundwater level and, as such, groundwater inflow to the excavation should be expected, particularly within the fill deposits below the water table. Provided that pier foundation excavations are shored with sheet piling or similar temporary structures which would control groundwater-induced ground loss, it is expected that groundwater inflow can be adequately controlled though the use of pumping from properly filtered sumps in the excavations. If required, subexcavation of peat and organic material at the east approach embankments could be carried out subaqueously.

Surficial drainage may be also required around the perimeter of the excavation due to the interference of the foundation excavations with the existing drainage ditches and pipes.

7.0 CLOSURE

This report was prepared by Ms. Erin S. O'Neill, P. Eng., under the direction of the Project Manager, Mr. Michael Snow, P.Eng. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

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Table 1 - Evaluation of Bridge Foundations/Construction Alternatives
West Abutment
Highway 401 Bridge Rehabilitation/Widening Over CNR Line
G.W.P. 78-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on bedrock	Preferred option for west abutment	▪ Relatively simple construction ▪ Compatible with existing west abutment foundations ▪ Negligible settlement ▪ Semi-integral design feasible	▪ May require temporary support of adjacent roadway during construction ▪ May need to be changed to piled foundation if bedrock is deeper than expected (i.e., north side, west abutment)	Likely least expensive option	▪ Low risk option ▪ Possibility of deeper than expected bedrock surface at north end of west abutment – may need to be piled ▪ Possible additional subexcavation needed, due to fractured bedrock near surface
Steel H-piles	Feasible for support of west abutment	▪ High bearing resistance ▪ Negligible settlement ▪ Semi-integral design is feasible	▪ Depth to bearing is shallow. May require overexcavation of bedrock to achieve minimum length for pile design.	Less expensive than rock-socketed caisson option	▪ Risk of having to pre-drill or drive additional piles ▪ Risk of damage to piles due to rock fill
Cast-in-place concrete caissons founded on or socketed nominally into rock	Feasible for support of west abutment	▪ High bearing resistance ▪ Negligible settlement ▪ Semi-integral design feasible	▪ Permanent casings required to construct caissons ▪ High likelihood of encountering rock fill during drilled shaft installation ▪ If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock	Most expensive option	▪ Risk of construction difficulties due to rock fill ▪ Vibrations induced by churn drilling may impact existing foundations

Table 2 - Evaluation of Bridge Foundations/Construction Alternatives
West Pier
Highway 401 Bridge Rehabilitation/Widening Over CNR Line
G.W.P. 78-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Spread footings supported on bedrock	Feasible for support of lightly loaded west pier foundations, but not recommended	▪ Relatively simple construction ▪ Compatible with reported existing west pier foundations ▪ Semi-integral design feasible	▪ Will require temporary support of adjacent railway tracks during construction ▪ Very low ULS and SLS bearing capacities on rock slabs separated by voids and/or loose soil ▪ Significant risk of differential settlement relative to existing pier foundations ▪ May require grouting to infill voids	Likely least expensive, although shoring and grouting would increase costs	▪ high risk option ▪ may need grouting or additional excavation ▪ differential settlement due to squeezing of voids/ soil infills
Pre-drilled Steel H-piles	Feasible for support of west pier	▪ High bearing resistance ▪ Allows for penetration of rock slabs into firm bearing stratum ▪ Negligible settlement if driven to bedrock	▪ Drilling through rock slabs to underlying bedrock required at each pile location with a large diameter hole ▪ Drilling may induce some vibrations that affect the existing pier foundations ▪ Not consistent with existing bridge or abutment foundations ▪ May require a specialist contractor	Less expensive than rock-socketed caisson and about the same as drilled pile option.	▪ Moderate risk option ▪ Induced vibrations during drilling and pile installation may adversely impact existing foundations

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Drilled pipe piles socketed into bedrock	Feasible for support of west pier	▪ High bearing resistance. ▪ Negligible settlement. ▪ Allows for penetration of rock slabs into firm bearing stratum ▪ One step construction – casing is the pile, no additional central reinforcement (H-pile or bars) required ▪ Smaller diameter hole than pre-drilled steel H-piles will induce less vibration	▪ Drilling through rock slabs to underlying bedrock required at each pile location ▪ Drilling may induce some vibrations that affect the existing pier foundations ▪ Not consistent with existing bridge or abutment foundations ▪ Difficult to ensure clean contact at the base of the pile for geotechnical capacity ▪ Requires specialist contractor	Moderately expensive	▪ Moderate risk option ▪ Induced vibrations during drilling and pile installation may adversely impact existing foundations
Drilled Micro-piles	Recommended option for west pier	▪ High bearing resistance ▪ Negligible settlement ▪ Allows for penetration of rock slabs to rock bearing stratum ▪ Reduced vibration during construction, minimizes potential impact on existing bridge foundations ▪ Smaller equipment footprint, less impact on CNR activities	▪ Drilling through rock slabs into underlying bedrock required at each pile location ▪ Not consistent with existing bridge or abutment foundations ▪ Requires specialist contractor	More expensive than drilled piles	Low risk Option

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Cast-in-place concrete caissons founded on or socketed nominally into rock	Feasible for support of west pier	▪ High bearing resistance ▪ Negligible settlement	▪ Permanent casings required to construct caissons ▪ High likelihood of encountering rock slabs during drilled shaft installation ▪ If rock socket required, extensive coring or churn drilling will be required to form socket in medium strong bedrock beneath rock slabs ▪ Cleanout of base needed for end-bearing may be difficult	Most expensive option	▪ Risk of construction difficulties due to rock slabs ▪ Vibrations induced by churn drilling may adversely impact existing foundations

Table 3 - Evaluation of New Bridge Foundations/Construction Alternatives
East Pier
Highway 401 Bridge Rehabilitation/Widening Over CNR Line
G.W.P. 78-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Steel H-piles driven to bedrock	Preferred option for east pier	▪ High bearing resistance ▪ Negligible settlement ▪ Ease of installation ▪ Compatible with existing foundation elements at east abutment ▪ Preferred option for east pier	▪ Proximity of railway may limit ability to install temporary excavation shoring without interfering with railway operations ▪ Possibility of encountering cobbles or boulders in embankment fill during pile driving may require some pre-augering	Less expensive than caissons	▪ Risk of damage to piles due to boulders in rock fill ▪ Risk of having to pre-auger for some piles through surface fills
Cast-in-place concrete caissons founded on or socketed nominally into bedrock	Feasible for support of east pier	▪ High bearing resistance ▪ Negligible settlement ▪ Minimal excavation required adjacent to rail	▪ Permanent casings required to construct caissons ▪ Depth of bearing stratum (approx. 12 – 14 m) may be cost-prohibitive ▪ Possibility of encountering cobbles or boulders in rock fill or in till at bottom 1-2 m of caisson depth ▪ If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock ▪ Cleanout of base needed for end-bearing may be difficult	More expensive than driven piles	▪ Risk of construction difficulties due to boulders in rock fill near surface

Table 4 - Evaluation of New Bridge Foundations/Construction Alternatives
East Abutment
Highway 401 Bridge Rehabilitation/Widening Over CNR Line
G.W.P. 78-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Steel H-piles driven to bedrock	Preferred option for east abutment	▪ High bearing resistance ▪ Negligible settlement ▪ Ease of installation ▪ Compatible with existing foundation elements at east abutment ▪ Integral or semi-integral design is feasible	▪ Negative skin friction ("down drag") loads must be considered in design ▪ Care must be taken with driving of battered piles to ensure that the piles do not deflect along the bedrock surface ▪ Possibility of encountering cobbles or boulders in embankment fill during pile driving	Less expensive than caissons	▪ Risk of damage to piles due to boulders in rock fill or till ▪ Risk of having to pre-drill or drive additional piles
Cast-in-place concrete caissons founded on or socketed nominally into till/bedrock	Feasible for support of east abutment	▪ High bearing resistance ▪ Negligible settlement	▪ Permanent casings required to construct caissons ▪ Negative skin friction ("down drag") loads must be considered in design ▪ Depth of bearing stratum (approx. 22 – 23 m) may be cost-prohibitive ▪ Possibility of encountering cobbles or boulders in rock fill or in till at bottom 1-2 m of caisson ▪ If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock ▪ Cleanout of base needed for end-bearing may be difficult	More expensive option	▪ Risk of construction difficulties due to boulders in rock fill ▪ Possible ground loss associated with liner installation and socket construction

Table 5 - Evaluation of Settlement Mitigation Alternatives
East Abutment
Highway 401 Bridge Rehabilitation/Widening Over CNR Line
G.W.P. 78-99-00

Mitigation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Allow settlement to occur, with periodic pad/overlay	Likely not feasible given high traffic volumes	▪ No impact on construction schedule	▪ Requires post-construction maintenance ▪ Possible interim safety issue, between overlays, due to settlement	Relatively low costs, but must consider post-construction maintenance costs	▪ Excessive roadway settlement in short term
Pre-Loading	Feasible to reduce downdrag on new east abutment piles and reduce post-construction settlement	▪ No post-construction maintenance required ▪ Eliminates downdrag on new east abutment piles	▪ Delays paving ▪ Does not eliminate downdrag on existing east abutment foundation piles ▪ May require construction and removal of a temporary retained soil system near abutment ▪ Settlement monitoring required	Less expensive than lightweight fill	▪ Some uncertainly about schedule, since roadway construction cannot be completed until monitoring indicates sufficient settlement has occurred
Light Weight Fill	Feasible if downdrag loads on existing east abutment piles cannot be accommodated structurally	▪ No post-construction maintenance of highway ▪ Minimal impact to construction schedule for bridge foundations or widened embankment ▪ Eliminates downdrag on new and existing east abutment piles	▪ Expensive	Expensive	▪ Low risk option, but contractor may successfully propose another option as change order

MINISTRY OF TRANSPORTATION, ONTARIO

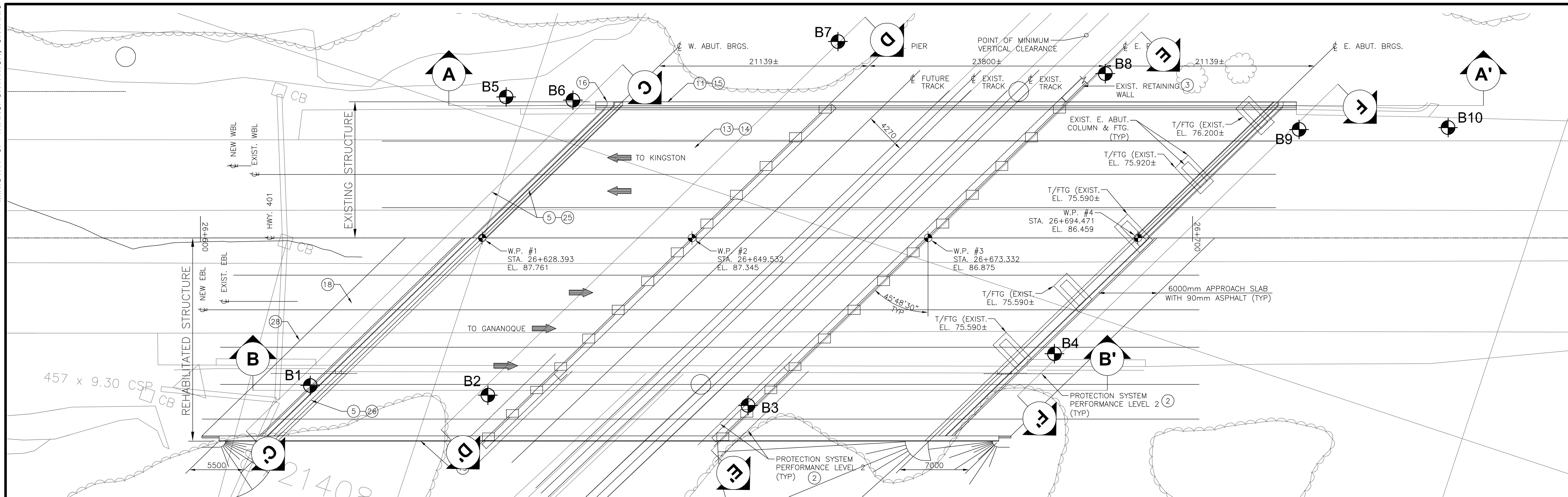
CONT No.
WP No. 78–99–01

SHEET

C.N.R. OVERHEAD
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates

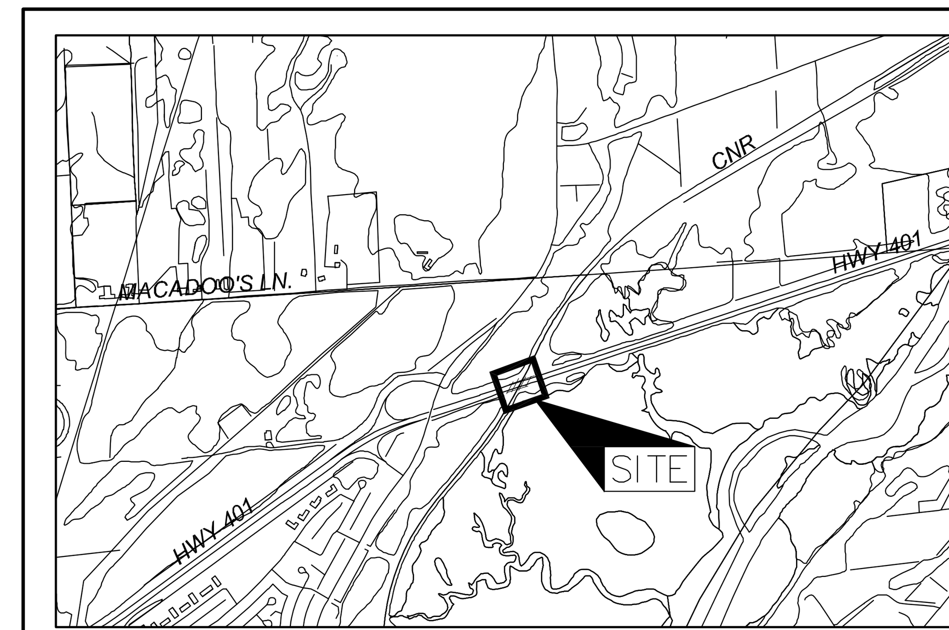
Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN

SCALE

5 0 5 10 m



KEY PLAN

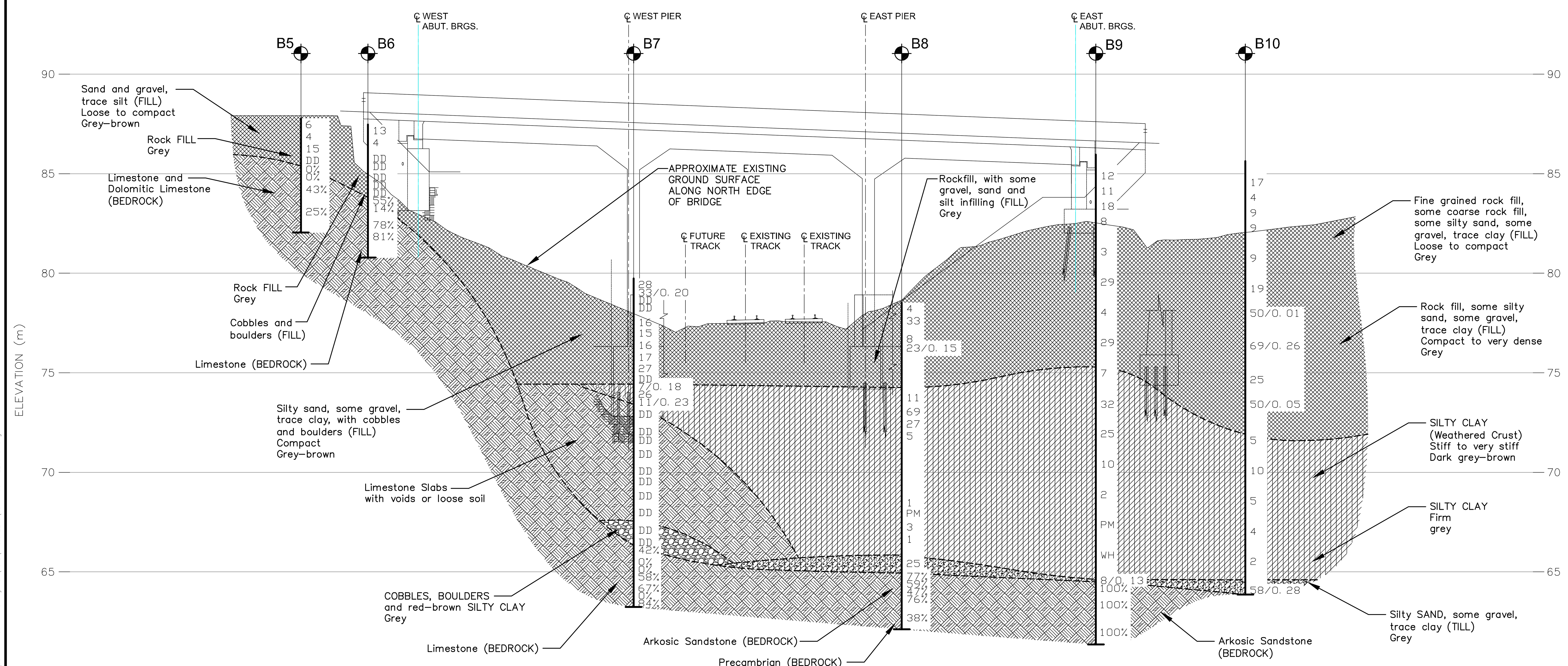
SCALE

2 0 1 1 km

LEGEND

(symbol)	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock quality designation
	Seal
	Piezometer

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
B1	87.6	4904268.6	307188.1
B2	80.4	4904273.3	307205.4
B3	77.3	4904280.6	307230.6
B4	86.4	4904295.2	307258.3
B5	87.8	4904302.4	307197.7
B6	87.5	4904304.2	307204.2
B7	79.7	4904318.2	307227.7
B8	78.5	4904323.6	307254.3
B9	86.0	4904324.4	307274.6
B10	85.6	4904329.3	307288.8



PROFILE A–A'

HORIZONTAL SCALE

5 0 5 10 m

2.5 0 2.5 5 m

VERTICAL SCALE

NOTES

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REFERENCE

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METRIC

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

NO.	DATE	BY	REVISION

Geocres No. 31C–202

HWY. 401		PROJECT NO. 08–1111–0044	DIST.
SUBM'D. EO	CHKD. MSS	DATE: 1/20/2011	SITE:
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

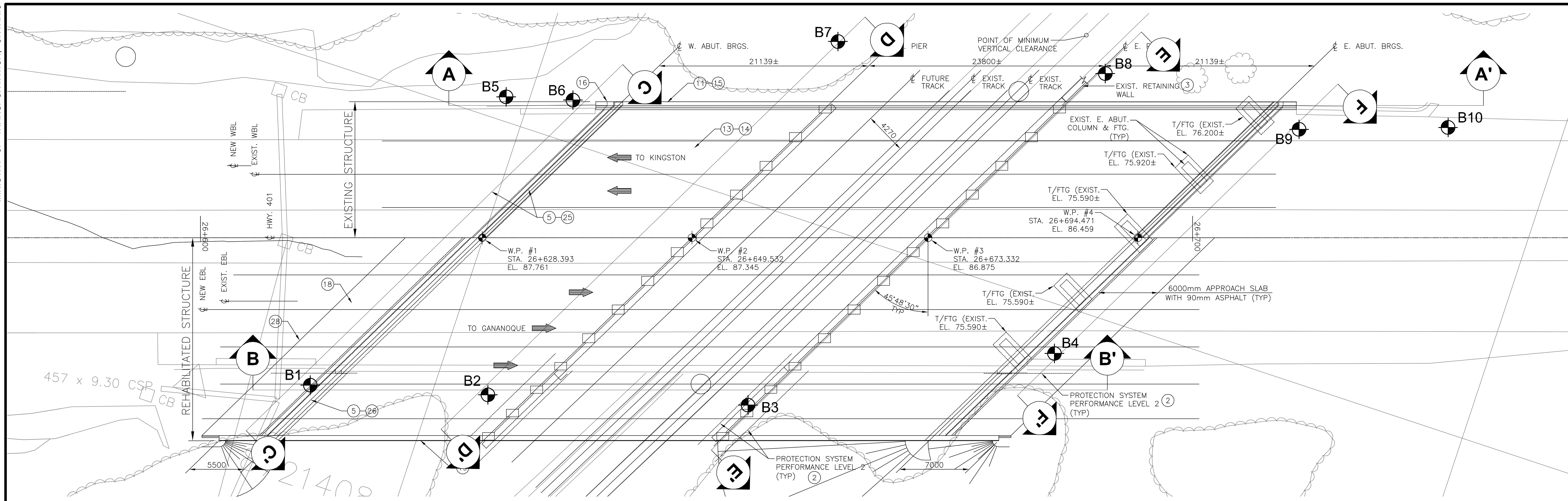
CONT No.
WP No. 78-99-01

SHEET

C.N.R. OVERHEAD
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA

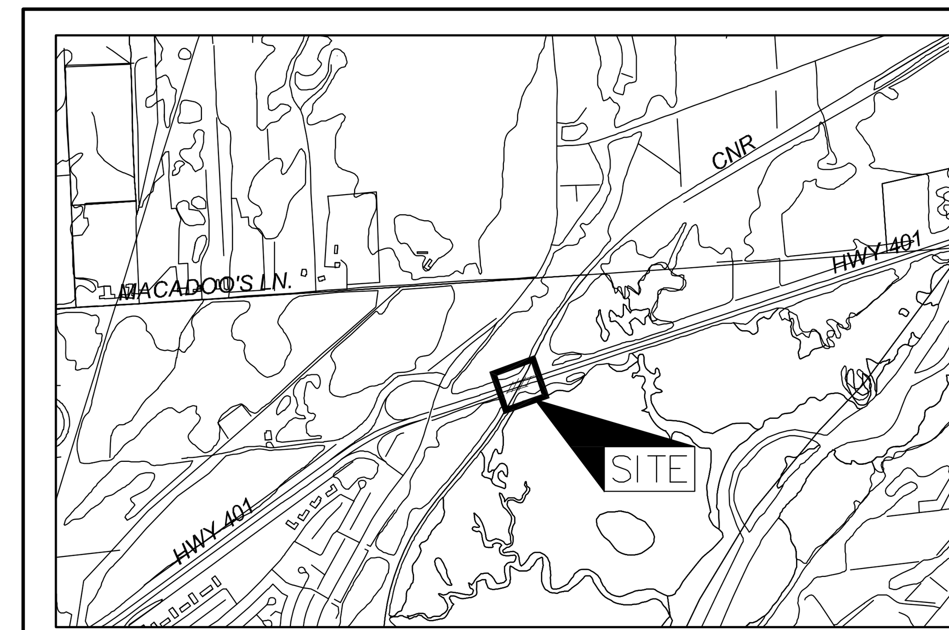
Golder Associates

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN

SCALE
5 0 5 10 m



KEY PLAN

SCALE
2 0 1 1 km

LEGEND

⊕	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock quality designation
	Seal
	Piezometer
▼	WL in piezometer on Sept. 29, 2009

No.	ELEVATION	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
B1	87.6	4904268.6	307188.1
B2	80.4	4904273.3	307205.4
B3	77.3	4904280.6	307230.6
B4	86.4	4904295.2	307258.3
B5	87.8	4904302.4	307197.7
B6	87.5	4904304.2	307204.2
B7	79.7	4904318.2	307227.7
B8	78.5	4904323.6	307254.3
B9	86.0	4904324.4	307274.6
B10	85.6	4904329.3	307288.8

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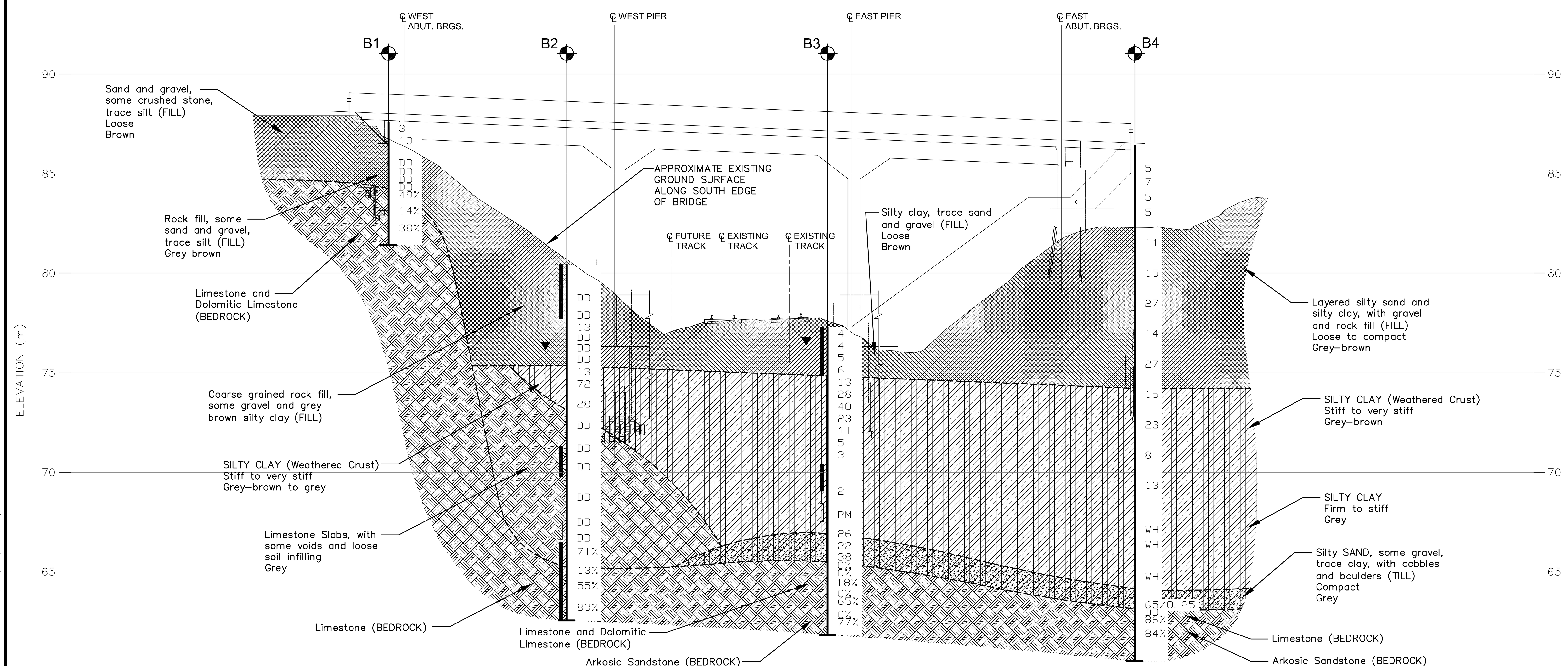


PROFILE B-B'

HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE

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

NO.	DATE	BY	REVISION

Geocres No. 31C-202

HWY. 401		PROJECT NO. 08-1111-0044	DIST.
SUBM'D. EO	CHKD. MSS	DATE: 1/20/2011	SITE:
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 2

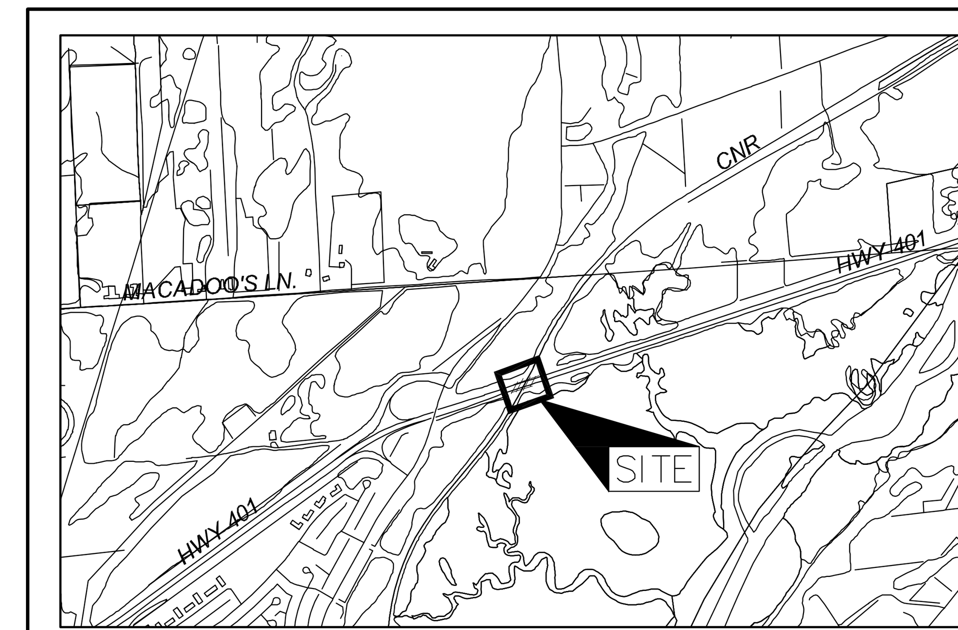
PLOT DATE: January 20, 2011

CONT No.
WP No. 78-99-01

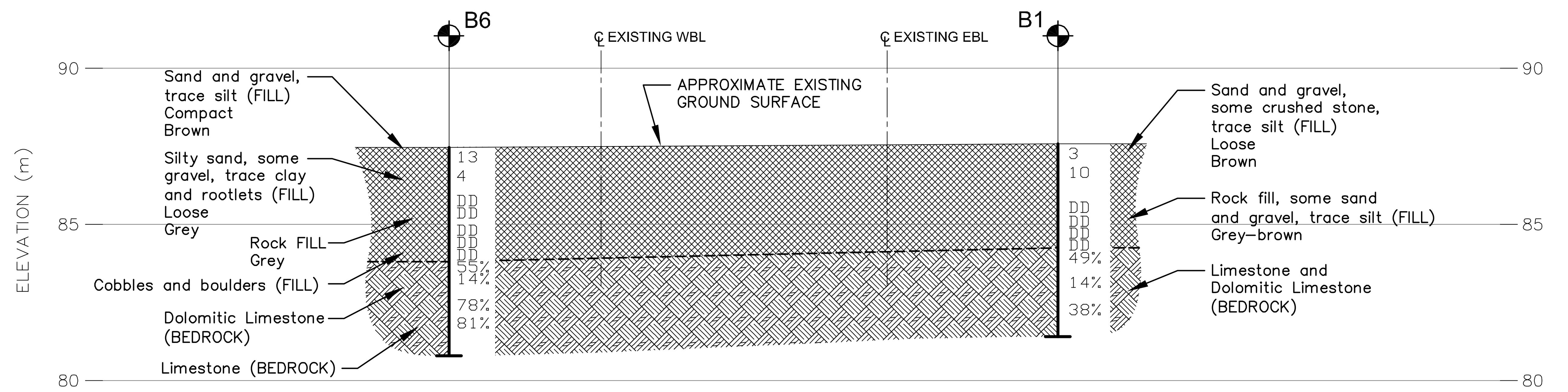
SHEET

C.N.R. OVERHEAD
HIGHWAY 401
SOIL STRATA

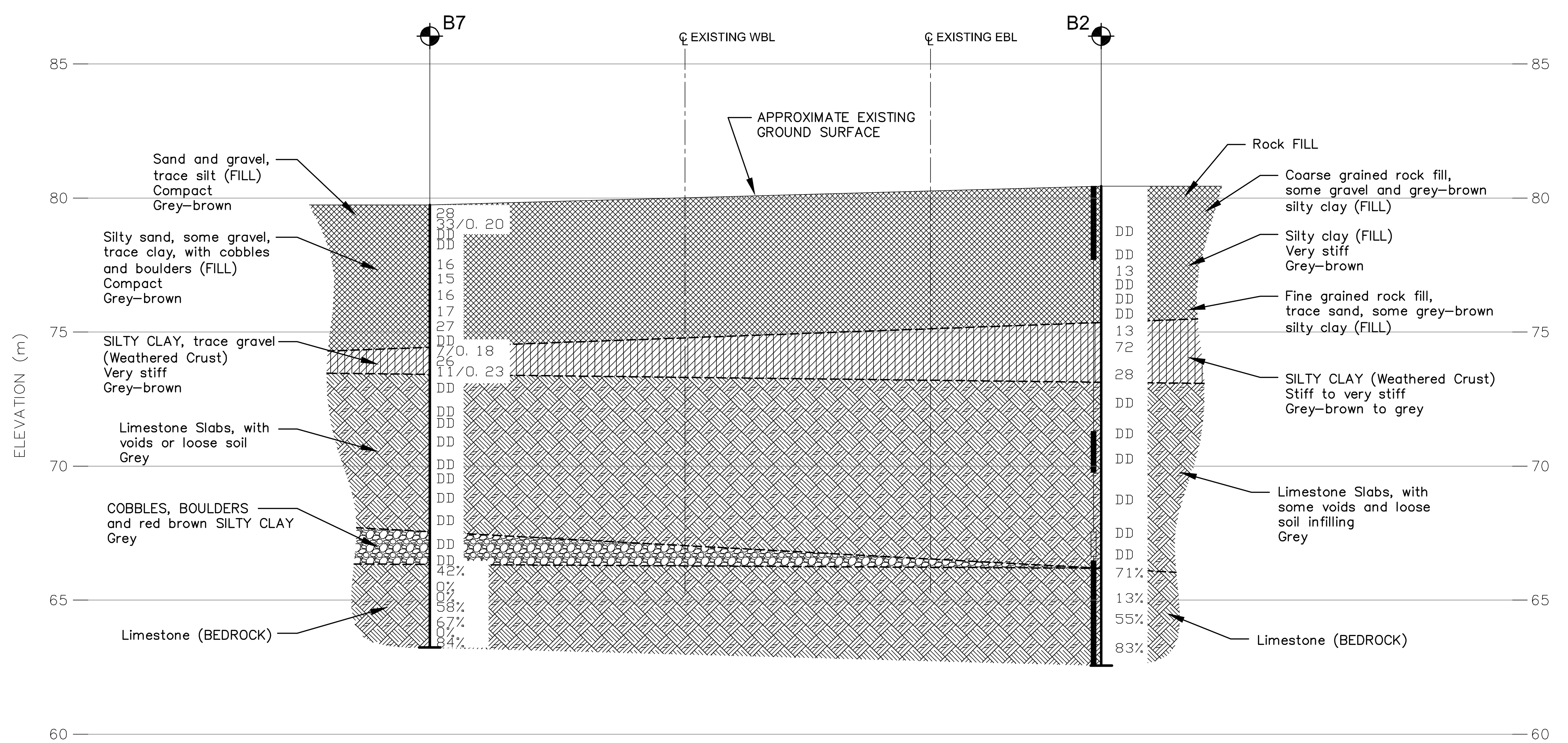
Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN
SCALE
2 0 1 1 km



PROFILE C–C'
HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE



PROFILE D–D'
HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE

LEGEND

⊕	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock quality designation
	Seal
	Piezometer
▼	WL in piezometer on Sept. 29, 2009

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
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B2	80.4	4904273.3	307205.4
B3	77.3	4904280.6	307230.6
B4	86.4	4904295.2	307258.3
B5	87.8	4904302.4	307197.7
B6	87.5	4904304.2	307204.2
B7	79.7	4904318.2	307227.7
B8	78.5	4904323.6	307254.3
B9	86.0	4904324.4	307274.6
B10	85.6	4904329.3	307288.8

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NO.	DATE	BY	REVISION

Geocres No. 31C-202

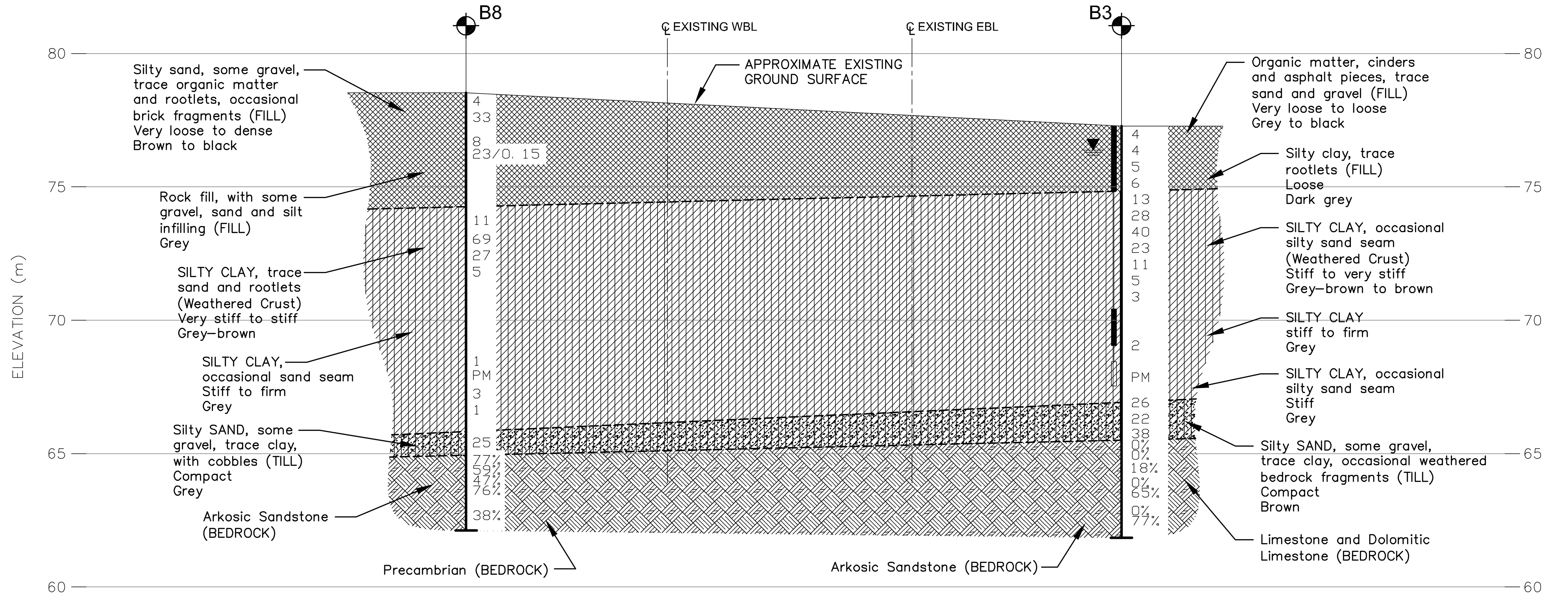
HWY. 401		PROJECT NO. 08-1111-0044	DIST.
SUBM'D. EO	CHKD. MSS	DATE: 1/20/2011	SITE:
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 3

CONT No.
WP No. 78-99-01

C.N.R. OVERHEAD
HIGHWAY 401
SOIL STRATA

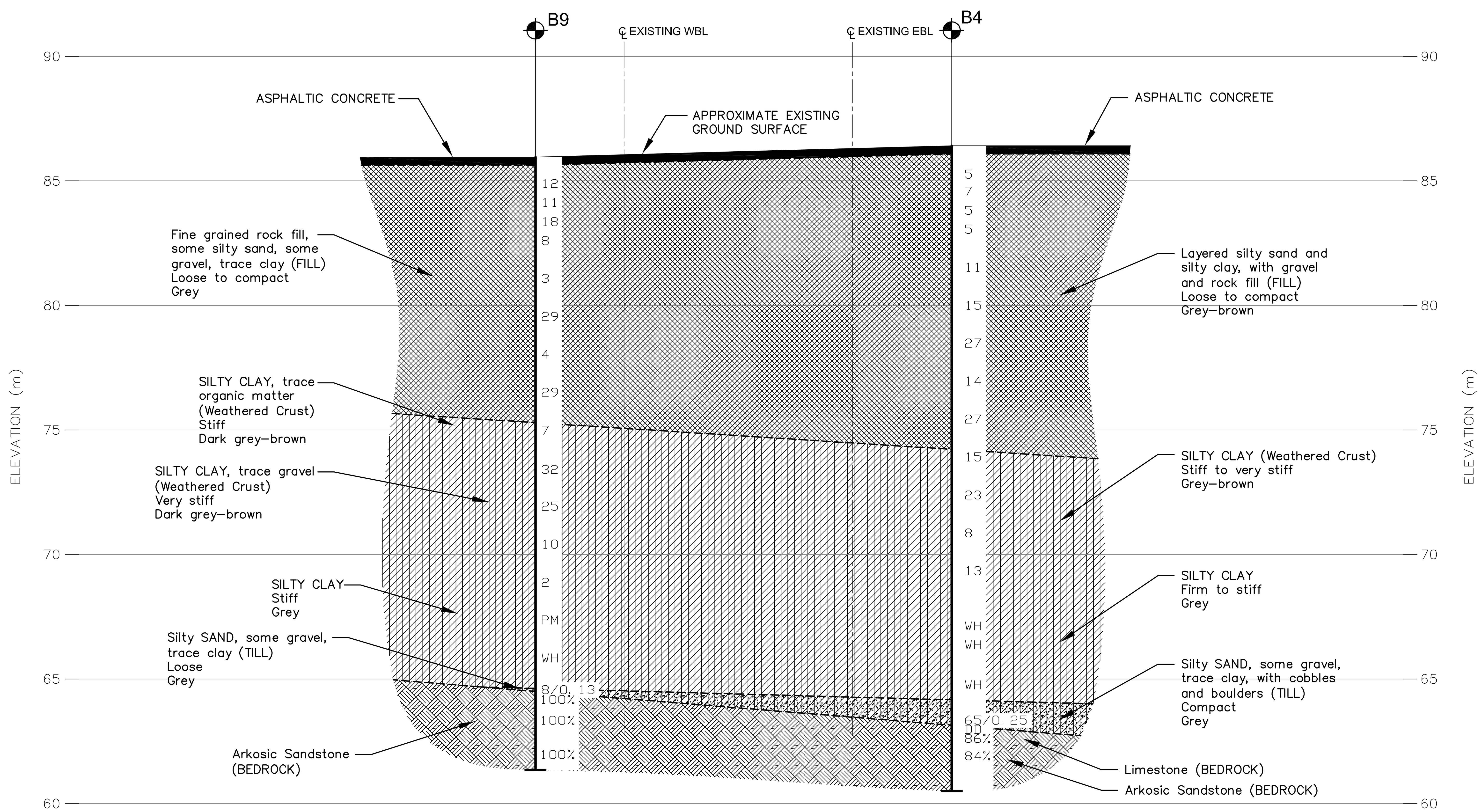
SHEET

Golder Associates
Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



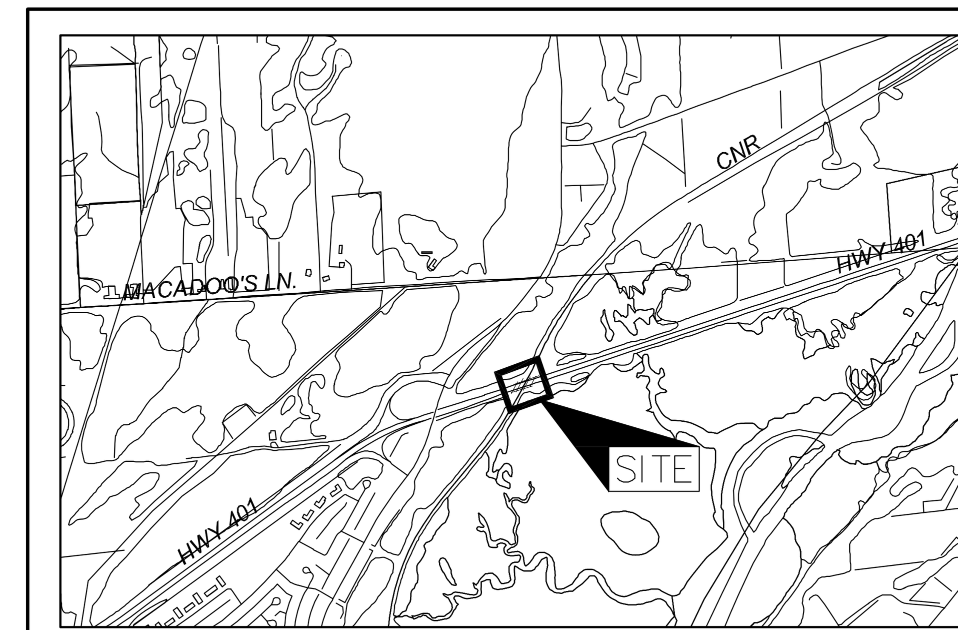
PROFILE E-E'

HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE



PROFILE F-F'

HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE



KEY PLAN

SCALE
2 0 1 1 km

LEGEND

⊕	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock quality designation
	Seal
	Piezometer
▼	WL in piezometer on Sept. 29, 2009

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B1	87.6	4904268.6	307188.1
B2	80.4	4904273.3	307205.4
B3	77.3	4904280.6	307230.6
B4	86.4	4904295.2	307258.3
B5	87.8	4904302.4	307197.7
B6	87.5	4904304.2	307204.2
B7	79.7	4904318.2	307227.7
B8	78.5	4904323.6	307254.3
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METRIC
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NO.	DATE	BY	REVISION

Geocres No. 31C-202

HWY. 401	PROJECT NO. 08-1111-0044		DIST.
SUBM'D. EO	CHKD. MSS	DATE: 1/20/2011	SITE:
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 4

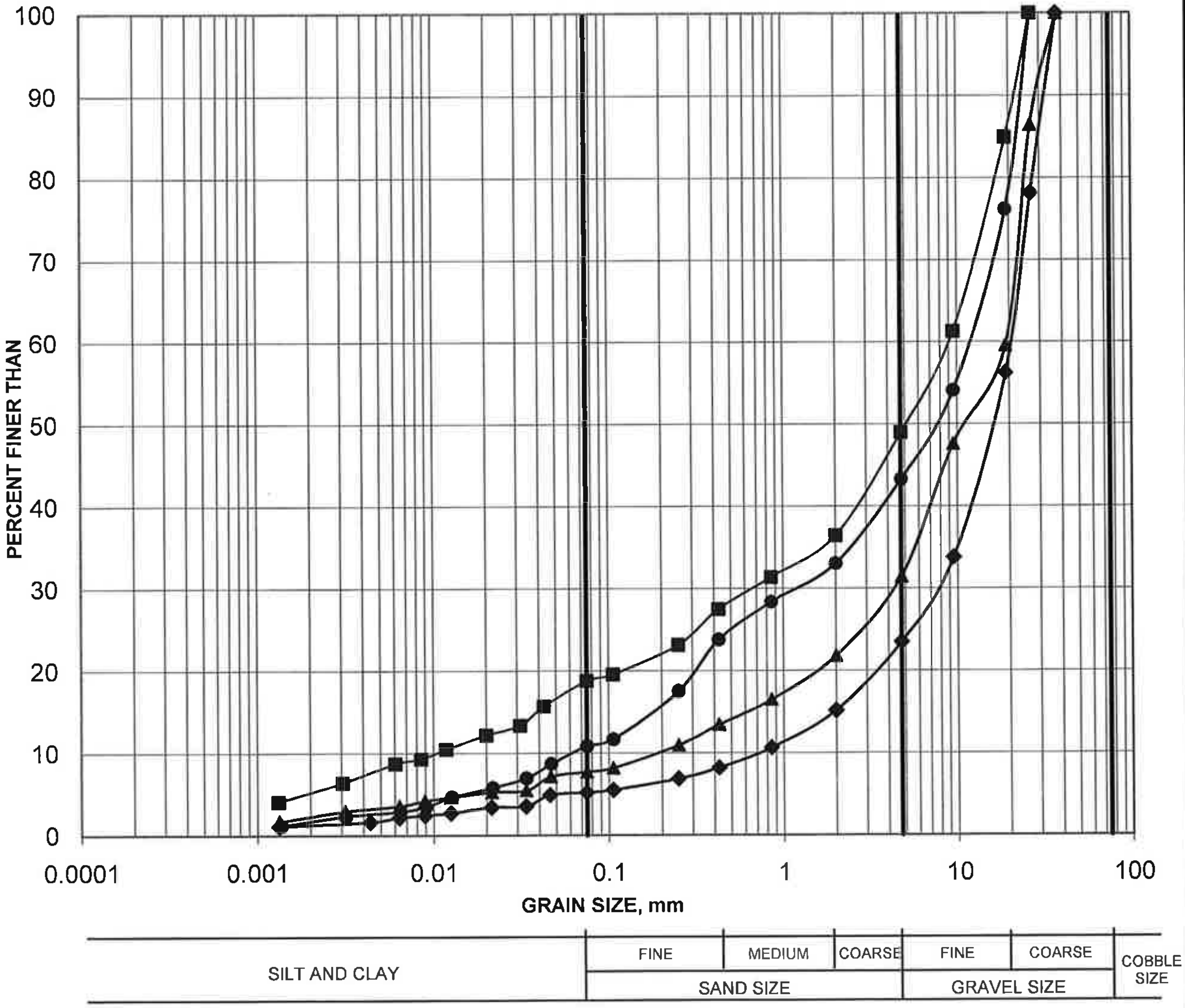
MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: January 20, 2011

GRAIN SIZE DISTRIBUTION

FIGURE 1

ROCK FILL



Borehole	Sample	Depth (m)
B4 (■)	5	2.29-2.90
B4 (◆)	11	10.67-11.28
B9 (●)	3	1.52-2.13
B10 (▲)	5	4.57-5.18

Project: 08-1111-0044

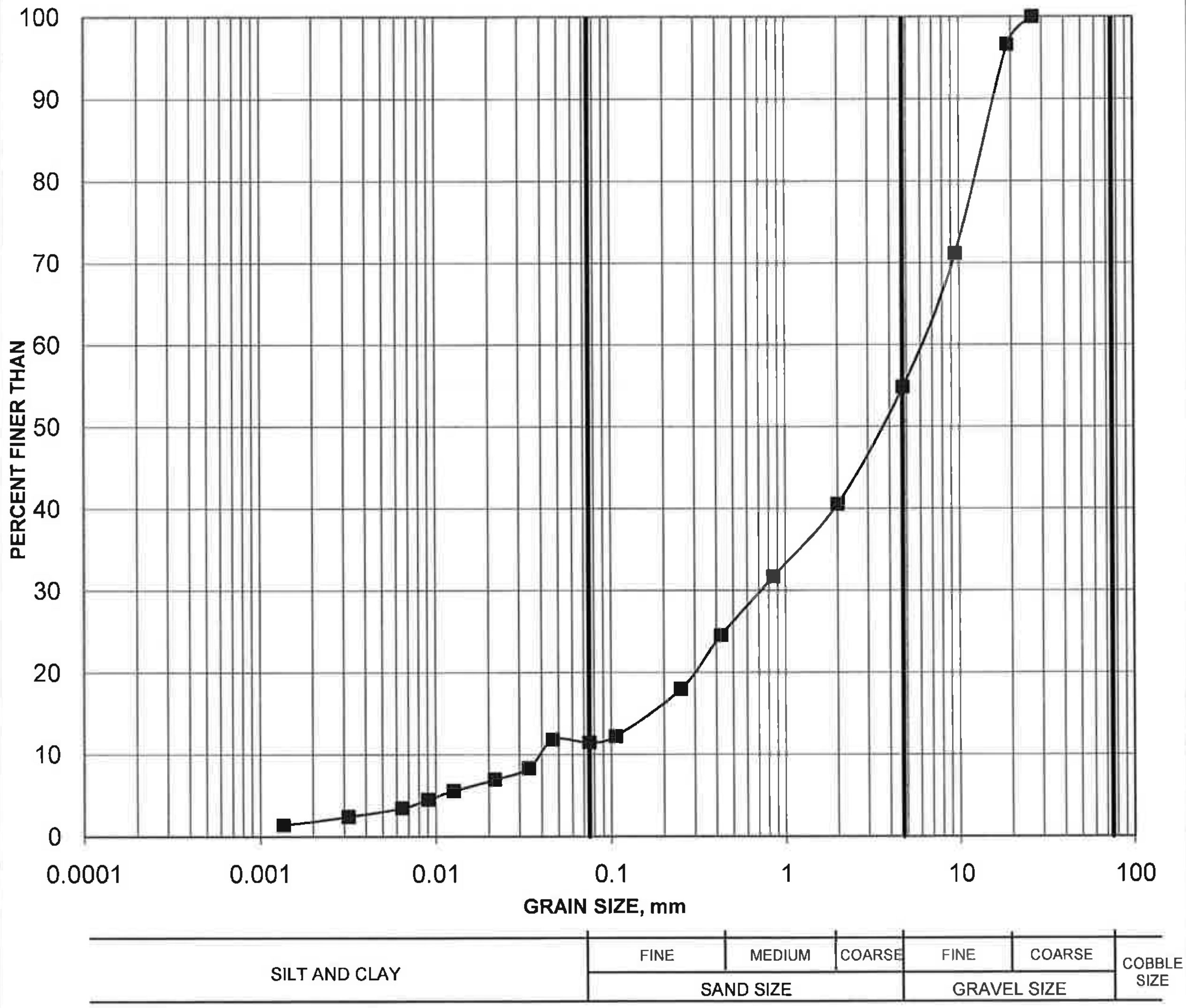
Golder Associates

Created by:	KSL
Checked by:	CNM

GRAIN SIZE DISTRIBUTION

FIGURE 2

SAND & GRAVEL FILL



Borehole	Sample	Depth (m)
B5	2	0.61-1.22

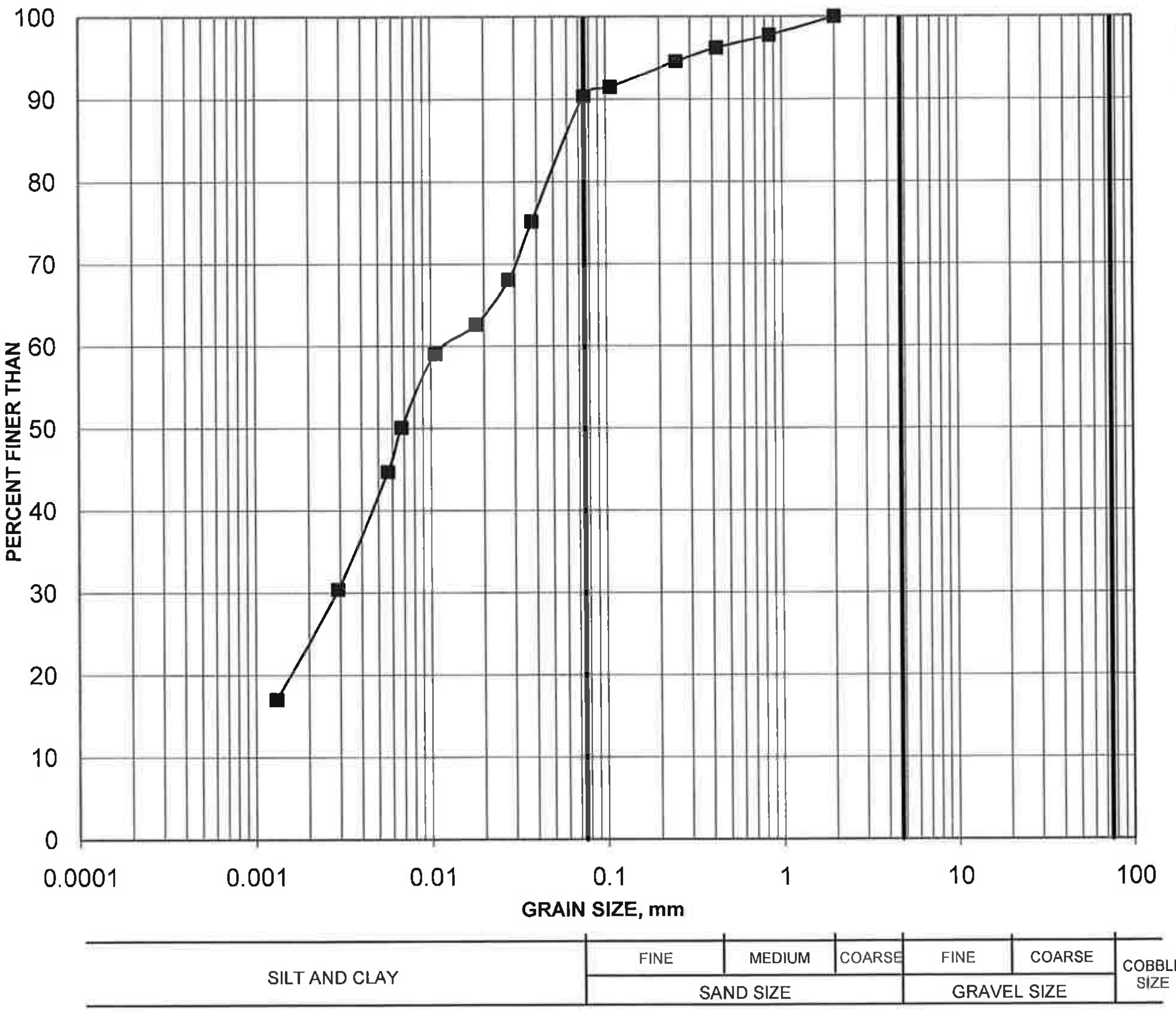
Project: 08-1111-0044

Golder Associates

Created by:	KSL
Checked by:	CNM

GRAIN SIZE DISTRIBUTION	FIGURE 3

SILT FILL



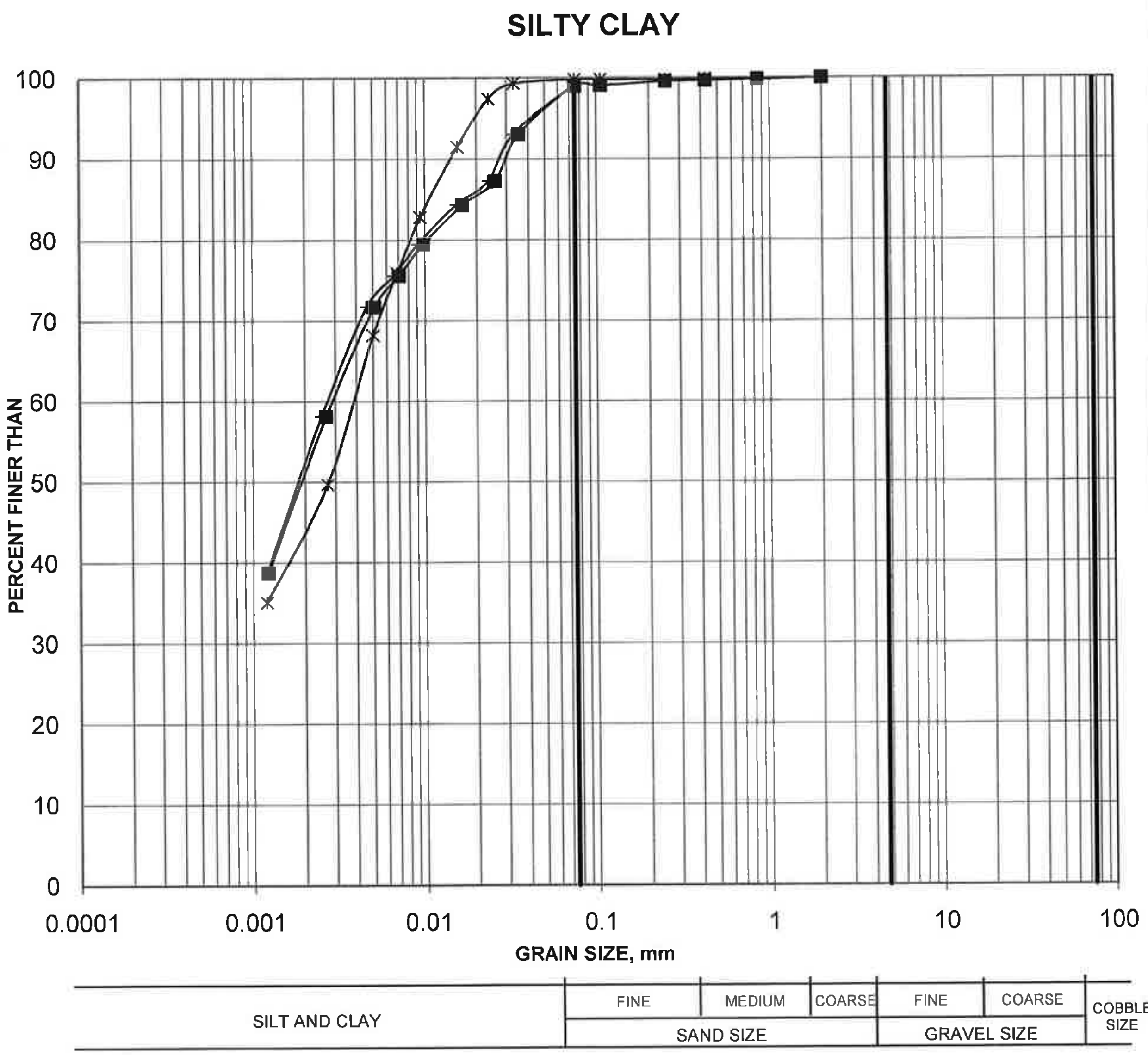
Borehole	Sample	Depth (m)
B7	8B	3.81-4.27

Project: 08-1111-0044

Golder Associates

Created by:	KSL
Checked by:	CNM

GRAIN SIZE DISTRIBUTION	FIGURE 4

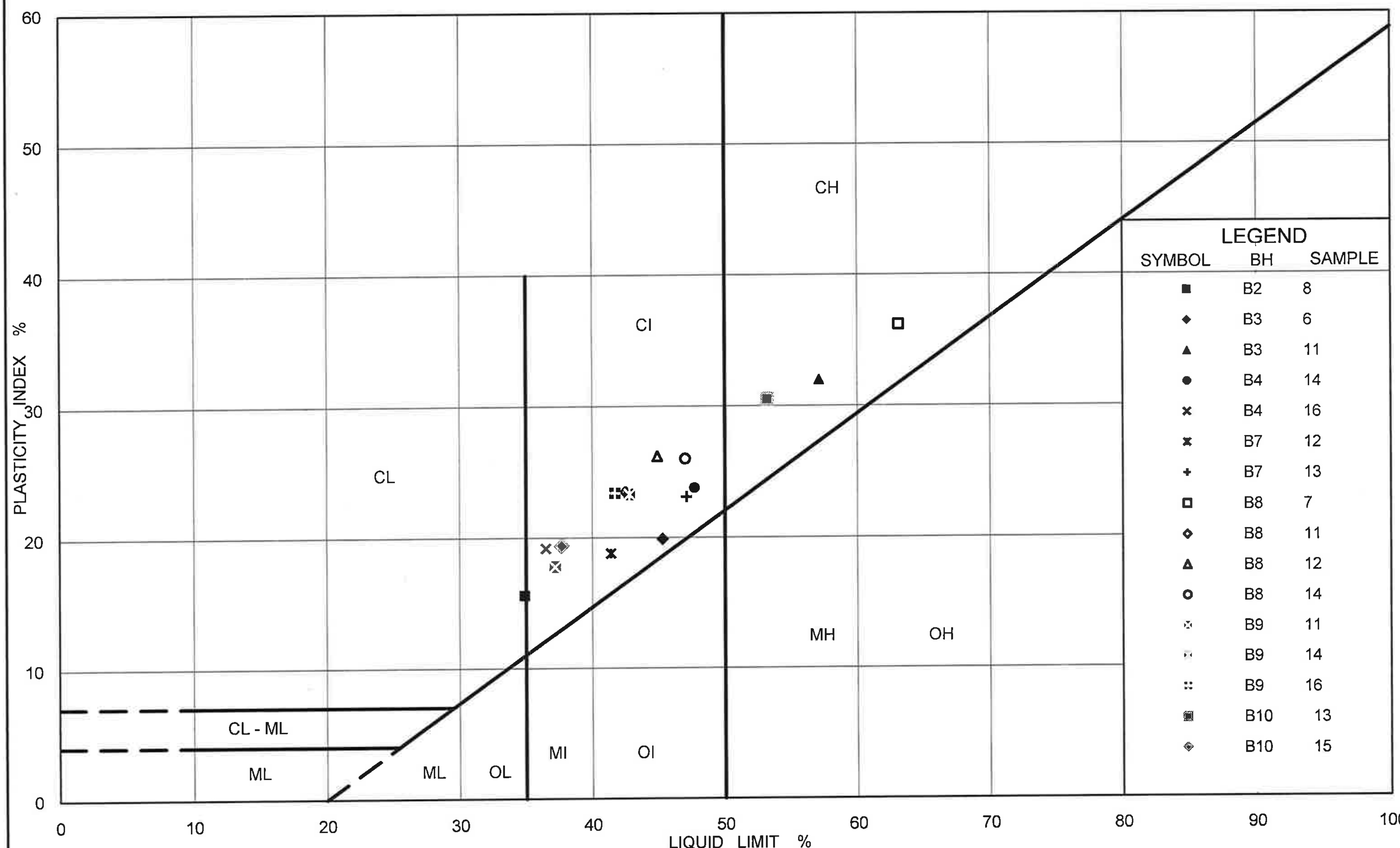


Borehole	Sample	Depth (m)
B4	14	15.24-15.85
B4	16	19.10-19.50
B8	12	10.40-10.80
#REF!	14	16.77-17.38
#REF!	13	16.77-17.38
#REF!	15	19.82-20.43

Project: 08-1111-0044 **Golder Associates**

Created by:	KSL
Checked by:	CNM

Oct 75, FF-S-21



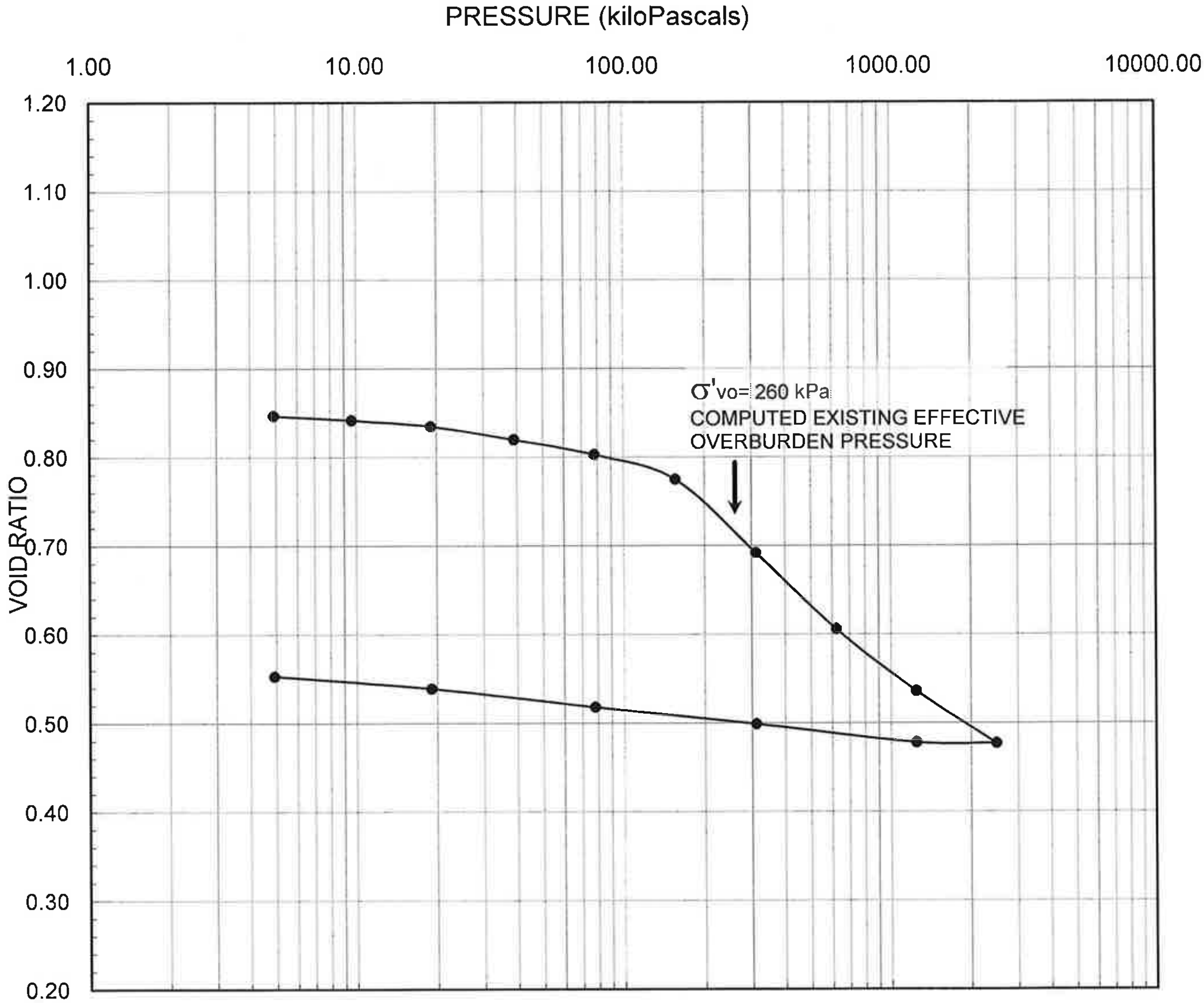
Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay to Clay

FIG No. 5

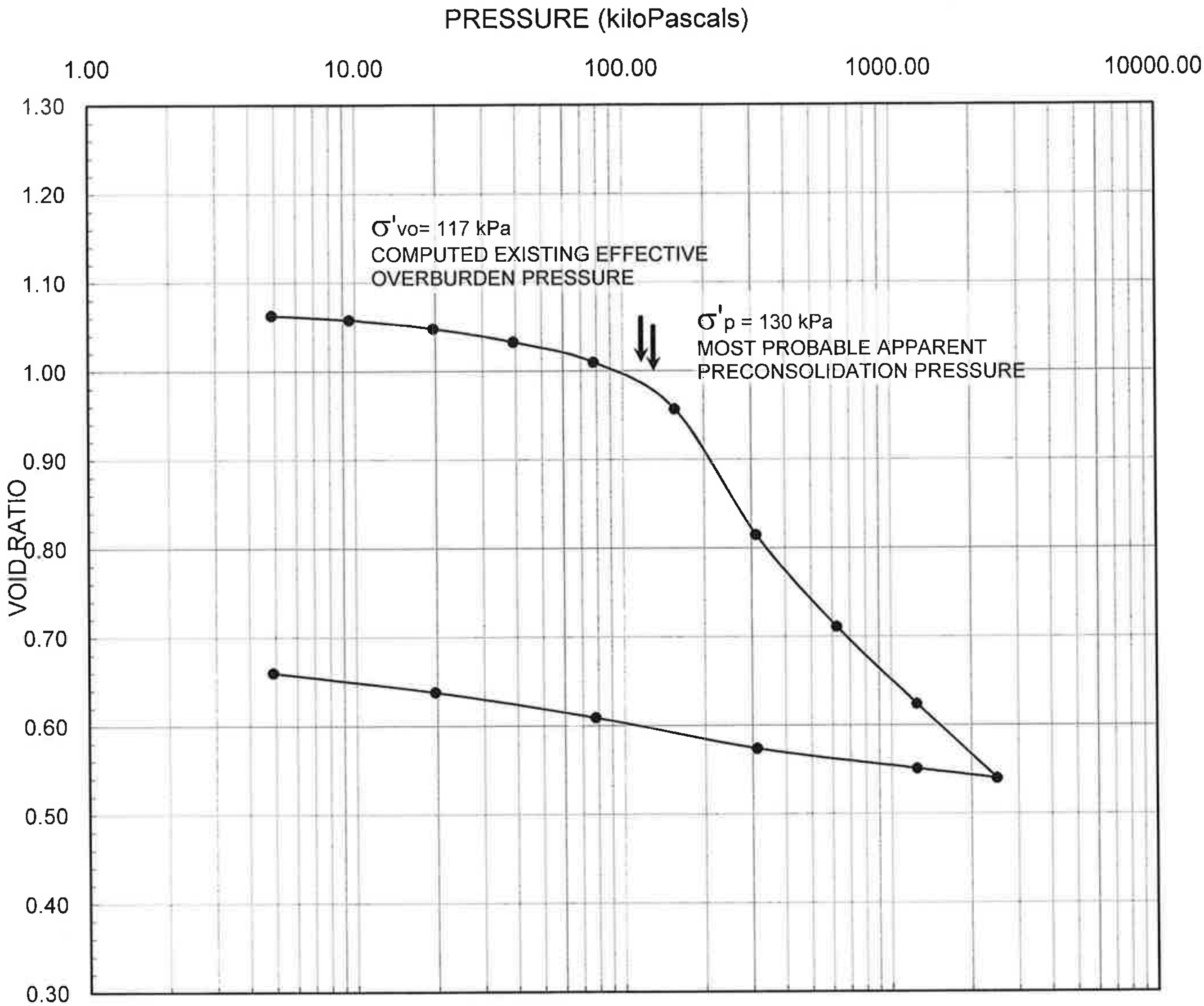
Project No. 08-1111-0044



LEGEND

Borehole: B4	w_i = 30.8%	S_o = 101%
Sample: 16	w_f = 21.4%	C_c = 0.29
Depth (m): 19.1-19.5		C_r = 0.033

Golder Associates	SCALE: AS SHOWN	TITLE	
	DATE: 12/03/10	**CONSOLIDATION TEST RESULTS**	
	DESIGN: NA		
	CADD: MM		
FILE No. Consolidation summary	CHECK: CNM		
PROJECT No. 08-1111-0044 REV. 0	REVIEW: ESO		FIGURE 6



LEGEND

Borehole: B8
Sample: 12
Depth (m): 10.4-10.8

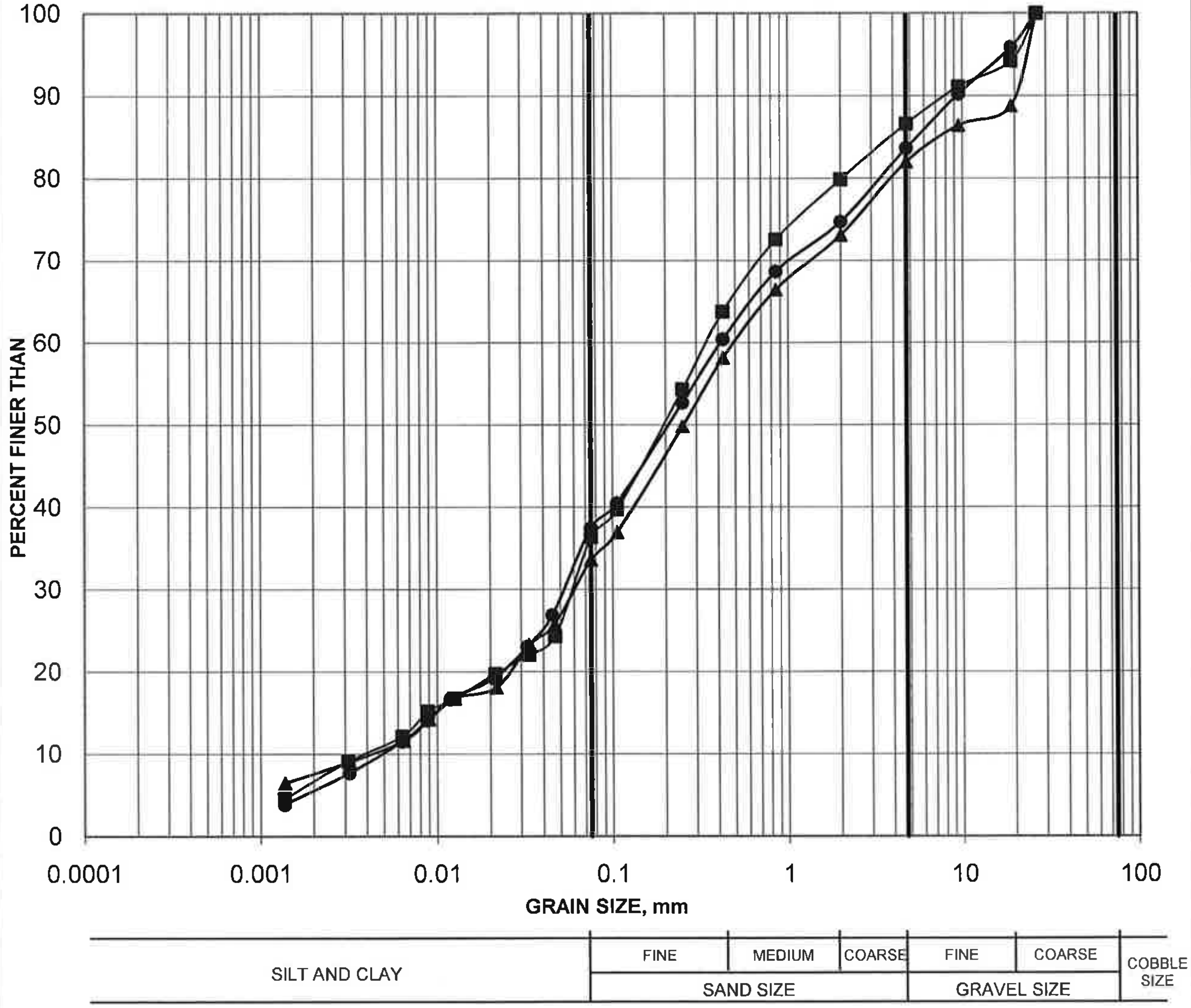
w_i = 37.8%
w_f = 24.8%

S_o = 100%
C_c = 0.47
C_r = 0.053

Golder Associates	SCALE: AS SHOWN	TITLE: CONSOLIDATION TEST RESULTS	
	DATE: 12/03/10		
	DESIGN: NA		
	CADD: MM		
FILE No.: Consolidation summary	CHECK: CNM		
PROJECT No.: 08-1111-0044 REV. 0	REVIEW: ESO		FIGURE 7

GRAIN SIZE DISTRIBUTION	FIGURE 8

SILTY SAND TILL

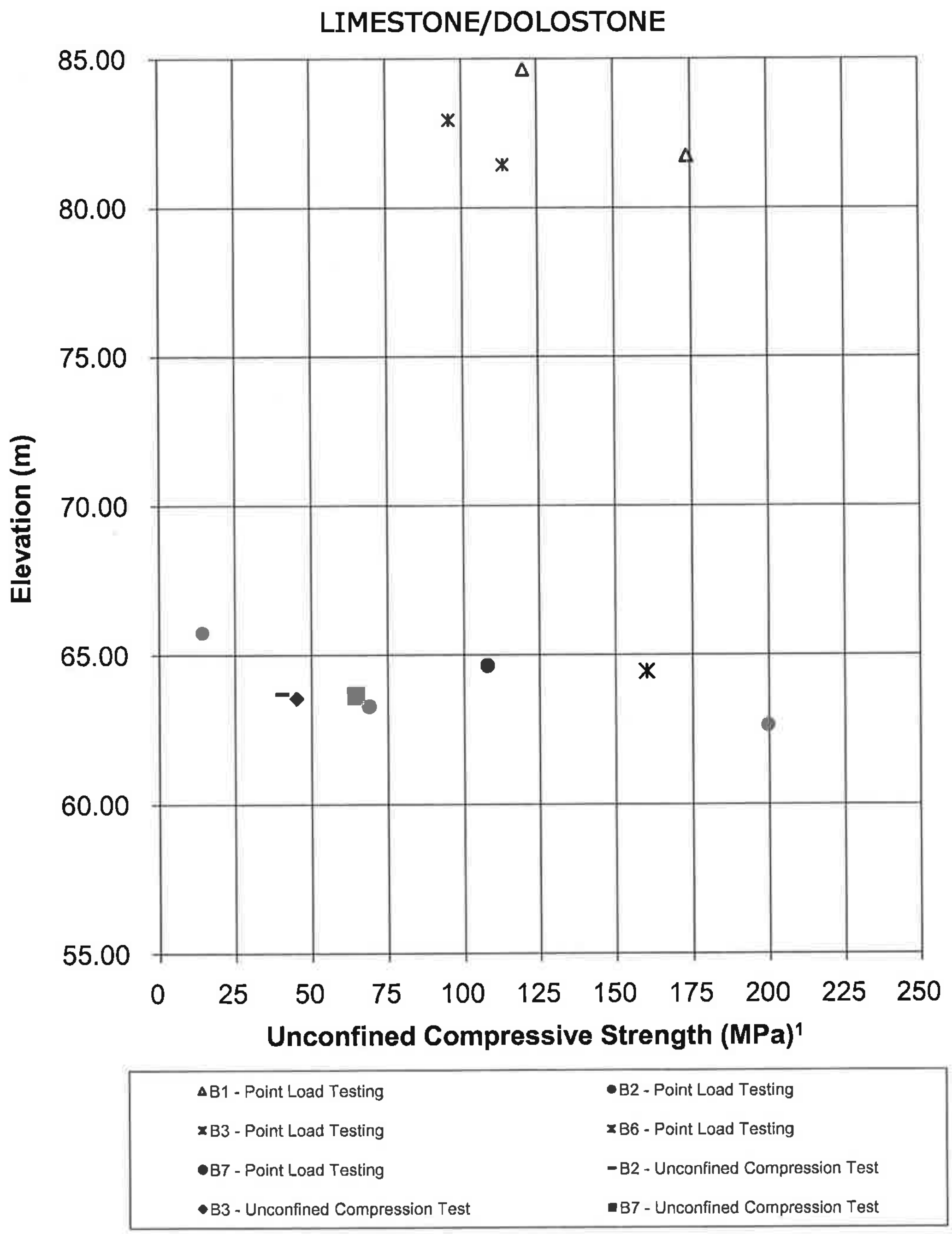


Borehole	Sample	Depth (m)
B4	19	22.86-23.24
B8	15	12.80-13.41
B10	16	21.34-21.77

Project: 08-1111-0044 **Golder Associates**

Created by:	KSL
Checked by:	CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH MEASUREMENTS	FIGURE 9

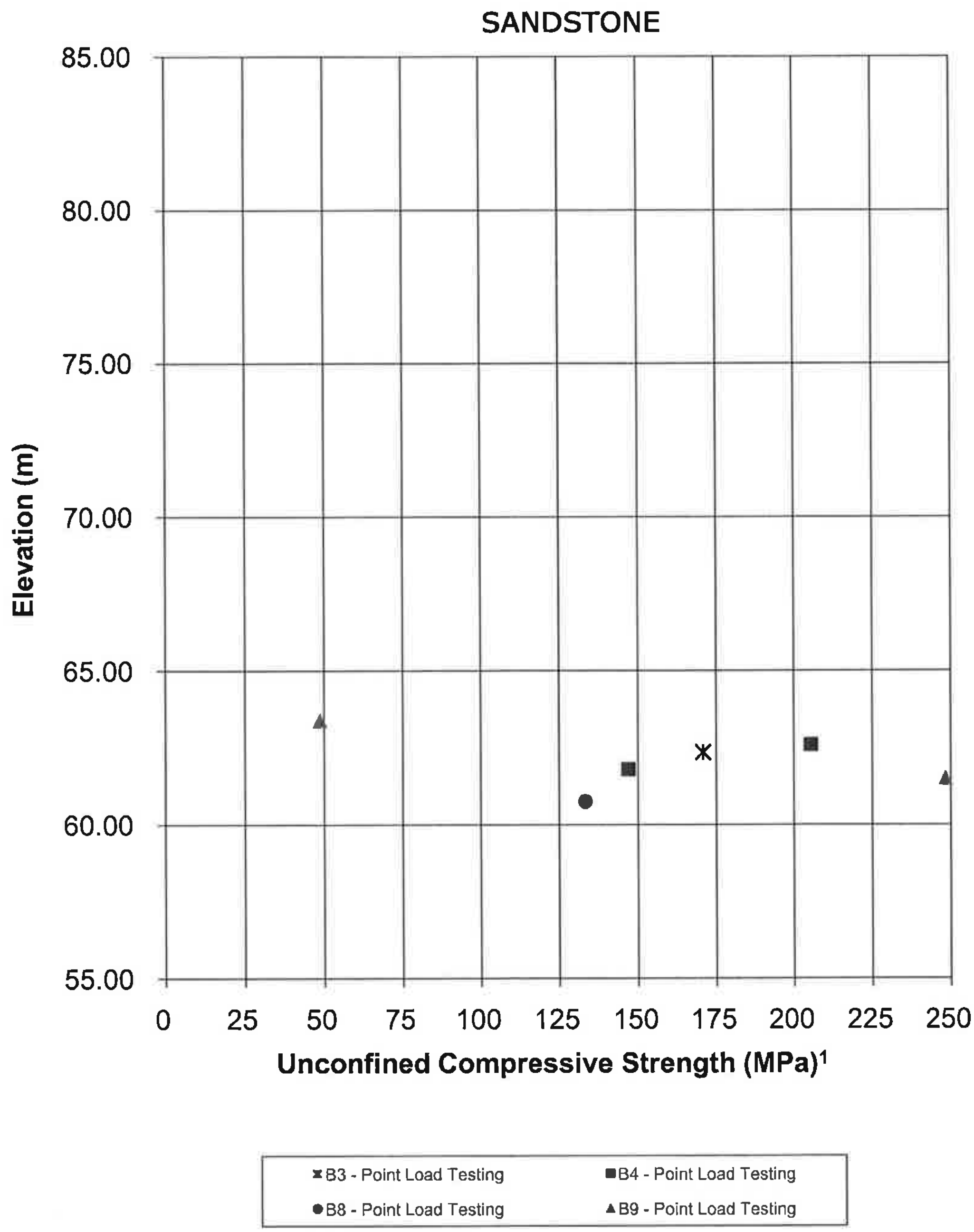


[1] Unconfined compressive strengths from point load testing inferred using $Is_{50}xC$ where C=21, from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

Project: 08-1111-0044	Golder Associates	Created by:	KSL
		Checked by:	ESO

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH MEASUREMENTS

FIGURE 10

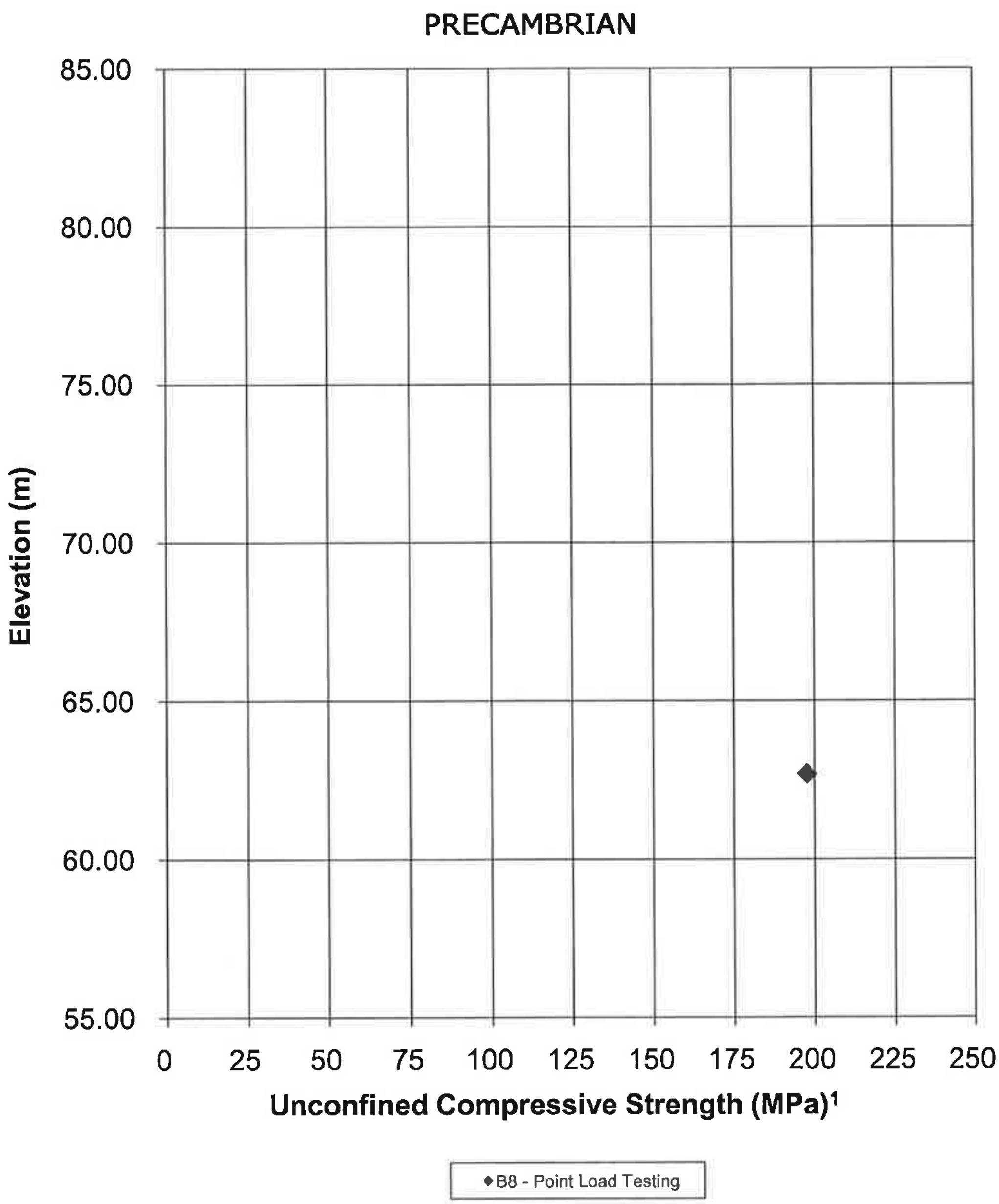


[1] Unconfined compressive strengths from point load testing inferred using $Is_{50}xC$ where C=21, from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

Project: 08-1111-0044 **Golder Associates**

Created by:	KSL
Checked by:	ESO

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH MEASUREMENTS	FIGURE 11



[1] Unconfined compressive strengths from point load testing inferred using Is_{50}xC where C=21, from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

Project: 08-1111-0044

Golder Associates

Created by:	KSL
Checked by:	ESO

APPENDIX A

List of Abbreviations and Symbols
Rock Description Terminology
Record of Borehole Sheets

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
DO	Drive open
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
DT	Dual Tube 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.)
DD- Diamond Drilling

Dynamic 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, 60^0 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

Peizo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60^0 conical tip and a projected 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 or 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 limited
w_l	liquid limit
C	consolidaiton (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 test (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	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 stresses (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 ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_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 (overconsolidated 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	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	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. τ=c'σ' tan ∤'
2. Shear strength=(Compressive strength)/2

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 - 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	<2 microns

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

O:\ Templates\Rock Description Terminology

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 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including 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 90^0 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 information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B -	Bedding	Ca -	Calcite
FO -	Foliation/Schistosity	P -	Polished
CL -	Cleavage	S -	Slickensided
SH -	Shear Plane/Zone	SM -	Smooth
VN -	Vein	R -	Ridged/Rough
F -	Fault	ST -	Stepped
CO -	Contact	PL -	Planar
J -	Joint	FL -	Flexured
FR-	Fracture	UE -	Uneven
MF -	Mechanical	W -	Wavy
A-	Angular	C -	Curved
BP-	Bedding Plane	H -	Hackly
BL-	Blast Induced	SL -	Sludge Coated
‖ -	Parallel To	TCA -	To Core Axis
⊥ -	Perpendicular To	STR -	Stress Induced

Golder Associates

RECORD OF BOREHOLE No B1

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
LOCATION N 4904268.6 ;E 307188.1
ORIGINATED BY DG
DIST HWY 401
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
COMPILED BY JM
DATUM Geodetic
DATE June 17, 2009
CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.6	GROUND SURFACE										
0.0	Sand and gravel, some crushed stone, trace silt (FILL) Loose Brown Moist		1	SS	3		87				
			2	SS	10						
86.4 1.2	Rock fill, some sand and gravel, trace silt (FILL) Grey brown Moist						86				
			3	AW RC	DD		85				
			4	AW RC	DD				o		
			5	AW RC	DD						
			6	AW RC	DD						
84.3 3.3	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fractured Laminated Medium strong Grey Note: Bedrock cored between 3.3 m and 6.2 m depth. For bedrock coring details refer to Record of Drillhole B1.		C1	BW RC	REC 100%		84				RQD = 49%
			C2	BW RC	REC 100%		83				RQD = 14%
			C3	BW RC	REC 100%		82				RQD = 38%
81.4 6.2	End of Borehole										

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 (%) 25 50 75.

MIS-MTO 001 08-1111-0044.GPJ GAL-MISS.GDT 12/3/10 DD

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

PROJECT: 08-1111-0044

RECORD OF DRILLHOLE: B1

SHEET 1 OF 1

LOCATION: N 4904268.6 ;E 307188.1

DRILLING DATE: June 17, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable

DRILLING CONTRACTOR: OGS

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH / COLOUR % RETURN	RECOVERY: TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS / TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole B1		84.30										
4–6		LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fractured Laminated Medium strong Grey		3.30	C1, C2, C3									
		End of Drillhole		81.40 6.20										

DEPTH SCALE 1 : 75

LOGGED: DG

CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B2

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01 LOCATION N 4904273.3 ;E 307205.4 ORIGINATED BY HEC
DIST ____ HWY 401 BOREHOLE TYPE Portable Equipment, Continuous Sampling, NW, BW, AW, EW Casing, Wash Boring COMPILED BY JM
DATUM Geodetic DATE August 26 - September 1, 2009 CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
80.4	GROUND SURFACE				
0.0	Rock FILL				
79.2 / 1.2	Coarse grained rock fill, some gravel and grey-brown silty clay (FILL) Moist	1	BW RC	DD	
		2	BW RC	DD	
77.5	Silty clay (FILL) Very stiff Grey-brown Moist	3	SS	13	
77.1 / 3.4	Fine grained rock fill, trace sand, some grey-brown silty clay (FILL) Moist	4	AW RC	DD	
		5	AW RC	DD	
		6	AW RC	DD	
75.4 / 5.1	SILTY CLAY (Weathered Crust) Stiff to very stiff Grey-brown to grey Moist	7	SS	13	
		8	SS	72	
		9	SS	28	
73.1 / 7.3	LIMESTONE SLABS, with some voids and soil infilling Grey	10	RC	DD	
		11	RC	DD	
70.4	Void or loose soil	12	RC	DD	
	Void or loose soil				
10.3	LIMESTONE SLABS, with some voids and soil infilling Grey				
69.3	Void or loose soil				
11.3	LIMESTONE SLABS, with some voids and soil infilling Grey	13	RC	DD	
		14	RC	DD	
		15	RC	DD	
66.2 / 14.3		C1	RC	REC 97%	RQD = 71%

Elevation scale: 80, 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66

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 (%) 25 50 75
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No B2

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ___ HWY 401
DATUM Geodetic
LOCATION N 4904273.3 ;E 307205.4
BOREHOLE TYPE Portable Equipment, Continuous Sampling, NW, BW, AW, EW Casing, Wash Boring
DATE August 26 - September 1, 2009
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY KSL

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
		C1	RC			RQD = 71%
65.2 / 15.2	LIMESTONE (BEDROCK) Fresh, Thinly bedded, Weak, Greenish-grey	C2	RC	REC 71%	65	RQD = 13%
64.3 / 16.1	LIMESTONE (BEDROCK) Fresh to weathered, Thinly bedded, Weak, Greenish-grey and reddish-grey	C3	RC	REC 83%	64	RQD = 55%
	LIMESTONE (BEDROCK) Fresh, Thinly bedded, Weak, Reddish-grey					
63.2 / 17.2		C4	RC	REC 100%	63	RQD = 83%
62.6 / 17.9	LIMESTONE (BEDROCK) Fresh, Thinly bedded, Medium strong, Grey					

Note:
Bedrock cored between 14.3 m and 17.9 m depth.
For bedrock coring details refer to Record of Drillhole B2.

End of Borehole

Note:
Water level in well screen at 4.3 m depth (Elev. 76.1) on Sept. 29, 2009.

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 (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

RECORD OF DRILLHOLE: B2

PROJECT: 08-1111-0044

SHEET 1 OF 1

LOCATION: N 4904273.3 ;E 307205.4

DRILLING DATE: August 26 - September 1, 2009

DATUM: Geodetic

DRILL RIG: Portable

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: OGS

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.
	Continued from Record of Borehole B2	66.10	
	LIMESTONE (BEDROCK) Fresh Thinly bedded Weak Greenish-grey	14.30	C1
15	LIMESTONE (BEDROCK) Fresh to weathered Thinly bedded Weak Greenish-grey and reddish-grey	65.20 15.20	C2
16	LIMESTONE (BEDROCK) Fresh Thinly bedded Weak Reddish-grey	64.30 16.10	C3
17	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey	63.20 17.20	C4
18	End of Drillhole	62.50 17.90	

Other column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29

DEPTH SCALE

1 : 75

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LOGGED: HEC

CHECKED: KSL

MIS-RCK 001 08-1111-0044 (ROCK).GPJ GAL-MISS.GDT 12/3/10 DD

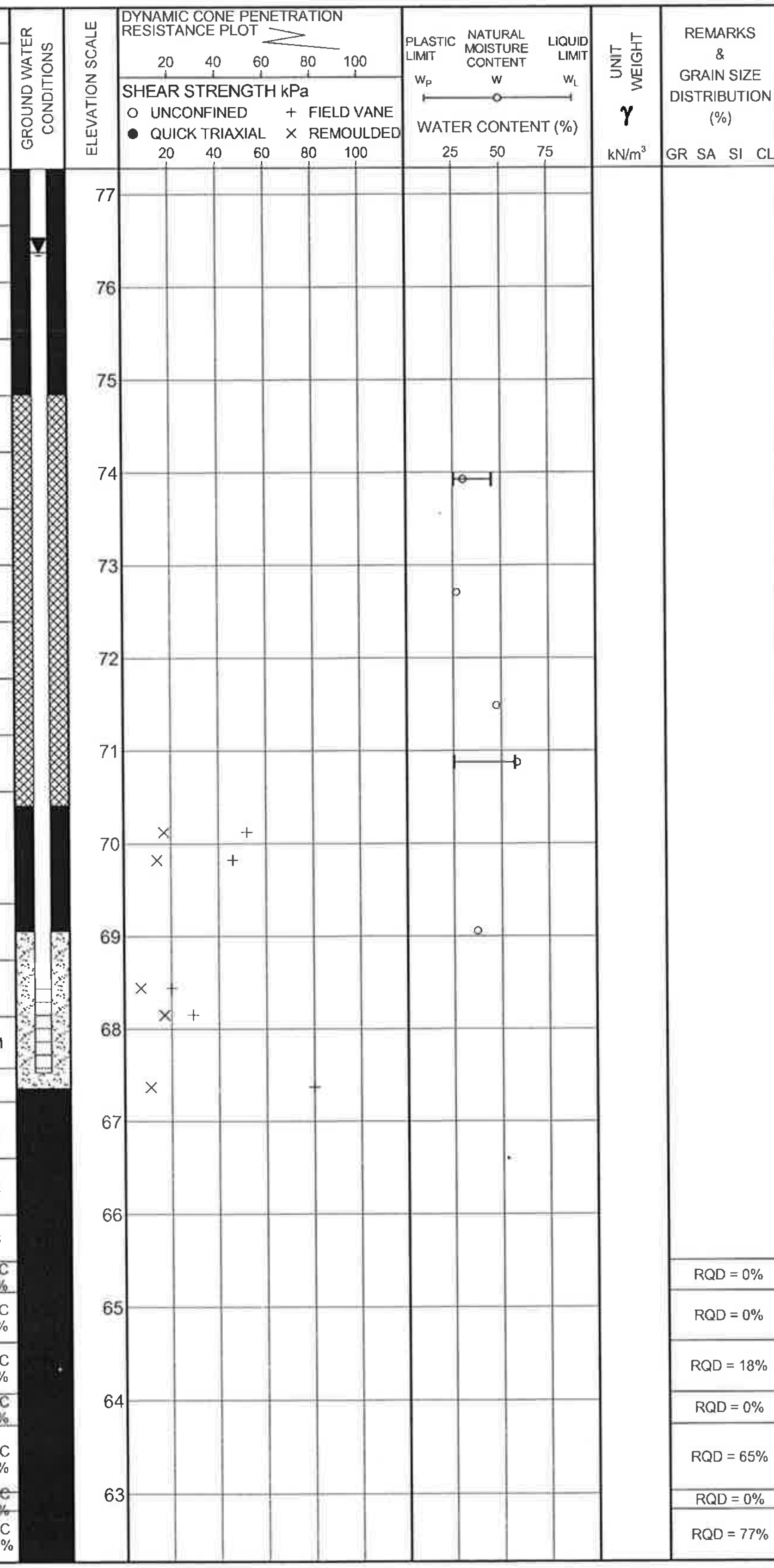
Golder Associates

RECORD OF BOREHOLE No B3

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4904280.6 ;E 307230.6
BOREHOLE TYPE Portable Equipment, Continuous Sampling, EW Casing, Wash Boring
DATE September 2-10, 2009
ORIGINATED BY DWM
COMPILED BY JM
CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
77.3	GROUND SURFACE				
0.0	Organic matter, cinders and asphalt pieces, trace sand and gravel (FILL) Very loose to loose Grey to black Dry	1	SS	4	
		2	SS	4	
76.1 / 1.2	Silty clay, trace sand and gravel (FILL) Loose Brown Moist	3	SS	5	
75.5 / 1.8	Silty clay, trace rootlets (FILL) Loose Dark grey Moist	4	SS	6	
74.8 / 2.4	SILTY CLAY, occasional silty sand seam (Weathered Crust) Stiff to very stiff Grey-brown to brown Moist	5	SS	13	
		6	SS	28	
		7	SS	40	
		8	SS	23	
		9	SS	11	
71.8 / 5.5	SILTY CLAY Stiff to firm Grey Wet	10	SS	5	
		11	SS	3	
		12	SS	2	
68.1 / 9.1	SILTY CLAY, occasional silty sand seam Stiff Grey Wet	13	TP	PM	
66.9 / 10.4	Silty SAND, trace gravel, some clay (TILL) Compact Grey-brown Wet	14	SS	26	
66.6 / 10.7	Silty SAND, some gravel, trace clay, occasional weathered bedrock fragments (TILL) Compact Brown Wet	15	SS	22	
		16	SS	38	
65.5 / 11.8	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh to slightly weathered Thinly bedded Grey Note: Occasional very thin soil seams in bedrock from about 12.8 to 14.0m depth.	C1	RC	REC 69%	RQD = 0%
		C2	RC	REC 67%	RQD = 0%
		C3	RC	REC 41%	RQD = 18%
		C4	RC	REC 15%	RQD = 0%
		C5	RC	REC 93%	RQD = 65%
63.0 / 14.3		C6	RC	REC 44%	RQD = 0%
		C7	RC	REC 100%	RQD = 77%



Continued Next Page

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

Golder Associates

PROJECT 08-1111-0044 **RECORD OF BOREHOLE No B3** **METRIC**

G.W.P. 78-99-01 LOCATION N 4904280.6 ;E 307230.6 ORIGINATED BY DWM

DIST HWY 401 BOREHOLE TYPE Portable Equipment, Continuous Sampling, EW Casing, Wash Boring COMPILED BY JM

DATUM Geodetic DATE September 2-10, 2009 CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION (--- CONTINUED FROM PREVIOUS PAGE ---)	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 (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	ARKOSIC SANDSTONE (BEDROCK) Fresh Fine to coarse grained Medium strong Grey		C7	RC	REC 100%		62				RQD = 77%
61.8 15.4	Note: Bedrock cored between 11.8 m and 15.4 m depth. For bedrock coring details refer to Record of Drillhole B3.										
	End of Borehole										
	Note: Water level in well screen at 0.9 m depth (Elev. 76.4) on Sept. 29, 2009.										

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

RECORD OF DRILLHOLE: B3

PROJECT: 08-1111-0044

LOCATION: N 4904280.6 ;E 307230.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 2-10, 2009

DRILL RIG: Portable

DRILLING CONTRACTOR: OGS

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY: TOTAL CORE % (80 60 40 20)	RECOVERY: SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 (5 10 15 20)	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole B3		65.50												
12		LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh to slightly weathered Thinly bedded Grey Note: Occasional very thin soil seams in bedrock from about 12.8 to 14.0m depth.		11.80	C1											
					C2											
13					C3											
					C4											
14					C5											
		ARKOSIC SANDSTONE (BEDROCK) Fresh Fine to coarse grained Medium strong Grey		63.00 14.30	C6											
15					C7											
		End of Drillhole		61.90 15.40												
16																
17																
18																
19																
20																
21																
22																
23																
24																
25																
26																

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DWM

CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B4

 METRIC

PROJECT	08-1111-0044		ORIGINATED BY DG
G.W.P.	78-99-01	LOCATION N 4904295.2 ;E 307258.3	COMPILED BY JM
DIST	HWY 401	BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem	CHECKED BY KSL
DATUM	Geodetic	DATE June 12, 2009	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.4	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE					
86.1 / 0.4	Crushed stone (FILL) Grey Moist	1	GRAB		86	
		2	GRAB			
85.5 / 0.9	Sand and gravel (FILL) Brown Moist	3	SS	5	85	
	Layered silty sand and silty clay, with gravel and rock fill (FILL) Loose to compact Grey-brown Moist	4	SS	7		
		5	SS	5	84	51 30 14 5
		6	SS	5	83	
		7	SS	11	82, 81	
		8	SS	15	80	
		9	SS	27	79, 78	
		10	SS	14	77	
		11	SS	27	76, 75	76 19 3 2
74.2 / 12.2	SILTY CLAY (Weathered Crust) Stiff to very stiff Grey-brown Moist	12	SS	15	74	
		13	SS	23	73, 72	

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 (%) 25 50 75

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No B4

 METRIC

PROJECT 08-1111-0044

G.W.P. 78-99-01

LOCATION N 4904295.2 ;E 307258.3

ORIGINATED BY DG

DIST ___ HWY 401

BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem

COMPILED BY JM

DATUM Geodetic

DATE June 12, 2009

CHECKED BY KSL

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 (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION
	SILTY CLAY (Weathered Crust) Stiff to very stiff Grey-brown Moist
68.1 / 18.3	SILTY CLAY Firm to stiff Grey Wet
64.2 / 22.3	Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Compact Grey Wet
63.1	LIMESTONE (BEDROCK) Grey
62.8 / 23.6	ARKOSIC SANDSTONE (BEDROCK) Fresh Fine to coarse grained Medium strong Grey, red and greenish grey Note: Bedrock cored between 23.6 m and 25.9 m depth. For bedrock coring details refer to Record of Drillhole B4.
60.5 / 25.9	End of Borehole

NUMBER	TYPE	"N" VALUES	REMARKS / GRAIN SIZE (GR SA SI CL)
14	SS	8	0 1 47 52
15	SS	13	
16	TP	WH	0 0 56 44
17	TP	WH	
18	SS	WH	
19	SS	65/0.25	13 50 30 7
20	NQ RC	DD	
C1	NQ RC	REC 100%	RQD = 86%
C2	NQ RC	REC 100%	RQD = 84%

Elevation scale: 71, 70, 69, 68, 67, 66, 65, 64, 63, 62, 61

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

RECORD OF DRILLHOLE: B4

PROJECT: 08-1111-0044
LOCATION: N 4904295.2 ;E 307258.3
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 12, 2009
DRILL RIG: CME 75
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole B4		62.80												
24		ARKOSIC SANDSTONE (BEDROCK) Fresh Fine to coarse grained Medium strong Grey, red and greenish grey		23.60	C1											
25					C2											
26		End of Drillhole		60.50 25.00												
27																
28																
29																
30																
31																
32																
33																
34																
35																
36																
37																
38																

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: DG
CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B5

 METRIC

PROJECT	08-1111-0044				
G.W.P.	78-99-01	LOCATION	N 4904302.4 ;E 307197.7	ORIGINATED BY	DG
DIST	HWY 401	BOREHOLE TYPE	Portable Equipment, Continuous Sampling, BW Casing, Wash Boring	COMPILED BY	JM
DATUM	Geodetic	DATE	June 18, 2009	CHECKED BY	KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.8	GROUND SURFACE						
0.0	Sand and gravel, trace silt (FILL) Loose to compact Grey-brown Moist	1	SS	6			
		2	SS	4	87	o	45 43 10 2
86.0		3	SS	15	86		
1.8	Rock FILL Grey	4	AW RC	DD			
85.4 / 2.4	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fractured Medium laminated Weak to medium strong Grey Note: Bedrock cored between 2.4 m and 5.7 m depth. For bedrock coring details refer to Record of Drillhole B5.	C1	BW RC	REC 100%			RQD = 0%
		C2	BW RC	REC 100%	85		RQD = 0%
		C3	BW RC	REC 100%	84		RQD = 43%
82.0		C4	BW RC	REC 100%	83		RQD = 25%
5.7	End of Borehole						

Other column headings: 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, 25 50 75; UNIT WEIGHT γ kN/m³

MIS-MTO 001 08-1111-0044.GPJ GAL-MISS.GDT 12/3/10 DD

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

PROJECT: 08-1111-0044

LOCATION: N 4904302.4 ;E 307197.7

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: B5

DRILLING DATE: June 18, 2009

DRILL RIG: Portable

DRILLING CONTRACTOR: OGS

SHEET 1 OF 1

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN	FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole B5		85.40													
3, 4, 5		LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fractured Medium laminated Weak to medium strong Grey		2.40	C1 C2 C3 C4												
				82.10 5.70													
6		End of Drillhole															

Depth scale: 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17

MIS-RCK 001 08-1111-0044 (ROCK).GPJ GAL-MISS.GDT 12/3/10 DD

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DG

CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B6

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
LOCATION N 4904304.2 ;E 307204.2
ORIGINATED BY DG
DIST HWY 401
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
COMPILED BY JM
DATUM Geodetic
DATE June 17, 2009
CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
87.5	GROUND SURFACE					
0.0	Sand and gravel, trace silt (FILL) Compact Brown Moist	1	SS	13	87	
86.9 / 0.6	Silty sand, some gravel, trace clay and rootlets (FILL) Loose Grey Moist	2	SS	4		
85.9 / 1.6	Rock FILL Grey	3	AW RC	DD	86	
		4	AW RC	DD		
		5	AW RC	DD	85	
84.7 / 2.9	Wood (FILL) Cobbles and boulders (FILL)	6	AW RC	DD		
		7	AW RC	DD	84	
83.8 / 3.7	DOLOMITIC LIMESTONE (BEDROCK) Fractured Laminated Medium strong Grey	C1	BW RC	REC 100%		RQD = 55%
		C2	BW RC	REC 100%	83	RQD = 14%
82.6 / 4.9	LIMESTONE (BEDROCK) Fresh Medium bedded Medium strong Grey Note: Bedrock cored between 3.7 m and 6.7 m depth. For bedrock coring details refer to Record of Drillhole B6.	C3	BW RC	REC 100%	82	RQD = 78%
		C4	BW RC	REC 100%	81	RQD = 81%
80.8 / 6.7	End of Borehole					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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 (%) 25 50 75; UNIT WEIGHT γ kN/m³; GR SA SI CL

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

PROJECT: 08-1111-0044

RECORD OF DRILLHOLE: B6

SHEET 1 OF 1

LOCATION: N 4904304.2 ;E 307204.2

DRILLING DATE: June 17, 2009

DATUM: Geodetic

DRILL RIG: Portable

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: OGS

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH / COLOUR % RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole B6		83.80												
4		DOLOMITIC LIMESTONE (BEDROCK) Fractured Laminated Medium strong Grey		3.70	C1 C2											
5		LIMESTONE (BEDROCK) Fresh Medium bedded Medium strong Grey		82.60 4.90	C3 C4											
6																
7		End of Drillhole		80.80 6.70												

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: DG

CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B7

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01 LOCATION N 4904318.2 ;E 307227.7 ORIGINATED BY DG
DIST HWY 401 BOREHOLE TYPE Portable Equipment, Continuous Sampling, AW, BW Casing, Wash Boring COMPILED BY JM
DATUM Geodetic DATE June 19, 2009 CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.7	GROUND SURFACE				
0.0	Sand and gravel, trace silt (FILL) Compact Grey-brown Moist	1	SS	28	
78.9	Rock FILL Grey	2	SS	33/0.20	
		3	AW RC	DD	
1.0	Silty sand, some gravel, trace clay, with cobbles and boulders (FILL) Compact Grey-brown Moist to wet	4	AW RC	DD	
		5	SS	16	
		6	SS	15	
		7	SS	16	
75.9 / 3.8	Silt, some clay, trace sand (FILL) Grey-brown Wet	8	SS	17	0 9 67 24
75.2 / 4.6	Silty sand, with cobbles and boulders (FILL) Grey Wet	9	SS	27	
		10	AW RC	DD	
74.4 / 5.3	SILTY CLAY, trace gravel (Weathered Crust) Very stiff Grey-brown	11	SS	7/0.18	
		12	SS	26	
		13	SS	11/0.23	
73.4	LIMESTONE SLABS				
73.1	VOID or loose soil	14	AW RC	DD	
72.7	LIMESTONE SLAB				
72.3 / 7.4	VOID or loose soil	15	AW RC	DD	
71.7	LIMESTONE SLAB	~~16~~	~~AW RC~~	~~DD~~	
71.4	VOID or loose soil				
71.0	LIMESTONE SLABS	17	AW RC	DD	
70.6	VOID or loose soil				
70.3	LIMESTONE SLABS				
	VOID or loose soil	18	AW RC	DD	
69.6	LIMESTONE SLABS				
	VOID or loose soil	19	AW RC	DD	
	VOID or loose soil				
10.5	From 6.3m to 12.2m depth: LIMESTONE SLABS, with numerous voids and occasional inclined bedding planes Grey	20	AW RC	DD	
		21	AW RC	DD	
67.6 / 12.2	Grey COBBLES, BOULDERS and red brown SILTY CLAY	22	EW RC	DD	
		23	EW RC	DD	
66.3 / 13.4	LIMESTONE (BEDROCK) Fractured Medium strong Grey to reddish brown	C1	EW RC	REC 84%	RQD = 42.3%
65.7 / 14.1		~~C2~~	~~EW RC~~	REC 100%	~~RQD = 0%~~
		C3	EW RC	REC 76.3%	RQD = 0%
		C4	EW RC	REC 100%	RQD = 58.1%

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 (%) 25 50 75
UNIT WEIGHT γ kN/m³
ELEVATION SCALE 79 78 77 76 75 74 73 72 71 70 69 68 67 66 65

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No B7

 METRIC

PROJECT 08-1111-0044			
G.W.P. 78-99-01	LOCATION N 4904318.2 ;E 307227.7	ORIGINATED BY DG	
DIST	HWY 401	BOREHOLE TYPE Portable Equipment, Continuous Sampling, AW, BW Casing, Wash Boring	COMPILED BY JM
DATUM Geodetic	DATE June 19, 2009	CHECKED BY KSL	

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 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 (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey to reddish brown		C4	EW RC	REC 100%						RQD = 58.1%
			C5	EW RC	REC						RQD = 66.7%
			C6	EW RC	100%		64				RQD = 0%
	Note: Bedrock cored between 13.4 m and 16.5 m depth.		C7	EW RC	REC 100% REC 100%						RQD = 83.9%
63.2 16.5	For bedrock coring details refer to Record of Drillhole B7. End of Borehole										

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 08-1111-0044

RECORD OF DRILLHOLE: B7

SHEET 1 OF 1

LOCATION: N 4904318.2 ;E 307227.7

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 19, 2009

DRILL RIG: Portable

DRILLING CONTRACTOR: OGS

FR/FX-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
	Continued from Record of Borehole B7	66.30 / 13.40	
14	LIMESTONE (BEDROCK) Fractured Medium strong Grey to reddish brown	65.60 / 14.10	C1, C2
15, 16	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey to reddish brown	63.20 / 16.50	C3, C4, C5, C6, C7
	End of Drillhole		

Column headings: DEPTH SCALE METRES; DRILLING RECORD; DESCRIPTION; SYMBOLIC LOG; ELEV. DEPTH (m); RUN No.; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 5 10 15 20; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6; NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DG

CHECKED: KSL

Foundation Design

Golder Associates

RECORD OF BOREHOLE No B8

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
LOCATION N 4904323.6 ;E 307254.3
ORIGINATED BY DWM
DIST HWY 401
BOREHOLE TYPE Portable Equipment, Continuous Sampling, NW, AW, EW Casing, Wash Boring
COMPILED BY JM
DATUM Geodetic
DATE September 10 - 16, 2009
CHECKED BY KSL

SOIL PROFILE

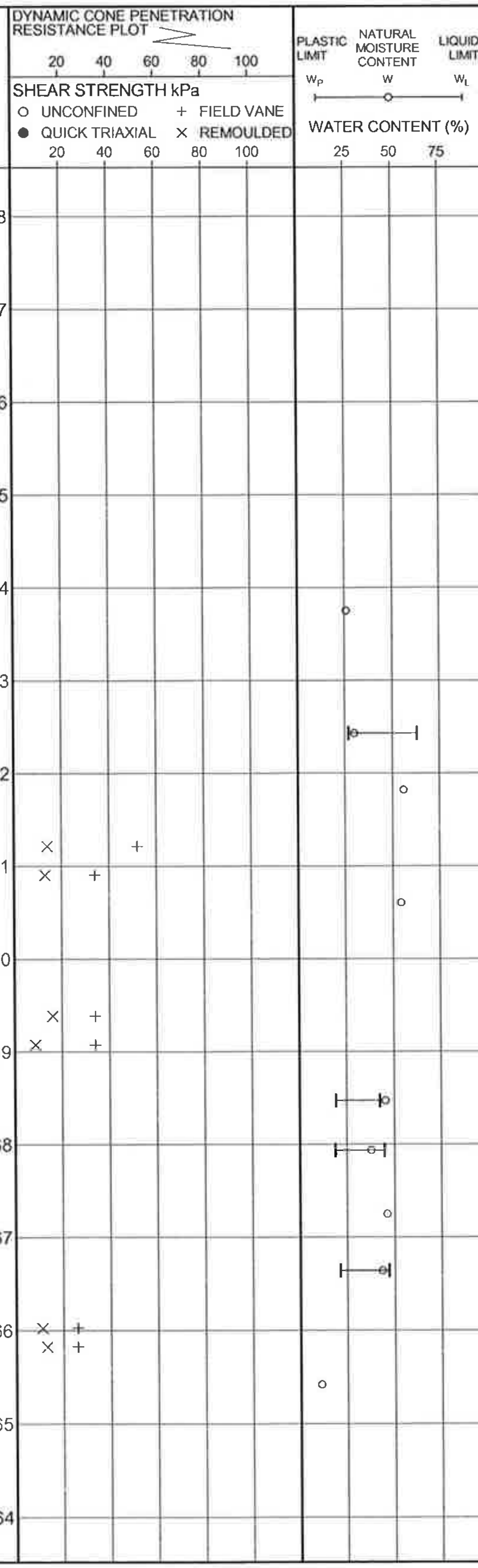
ELEV / DEPTH	DESCRIPTION
78.5	GROUND SURFACE
0.0	Silty sand, some gravel, trace organic matter and rootlets, occasional brick fragments (FILL) Very loose to dense Brown to black Dry
77.0 / 1.5	Rock fill, with some gravel, sand and silt infilling (FILL) Grey Wet
74.3 / 4.3	SILTY CLAY, trace sand and rootlets (Weathered Crust) Stiff Grey-brown Moist
73.3 / 5.2	SILTY CLAY, occasional sand seam (Weathered Crust) Very stiff to stiff Grey-brown Moist
72.1 / 6.4	SILTY CLAY, occasional sand seam Stiff to firm Grey Moist to wet
65.8 / 12.7	Silty SAND, some gravel, trace clay, with cobbles (TILL) Compact Grey Wet
64.9 / 13.6	ARKOSIC SANDSTONE (BEDROCK) Fresh to sightly weathered Fine grained Reddish grey

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	33	
3	SS	8	
4	SS	23/0.15	
5	SS	11	
6	SS	69	
7	SS	27	
8	SS	5	
9	AS		
10	AS		
11	SS	1	
12	TP	PM	0 1 43 57
13	SS	3	
14	SS	1	
15	SS	25	18 49 26 7
C1	RC	REC 100%	RQD = 77%
C2	RC	REC 100%	RQD = 59%
C3	RC	REC 100%	RQD = 47%
C4	RC	REC 100%	RQD = 76%

GROUND WATER CONDITIONS

ELEVATION SCALE: 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64



UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No B8

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904323.6 ;E 307254.3
BOREHOLE TYPE Portable Equipment, Continuous Sampling, NW, AW, EW Casing, Wash Boring
DATE September 10 - 16, 2009

ORIGINATED BY DWM
COMPILED BY JM
CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ 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 (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —		C4	RC	REC 100%		63				RQD = 76%
62.8 15.8	PRECAMBRIAN (BEDROCK) Fresh Medium strong Black, grey and red		C5	RC	REC 79%						RQD = 38%
62.1 16.4	End of Borehole Note: Bedrock cored between 13.6 m and 16.4 m depth. For bedrock coring details refer to Record of Drillhole B8.										

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

RECORD OF DRILLHOLE: B8

PROJECT: 08-1111-0044
LOCATION: N 4904323.6 ;E 307254.3
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 10 - 16, 2009
DRILL RIG: Portable
DRILLING CONTRACTOR: OGS

SHEET 1 OF 1
DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
	Continued from Record of Borehole B8	64.90	
14	ARKOSIC SANDSTONE (BEDROCK) Fresh to sightly weathered Fine grained Reddish grey	13.60	C1 C2 C3
15			C4
16	PRECAMBRIAN (BEDROCK) Fresh Medium strong Black, grey and red	62.70 15.80	C5
	End of Drillhole	62.10 16.40	

Column headings: DEPTH SCALE METRES; DRILLING RECORD; DESCRIPTION; SYMBOLIC LOG; ELEV. DEPTH (m); RUN No.; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 (5 10 15 20); DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6); NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: DWM
CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B9

 METRIC

PROJECT 08-1111-0044

G.W.P. 78-99-01 | LOCATION N 4904324.4 ;E 307274.6 | ORIGINATED BY DG

DIST ___ HWY 401 | BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem | COMPILED BY JM

DATUM Geodetic | DATE June 9, 2009 | CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.0	GROUND SURFACE						
0.0 / 85.6	ASPHALTIC CONCRETE						
0.5 / 85.1	Crushed stone (FILL) Grey Moist	1	GRAB				
0.9	Sand and gravel (FILL) Brown Mosit	2	SS	12	85		
84.4 / 1.5	Sand (FILL) Brown Moist	3	SS	11	84	o	57 32 9 2
	Fine grained rock fill, some silty sand, some gravel, trace clay (FILL) Loose to compact Grey Moist	4	SS	18	83		
		5	SS	8	82		
		6	SS	3	81		
		7	SS	29	80, 79		
		8	SS	4	78, 77		
		9	SS	29	76		
75.3 / 10.7	SILTY CLAY, trace organic matter (Weathered Crust) Stiff Dark grey-brown Moist	10	SS	7	75		
74.7 / 11.3	SILTY CLAY (Weathered Crust) Very stiff Grey-brown Moist	11	SS	32	74, 73	plotted with Wp–WL range	
		12	SS	25	72		

Continued Next Page

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 (%) 25 50 75
UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No B9

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01 LOCATION N 4904324.4 ;E 307274.6 ORIGINATED BY DG
DIST HWY 401 BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem COMPILED BY JM
DATUM Geodetic DATE June 9, 2009 CHECKED BY KSL

— CONTINUED FROM PREVIOUS PAGE —

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
70.7 / 15.2	SILTY CLAY, trace gravel (Weathered Crust) Very stiff Dark grey-brown Moist	13	SS	10	
69.2 / 16.8	SILTY CLAY Stiff Grey Moist to wet	14	SS	2	1 1 52 46
		15	SS	PM	
		16	SS	WH	
64.6 / 21.5	Silty SAND, some gravel, trace clay (TILL) Loose Grey Wet	17	SS	8/0.13	
	ARKOSIC SANDSTONE (BEDROCK) Fresh Fine to medium grained Medium strong Greenish grey and reddish grey	C1	NQ RC	REC 100%	RQD = 100%
	Note: Bedrock cored between 21.5 m and 24.6 m depth. For bedrock coring details refer to Record of Drillhole B9.	C2	NQ RC	REC 100%	RQD = 100%
		C3	NQ RC	REC 100%	RQD = 100%
61.4 / 24.6	End of Borehole				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (70, 69, 68, 67, 66, 65, 64, 63, 62); 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 (%) 25 50 75; UNIT WEIGHT γ kN/m³.

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

PROJECT: 08-1111-0044

RECORD OF DRILLHOLE: B9

SHEET 1 OF 1

LOCATION: N 4904324.4 ;E 307274.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 9, 2009

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole B9		64.50													
22		ARKOSIC SANDSTONE (BEDROCK) Fresh Fine to medium grained Medium strong Greenish grey and reddish grey		21.50	C1												
23					C2												
24				61.40 24.60	C3												
25		End of Drillhole															

26 27 28 29 30 31 32 33 34 35 36

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: DG
CHECKED: KSL

Golder Associates

RECORD OF BOREHOLE No B10

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01 LOCATION N 4904329.3 ;E 307288.8 ORIGINATED BY DG
DIST HWY 401 BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem COMPILED BY JM
DATUM Geodetic DATE June 8, 2009 CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
85.6	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE					
85.3 / 0.3	Crushed stone (FILL) Grey Moist					
84.8 / 0.8	Sand and gravel (FILL) Compact Brown Moist	1	SS	17		
84.1 / 1.5	Fine grained rock fill, some coarse rock fill, some silty sand, some gravel, trace clay (FILL) Loose to compact Grey Moist	2	SS	4		
		3	SS	9		
		4	SS	9		
		5	SS	9	o	68 24 5 3
		6	SS	19		
78.2 / 7.5	Rock fill, some silty sand, some gravel, trace clay (FILL) Compact to very dense Grey Moist to wet	~~7~~	~~SS~~	~~50/0.01~~		
		8	SS	69/0.26		
		9	SS	25		
		~~10~~	~~SS~~	~~50/0.05~~		
71.9 / 13.7	SILTY CLAY (Weathered Crust) Stiff to very stiff Dark grey-brown Wet	11	SS	5	o	

Chart headings: GROUND WATER CONDITIONS; ELEVATION SCALE (85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72, 71); 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 (%) 25 50 75; UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No B10

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01 LOCATION N 4904329.3 ;E 307288.8 ORIGINATED BY DG
DIST HWY 401 BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem COMPILED BY JM
DATUM Geodetic DATE June 8, 2009 CHECKED BY KSL

— CONTINUED FROM PREVIOUS PAGE —

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY (Weathered Crust) Stiff to very stiff Dark grey-brown Wet	12	SS	10	70	
		13	SS	5	69 / 68	0 2 33 65
		14	SS	4	67 / 66	
65.8 / 19.8	SILTY CLAY Firm Grey Wet	15	SS	2	65	0 1 62 37
64.6 / 21.0	Silty SAND, some gravel, trace clay (TILL) Grey Wet	16	SS	58/0.28	64	16 47 32 5
63.9 / 21.8	End of Borehole Auger Refusal					

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 (%) 25 50 75
UNIT WEIGHT γ kN/m³

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

APPENDIX B

Non Standard Special Provisions

DETERMINATION OF BEDROCK ELEVATIONS AT WEST ABUTMENT FOOTINGS- Item No.

Special Provision

The bedrock surface near the west abutment is expected to drop off quickly to the east. Prior to mass excavation for the west abutment footings, the contractor shall confirm the bedrock elevations within the footprint of the new west abutment footings by advancing probe holes at minimum 2 m spacing along the perimeter and in the centre of the proposed footings. The data should be submitted to the Contract Administrator a minimum of 4 weeks in advance of footing construction to allow for review and possible modifications to the structural and foundation design

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

DOWELS INTO ROCK – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the structure footings.

1.2 Instructions to Contractor

1.2.1 These instructions are to be read in conjunction with the Contract Drawings.

1.2.2 A total of 1 test Dowels into Rock are required for the Dowels into Rock at each structure footing.

1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.

1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
- All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
- Test results verifying the 28 day strength of non-shrink grout.
- The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- The procedures to verify hole length. Records of measurements that verify the hole length.
- Records of all drilling procedures, rock conditions encountered, and installation times.
- Test procedures for Dowels into Rock.
- Drawings and design calculations for a suitable reaction system for the applied test loads.
- Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- Drawings and details for reference system arrangement.
- Current calibration curves shall be provided for all gauges.
- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

1.7.1 Soils, rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout.

The Contractor shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout,
- installation procedures

3.0 EQUIPMENT

3.1 General

3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.

4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout

4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

4.3.3 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the each structure location.

5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.

5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.

5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

- The beams shall be independently supported with the support firmly embedded in the ground.
- The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
- Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit

Working Drawings that include the above noted records to the Contract Administrator.

5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at each structure location.

Tests for Dowels into Rock shall have a capacity of at least [insert value] kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

MICROPILES- Item No.

Special Provision

To be provided as part of the detailed design deliverable if this foundation option is chosen.

END OF SECTION

BOULDERS/OBSTRUCTIONS DURING PILING AND CAISSON INSTALLATION - Item No.

Special Provision

Rock fill was encountered at the bridge foundation locations, and cobbles and boulders were observed within the glacial till deposits overlying bedrock at some of the bridge foundation locations, as noted on the borehole records. Limestone slabs were also encountered at the location of the new west pier foundations.

For drilled piles and caissons, appropriate equipment and procedures will be required to penetrate/remove obstructions within the rock fill and penetrate the limestone slabs and cobbles/boulders in the glacial till as part of pile installation for the bridge foundations.

For driven piles, appropriate equipment and procedures will be required to protect the pile tip (e.g. bearing points) and penetrate/remove obstructions within the rock fill. Driven piles are not to be overdriven and pile damage caused by boulders/obstructions at depth should be prevented.

Sheet piles driven for temporary roadway and railway protection may be unable to penetrate boulders in the near surface rock fill at some locations. The contractor should make provisions for excavation of the fill material, where required.

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

DRIVING PILES ADJACENT TO EXISTING BATTERED PILES – Item No.

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for driving piles within close proximity to existing battered piles (i.e., where the anticipated distance between the new pile tip at depth and the existing battered pile tip at depth is less than 20% of the existing pile length.)

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

Construction

Work under this item shall adhere to the following requirements:

- For new piles driven within the potential zone of interference with the existing abutment or wing wall piles (defined as a distance around the existing pile tip at depth equal to 10% of the pile length) the driving operations shall be continuously monitored by the QVE.

- The contractor shall cease driving of the pile if the QVE indicates that the driven pile may have come in contact with an existing pile.

- If contact between the new and existing piles is believed to exist the contractor shall take remedial action as directed by the Contract Administrator, which may include extracting the pile and re-driving or replacing the pile.

Basis of Payment

Payment at the contract price for the above noted Tender Item includes full compensation for all labour, equipment and materials to do the required work.

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during pile installation works. Piles include driven or drilled foundation piles or caissons, as well as piles installed for temporary roadway or railway protection.

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

Submission Requirements

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

Qualifications of vibrations monitoring specialist.
Proposed instrumentation.
Proposed location of instruments.
Proposed frequency of readings.
Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Monitoring

The Contractor shall take readings during driving of each pile. The readings should be taken and recorded during the entire length of driving and during seating of the pile on the bedrock.

The pile(s) furthest from the monitored structure or utility should be driven or drilled first to assess the the vibration level at the existing structures. If necessary, the contractor must alter the pile driving/drilling procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

For piles installed adjacent to the existing west pier foundations, vibrations shall not exceed 50 mm/s (peak particle velocity). Unless otherwise indicated, the measured vibrations for all remaining piles shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven/drilled prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving/drilling log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving/drilling procedures until the vibrations are within acceptable levels. The above process must be repeated for each pile.

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

PERIMETER WALL CONTROL BLASTING NEAR NEW AND EXISTING BRIDGE FOUNDATIONS - ITEM NO.

Special Provision

This special provision outlines the procedure to be used where rock excavation (blasting) is required within 50 m of the new and existing bridge foundations.

All blasting shall be in accordance with OPSS 120 except as noted herein.

- **Blasting** shall be considered synonymous with Controlled Blasting and is defined as the use of explosive materials with procedures and techniques to limit ground vibration velocities, flyrock, permanent ground displacement, air concussion, and overbreak, so as to prevent damage to existing structures, services and utilities, as well as new foundation areas.
- **Perimeter Wall Control Blasting** includes line drilling along the limits of the excavation in conjunction with smooth wall blasting, cushion blasting, buffer blasting or any other approved wall control blasting technique used to provide a smooth, straight final wall.
- Perimeter Wall Control Blasting techniques shall be employed within 50 m of the new and existing bridge foundations to ensure that overbreak and damage to the final rock faces adjacent to the new and existing structure is minimized and the number of drillhole traces in the final face is maximized.
- Adequate stemming and blasting mats shall be in place prior to blasting to prevent damage to the existing structures and pavement from flyrock.
- As part of the blast design submission requirements contained in OPSS 120, the Contractor shall prepare and submit their proposed Perimeter Wall Control blast design techniques.
- Prior to blasting within 50 m of the new or existing structures, the Contractor shall carry out trial blasts using their proposed Perimeter Wall Control technique to demonstrate that the blast design is adequate to minimize damage to the rock face, overbreak and fly rock.
- Results of the trial blast shall be reviewed by the Contract Administrator and the blasting methodology must be accepted by the Contract Administrator prior to blasting within 50 m of new or existing structures.
- Acceptance by the Contract Administrator of the Perimeter Wall Control blasting plan and trial blasts shall in no way relieve the Contractor from responsibility for ensuring that the Blasting Operation is conducted in a safe and satisfactory manner, and in accordance with these specifications, nor shall the Contract Administrator assume responsibility for the adequacy of the blasting to achieve adequate breakage or acceptable results.

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.

RIGID EXPANDED POLYSTYRENE EMBANKMENT – Item No.

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene backfill and associated works as shown on the Contract Drawings.

As part of the work under this item, the Contractor shall supply and place a 300 mm thick layer of Granular A, mortar sand, polyethylene sheeting and concrete top pad as shown on the Contract Drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

2.1 National Standard of Canada

CAN/CGSB – 51.20 M87

2.2 ASTM

ASTM D1621	Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM C203	Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Test Method for Measuring the Minimum Oxygen Content
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS – Ontario Provincial Standard Specifications

OPSS 212	Borrow
OPSS 501	Compaction
OPSS 517	Dewatering
OPSS 1010	Aggregates – Granular A, B, M, and Selected Subgrade Material
OPSS 1605	Expanded Extruded Polystyrene Pavement Insulation
OPSS 1860	Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturers' requirement.

6.3 Construction

The Contractor shall submit full details of the following:

a. The method of foundation excavation and preparation.

b. Construction of the 300 mm thick Granular A layer and the up to 100 mm thick mortar sand levelling pad.

c. The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.

d. The method and limits of placement of polyethylene sheeting.

e. The method of placement of 125 mm thick reinforced 30 MPa concrete top pad.

f. The method of placement of subbase material.

g. The method of placement of side slope cover.

6.4 Quality Verification Engineer

(1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

(2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Backfill, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of mortar sand with gradation and physical requirements as specified in OPSS 1004.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, telephone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
    1. Geometry
    2. Nominal Density
    3. Compressive Strength
    4. Flexural Strength
    5. Thermal Resistance
    6. Dimensional Stability
    7. Flammability
    8. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	 1200 x 600 x 300 With tolerances ± 1% 10 mm in 3 m ± 0.5% -3, +5	
Compressive Strength	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m^2.°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (mm)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1,200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be ± 1%. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 mm to +5 mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance with ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be 0.7 $m^2.°C/W$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25mm} = \frac{R_{measured}}{\text{thickness (mm)}} \quad x25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863.

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

7.3 Polyethylene Sheeting

The plastic sheeting shall be 6 mil polyethylene sheeting or equivalent.

7.4 Concrete Top Pad

The concrete top pad shall consist of 125 mm of reinforced 30 MPa concrete.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendations.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the Drawings. Any softened, loosened or deleterious materials at the foundation/base elevation shall be subexcavated and replaced with OPSS 1010 Granular "A" material.

9.2 Levelling Pad

Place, level and compact a 300 mm thick layer of Granular A followed by an up to 100 mm thick layer of mortar sand material in accordance with OPSS 501 to within $\pm$ 30 mm of the design elevation. The levelling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The levelling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

(1) The individually marked blocks shall be placed on the prepared levelling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary. Contractor shall ensure all trimmed material is disposed of in accordance with all applicable regulations and that no trimmed debris enters any watercourse.

(2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.

(3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between

blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.

(4) Sloping end adjustments at the abutments shall be accomplished by levelling terraces in the subsoil in accordance with the block thickness.

(5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.

(6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.

(7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.

(8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

(9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

(10) The top surface and side surfaces of the expanded polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirements.

11.0 QUALITY ASSURANCE

11.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and test will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

11.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. At a minimum, three blocks shall be tested.

11.3 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

Measurement will be by volume in cubic metres of rigid expanded polystyrene material measured in its original position based on theoretical dimensions.

13.0 PAYMENT

13.1 Basis of Payment

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