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REPORT ON

DETAILED
FOUNDATION INVESTIGATION AND DESIGN
REHABILITATION OF THE HIGHWAY 401
SALMON RIVER BRIDGE, SITE NO. 11-207
G.W.P 82-98-00
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 8, KINGSTON

Submitted to:

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GEOCRES NO. 31C-168

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Lea Associates Ltd. (Lea) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a detailed foundation investigation as part of the detailed design for the rehabilitation of the Salmon River Bridge at Highway 401.

The terms of reference for the scope of work are outlined in Golder's proposal P21-1556, dated February 2003, that forms part of the Consultant's Agreement (Number P.O.4005-A-000213) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated April 28, 2003. The general arrangement drawing for the proposed rehabilitation of the bridge structure at the Salmon River was provided to Golder by Lea in June 2003.

The purpose of the investigation is to establish the subsurface conditions at the area of the bridge abutments and approach embankments. The piers were investigated as part of the Contingency 'A' work carried out under a separate agreement. The specific location of the investigation site is shown in plan on Drawing 1. The investigation was supplemented with information contained in the following report:

- Foundation Investigation Report, Tyendinaga Bridge No. 5 Crossing the Salmon River, Geocres No. 31C-131, dated May 16, 1955.

2.0 SITE DESCRIPTION

The existing Salmon River bridge is located on the four-lane stretch of Highway 401 between Belleville and Napanee, Ontario. The bridge is a 31 m wide, 5 span, 126 m long bridge with a 30 degree skew to the highway. The existing bridge was constructed in 1956 and was last rehabilitated in 1983.

The bridge site is located within the Salmon River valley and the terrain generally consists of open fields, bush areas, swamp areas (in the river valley), and limestone bedrock outcrops to the east and west of the site. The ground surface within the limits of the existing bridge and embankment area slopes from about Elevation 83 m at the east end of the bridge to 86 m at the west end.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Salmon River Bridge investigation was carried out between May 6 and May 9, 2003 and between July 14 and July 18, 2003. A total of nine (9) sampled boreholes were advanced at the site. All of the borehole locations and elevations of the investigated areas are shown in detail on Drawing 1.

Four (4) boreholes were drilled at the east and west abutment footing locations, north and south corners. All of the boreholes were advanced to refusal on inferred bedrock and were cored a minimum of 3 m into the bedrock. Two (2) boreholes were advanced to refusal on bedrock at the east and west approach embankments. Three (3) boreholes were drilled at the north and south side of Pier A and the south side of Pier D.

The field investigation was carried out using track-mounted CME 55 drill rigs supplied and operated by Walker Drilling Ltd. of Utopia, Ontario and by Marathon Drilling of Ottawa, Ontario. The boreholes put down with the drill rig were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Shelby tubes and in-situ vanes (N vanes) were obtained at regular intervals of depth through the soft stratum. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

All boreholes were advanced to depths ranging from 7.6 m to 21.3 m below the existing ground surface (including rock coring). The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in selected boreholes to permit monitoring of the groundwater level at these locations. The piezometers consist of a 25 mm outside diameter rigid PVC tubing with a 0.3 m long slotted tip that is sealed at a selected depth within the boreholes. The holes were backfilled with bentonite mixed with soil cuttings; typically one bag of bentonite was used per 3 m of hole backfilled. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg Limits and grain size distribution) as well as oedometer testing was carried out on selected samples.

On completion of the fieldwork, all investigated borehole locations were surveyed using the NAD 83 MTM (Zone 12) co-ordinate system and the geodetic datum for elevation. The surveying of the elevations of the as-drilled boreholes was carried out by members of our engineering staff, referenced to benchmark geodetic elevations provided by Lea. The northing and easting coordinates of the borehole locations were provided by Lea.

Boreholes 6 and 12 to 15 were advanced at the bridge site in 1955 by Racey MacCallum and Associates Ltd. The approximate locations of these boreholes are shown on Drawing 1 and labelled as 55-6, 55-12 to 55-15. The locations of the boreholes are estimated from the drawing provided in the original 1955 report and may differ from the actual location.

The 1955 boreholes were advanced to between depths of 12.14 m to 18.3 m (including rock coring). No piezometers were installed in these boreholes.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located in the physiographic region known as the Napanee plain. The Napanee plain is a flat to undulating limestone plain from which glaciers have stripped most of the overburden. The soils are generally only a few inches thick over much of the region, with some deeper glacial till occurring in the stream valleys (Chapman and Putman, "The Physiography of Southern Ontario", 3rd Edition, 1984). The glacial tills in the valleys are overlain by a variety of deposits of fluvial origin. The overburden at the site is underlain by limestone to shaley limestone from the Trenton and Black River Groups.

4.2 Subsurface Conditions and General Overview

The detailed subsurface soil 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 samples, are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy as encountered from the boreholes at the bridge location are shown on Drawings 1 and 2.

In general, the subsoils at the site consist of a surficial layer of topsoil underlain by alternating thin layers of sand and silty clay containing organics, or occasionally gravel containing organics. The surficial soils are underlain by a layer of silty clay underlain by a clayey silt to clay deposit. The clayey silt to clay deposit is underlain by limestone bedrock. The total overburden thickness ranges from 6.4 m at the southeast abutment and approaches to about 18.3 m below the ground surface in the centre of the valley. Bedrock is outcropped beyond the limits of the bridge. All of the boreholes, except the two at the approach embankments, were cored at least three metres into the bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt and Fill

About 0.8 m of an asphalt containing sand and gravel and organics was encountered below the ground surface in Borehole 4, underlain by about 0.4 m of sand and gravel fill which contained some organics. The surface of the asphalt was at Elevation 79.8 m. The asphalt was mixed with organics and roots and heavy grinding was encountered during augering.

Standard Penetration Testing (SPT) measured 'N' values ranged between 5 to 15 blows per 0.3 m of penetration, indicating a loose to compact state of packing.

The natural water content measured on one sample of this fill material was 17 percent.

4.2.2 Topsoil

Topsoil was encountered at the existing ground surface in Boreholes 1, 2, 3, 5, 8 and 9. The surface of the topsoil ranged between Elevation 79.4 m to 81.2 m and was about 0.1 m to 0.2 m thick.

4.2.3 Sand and Gravel to Sandy Silt

A surficial deposit of loose grey sand containing trace gravel was encountered below the topsoil at Borehole 1. The sand was 0.6 metres thick, and its surface was encountered at Elevation 80.1 m. A loose to compact grey sand and gravel deposit was observed below the topsoil in Boreholes 2 and 8. The sand and gravel was 0.7 metres thick, and its surface was encountered at Elevation 80.5 m and 79.7 m in Boreholes 2 and 8, respectively.

Surficial deposits were encountered during the 1955 drilling program in Boreholes 55-6 and 55-12 to 15. Very soft or very loose sandy silt to sand deposits containing organic material were observed at Boreholes 55-6, 55-12, 55-14 and 55-15. At Borehole 55-13, a loose gravel with clay and organic material was encountered. These surficial deposits ranged in thickness between 0.9 and 2.7 metres, and the top of the deposits were encountered between Elevation 77.6 m and 80.3 m.

At the borehole locations, Standard Penetration Testing (SPT) measured 'N' values ranged between 1 and 12 blows per 0.3 m of penetration, indicating a very loose to compact state of packing.

4.2.4 Silty Clay

A layer of silty clay was encountered at ground surface or underlying the topsoil and/or sand to sand and gravel in Boreholes 1 to 9. The black-brown to grey silty clay contained trace sand and gravel as well as roots and decayed wood fragments. The top of the silty clay deposit was encountered between Elevation 78.6 and 80.4 m in the boreholes, and the thickness varied from 0.7 m to 2.8 m.

SPT measured 'N' values within the silty clay ranged between 0 (weight of hammer) and 31 blows per 0.3 m of penetration, indicating a very soft to hard consistency. The higher 'N' values were encountered in Boreholes 3 and 5, located at the approaches to the bridge.

The natural water contents measured on samples of the sandy silt and silty sand were generally 14 to 35 percent; however, one sample in Borehole 9 had a measured natural water content of 135 percent. The high moisture contents can be attributed to the organics noted throughout the deposit.

Silty clay deposits containing organics were not identified in the 1955 report.

4.2.5 Sand

A thin layer of sand was encountered in Boreholes 1, 5 to 7 and 9, below the surficial silty clay deposit. The deposit is comprised of brown to black to grey, fine to coarse, sand to silty sand to sand and gravel and containing trace to some organics. The top of the deposit was encountered between Elevation 77.6 m and 78.8 m in the boreholes. The deposit ranged in thickness between 0.4 m and 1.8 m.

SPT measured 'N' values ranged between 1 and 8 blows per 0.3 m of penetration, indicating a very loose to loose state of packing.

The natural water content measured on selected samples of this deposit were generally between 21 and 23 percent; however, one sample in Borehole 9 had a measured natural water content of 78 percent, likely due to the presence of organics in the sample.

This sand deposit was not identified in boreholes from the 1955 report.

4.2.6 Silty Clay

Below the surficial silty clay and/or sand deposits in Boreholes 1 to 4 and 7, a deposit of mottled brown to grey silty clay containing trace to some sand and trace gravel was encountered. The stiff clay layer underlying the surficial sandy silt deposits encountered in Borehole 55-6 can be considered to be part of this deposit. The elevation of the top of this layer varied from 77.2 m to 79.8 m, and the layer ranged in thickness from 0.8 m to 2.0 m.

At the borehole locations, SPT measured 'N' values ranged between 3 and 45 blows per 0.3 m of penetration, and generally less than 16 blows, indicate a soft to hard consistency within this deposit. Generally, the deposit has a stiff to very stiff consistency.

Atterberg limits testing was carried out on two samples of the silty clay deposit from Boreholes 1 and 4. The test results are summarized in the following table.

Borehole	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*
1	5	77.3 – 76.6	47.1	19.5	27.6
4	6	76.0 – 75.4	46.1	19.2	26.9
Average	-	-	46.6	19.4	27.2

Grain size distribution curves for selected samples from this deposit are shown on Figure A1 in Appendix A. The results of the Atterberg limits testing are shown on the plasticity chart on Figure A2 in Appendix A. The natural water content measured on selected samples of this deposit ranged between 27 percent and 38 percent, with an average of 32 percent.

4.2.7 Clayey Silt to Clay

A deposit of grey clayey silt to clay was encountered beneath the silty clay or surficial sandy or clayey deposits in all the boreholes. The elevation of the top of this deposit varied from 75.2 m to 77.5 m and the stratum ranged between 3.5 m and 11.4 m in thickness. The lower clayey silt portion of the deposit is typically softer than the upper silty clay to clay portion. The surface of the clayey silt varied between Elevation 71.2 m and 76.6 m. The softer lower portion of the deposit was not encountered in Boreholes 4 and 5. In general, the deposit becomes siltier with depth. During Shelby tube extraction, it was noted that the clayey silt was layered deposit contains 1 cm to 2 cm layers of dark grey clay as well as zones up to 15 cm thick of silt.

This deposit was encountered in all boreholes from the 1955 report and is described as silt or silt-clay. The elevation of the top of this deposit varied from 75.6 to 76.8 metres, and was between 3.7 m to 11.5 m in thickness. The lower portion of the deposit is typically softer than the upper portion. The surface of the softer lower portion varied between Elevation 71.3 m and 75.6 m. The softer lower portion of the deposit was not encountered in Borehole 55-14.

At the borehole locations, the upper portion of the deposit had measured SPT 'N' values ranging from 4 to 14 blows per 0.3 m of penetration, suggesting a firm to stiff consistency. The lower portion of the deposit had SPT measured 'N' values ranging from weight of hammer to 6 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. A grain size distribution curve for one sample of this deposit is shown on Figure A3 in Appendix A.

Field vane testing was carried out within this deposit using a standard MTO 'N' vane and a torque wrench and the results are summarized in Table 1. The results of field vane tests indicate that the upper portion of the deposit has a stiff to very stiff consistency while the lower portion of the deposit has a soft to stiff consistency.

The results of Atterberg limits testing carried out on samples of the clayey silt to clay deposit from all the boreholes are summarized in Table 2. The results of the Atterberg limits tests are plotted on the plasticity chart on Figures A4 and A5 in Appendix A for the upper and lower portions of the deposit, respectively. Typically, in the lower portion of the deposit, the natural water content was higher than or at about the liquid limit which is consistent with the undrained shear strength measured. Based on the results of the Atterberg limits testing, the upper portion of the deposit is classified as a silty clay to clay while the lower portion is a clayey silt.

Laboratory oedometer (consolidation) testing was carried out on two specimens of the silty clay and clayey silt obtained from Borehole 2. Details of the test results are shown on Figures A6 to A9 in Appendix A and the results summarized in the table below. The details of the test results from BH 55-12 are given in Appendix B, following the borehole logs. It should be noted that the preconsolidation pressures shown on the original consolidation test data have been re-evaluated and the estimated values are given below:

Borehole and Sample No.	***Elevation (m)***	σ_{vo}' ***(kPa)***	σ_p' ***(kPa)***	***OCR***	e_o	C_r	C_c	c_v* ***(cm²/s)***
BH 2 Sa# 8	74.2 (6.4 m depth)	59	130	2.2	1.06	0.048	0.391	0.005
BH 2 Sa# 10	71.2 (9.4 m depth)	95	180	1.9	0.732	0.023	0.23	0.005
BH 55-12** Sa# TW-2	74.6 (2.5 m depth)	50	230	4.6	0.775	n/a	0.15	n/a
BH 55-12** Sa# TW-4	72.2 (4.9 m depth)	72	170	2.4	0.738	n/a	0.2	n/a
BH 55-12** Sa# TW-6	69.2 (7.9 m depth)	100	180	1.8	0.665	n/a	0.35	n/a

Note: * For stress range of $20 \leq \sigma_v' \leq 300$ kPa
** 1955 boreholes; depth related to depth below river bed level.

where:
- σ_{vo}' is the effective overburden pressure in kPa
- σ_p' is the preconsolidation pressure in kPa
- OCR is overconsolidation ratio
- e_o is initial void ratio
- C_c is the compression index (based on void ratio)
- C_r is the recompression index
- c_v is the coefficient of consolidation in cm^2/s

The natural water content measured on samples of the clayey silt to clay range from about 20 to 66 percent, and are generally less than about 40. The lower water contents are generally within the clayey silt (lower) portion of the deposit.

4.2.8 Silty Sand

A thin deposit of silty fine sand was encountered in Boreholes 6 and 7. In Borehole 6, this deposit was encountered below the clayey silt and in Borehole 7, this deposit was encountered within but near the bottom of the clayey silt deposit. The top of the deposit was encountered between Elevation 65.4 m and 68.1 m in Boreholes 6 and 7, respectively. The deposit ranged in thickness between 0.5 m and 1.4 m.

The measured SPT 'N' value for both samples of the deposit was weight of hammer, indicating a very loose state of packing.

The natural water content measured on one selected sample of this deposit was 24 percent.

This sand deposit was not identified in boreholes from the 1955 report.

4.2.9 Silty Sand to Sand and Silt (Till)

In Boreholes 5, 8 and 9, a deposit of till comprised of silty sand with gravel to sand and silt containing trace clay and gravel was encountered below the clayey silt to clay deposit between Elevation 71.8 m and 74.0 m. The deposit was about 0.8 m to 0.9 m thick. A grain size distribution curve for samples of this deposit is shown on Figure A10 in Appendix A.

This deposit was also encountered in Borehole 55-13 at Elevation 68.6 m; however, the deposit was described as silt and silt clay with gravel and some boulders. The deposit was 3.1 metres thick, and the boulders were encountered in the bottom 1.4 m of this deposit.

At the recent borehole locations, measured SPT ‘N’ values within the till were between 14 blows and greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense state of packing. From the casing penetration in Borehole 55-13, it is inferred that this layer was very dense/hard.

The natural water content measured on selected samples of this deposit were between 5 and 18 percent.

4.2.10 Bedrock

Bedrock was encountered in all the boreholes and cored in Boreholes 1, 2, 4 and 6 to 9. The presence of bedrock was inferred from refusal of further drilling advance in the Boreholes 3 and 5. The elevation and depth of the bedrock surface is given in the table below:

Borehole	*Location*	*Ground/Water Surface Elevation (m)*	*Bedrock Depth (m)*	*Bedrock Surface Elevation (m)*
1	Southwest Abutment	80.3	15.2	65.1
2	Northwest Abutment	80.6	11.1	69.5
3	Northwest Approach	81.2	7.6	73.6
4	Northeast Abutment	79.8	9.2	70.6
5	Northeast Approach	80.6	8.2	72.4
6	Pier A (South)	79.5	14.6	64.9
7	Pier A (North)	79.2	13.7	65.5
8	Southeast Abutment	79.8	6.4	73.4
9	Pier D (South)	79.4	8.4	71.0
55-2	East Approach	80.5	3.6	76.9
55-6	Pier B (South)	80.1	14.7	65.4
55-12	Pier A	79.9 (water)	15.4	64.5
55-13	Pier D (North)	79.7 (water)	14.2	65.5
55-14	Pier D (South)	79.5 (water)	9.1	70.4
55-15	West Approach	80.3	8.4	71.9

The bedrock samples obtained consist of grey, slightly weathered to fresh, strong, thinly to finely laminated to massive, fine to medium grained limestone, with shaley partings and laminae. The bedding planes were approximately horizontal. The Total Core Recovery was between 67 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples in Boreholes 1 to 5 ranged from about 0 to 12 percent, indicating a rock mass of poor to fair quality, however, it is considered that the low RQD values in these boreholes may be attributed in part to the rock coring procedures causing core breaks parallel to bedding planes. In Boreholes 6 to 9, the RQD measured in the core samples ranged from 59 to 90 percent, indicating a rock mass of fair to good quality. In general, based on all of the rock core information available, the rock mass is considered to be good quality.

Point load strength tests were performed on selected samples of the rock core from Boreholes 1, 2, 6 and 9. Axial and diametral point load strength index values are shown on the Record of Drillhole Sheets. Approximate diametral point load UCS (unconfined compressive strength) values range from 27 MPa to 148 MPa with an average of 72 MPa. The axial point load UCS values range from 51 MPa to 114 MPa with an average of 85 MPa. Using the Intact Rock Strength Classification table, these values indicate that the rock strength is medium strong to very strong parallel to bedding planes, and strong to very strong perpendicular to bedding planes.

4.2.11 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be moist to wet. Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. Water levels were observed throughout the drilling operations are noted in the table below:

Borehole	*Ground Surface Elevation (m)*	*Water Level Depth (m)*	*Water Level Elevation (m)*	*Comments*
1	80.3	0.6	79.7	Open borehole after completion of drilling
2	80.6	0.1	80.5	Shallow piezometer (July 14, 2003)
		1.1	79.5	Deep piezometer (July 14, 2003)
3	81.2	4.0	77.2	Open borehole after completion of drilling
4	79.8	0.7	79.1	Piezometer (July 17, 2003)
5	80.6	1.5	79.1	Open borehole after completion of drilling
6	79.5	0.7	78.8	Piezometer (2 days after installation)
7	79.2	0.0	79.2	Open borehole after completion of drilling
8	79.8	1.0	78.8	Open borehole after completion of drilling
9	79.4	0.4	79.0	Piezometer (after installation)

The above water levels are consistent with the adjacent river water level. The normal water level in the Salmon River fluctuates between about Elevation 79 m and 80 m. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

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PART B
DETAILED
FOUNDATION INVESTIGATION AND DESIGN
REHABILITATION OF THE HIGHWAY 401
SALMON RIVER BRIDGE, SITE 11-207
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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed rehabilitation/widening of the Salmon River Bridge structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site and the boreholes from the original report. The interpretation and recommendations provided are intended only 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. 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.

It is understood that the proposed rehabilitation of the existing 5-span Salmon River Bridge will involve widening of between 2.0 m and 2.8 m on both sides of the structure. This will require the abutment footings and the approach embankments to be widened. Widening of the pier footings is not anticipated at this time. The existing embankments are up to about 3 m and 6 m in height on the east and west approaches, respectively.

5.1 General

The Salmon River Bridge is a 5-span structure that was erected in 1956. The foundations for the piers and abutments consist of 14BP73 (HP360x108) piles driven to refusal on the limestone bedrock. The existing wing walls are also founded on piles.

Different foundation alternatives for the abutment footing widening were considered and a summary is presented in Table 3, following the text of this report. It was considered that steel H-piles driven to refusal on the limestone bedrock for support of the widened abutments as per the existing bridge is the most feasible option from a foundation perspective.

5.2 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone bedrock may be used for support of the widened abutments. It is assumed that the widened foundation will be constructed to match the existing. Therefore, the pile cap base will be at Elevations 77.1 m and 78.3 m at the east and west approaches, respectively. The proposed Highway 401 grade at about Elevation 82.8 m and 85.9 m at the east and west approaches, respectively. The anticipated pile length at the abutment widening locations is given in the table below:

Foundation Location	*Bedrock Surface Elevation (m)*	*Anticipated pile Length (m)*
Northeast	70.6	6.5
Southeast	73.4	3.7
Northwest	69.5	8.8
Southwest	65.1	13.2

5.2.1 Axial Geotechnical Resistance

5.2.1.1 Existing Piles

No pile driving records are available from the bridge construction, however it assumed that the piles were driven to refusal on limestone bedrock. The geotechnical capacity of piles driven to refusal on the limestone bedrock at the site will be greater than the structural capacity of the pile and as such, the structural capacity of the pile will govern. The structural capacity of the pile is generally calculated by using the yield strength of the steel and applying factors for structural and below-ground design and multiplied by the steel tip area. Historically, according to the CHBDC, steel manufactured in the 1950's had a yield strength of about 230 MPa, which is much lower than that of steel manufactured today. Test data on the steel used at the site indicates that the minimum yield strength of about 275 MPa. The estimated range of ultimate factored structural capacity of a single 14BP73 is pile between about 2,100 kN and 2,500 kN, based on a yield stresses of 230 MPa and 275 MPa, respectively. However, the structural capacity used should be calculated by a structural designer. Typically, for Ultimate Limit States (ULS) design, a value of 2,000 kN may be used for 14BP73 piles driven to refusal on limestone bedrock.

Based on the recommendations in Geocres No. 31C-131 it could be assumed that the original piles were to be driven to refusal on the limestone bedrock at the site. However, it is possible that some piles may have reached the required set in the gravely till material that was encountered above the bedrock. It is not possible to know with certainty where this may have occurred since pile driving records are not available. However, silt to silt clay with gravel and some boulders was encountered above the bedrock surface in Borehole 55-13. A sandy silt to sand and silt till deposit was encountered above the bedrock in Boreholes 5, 8 and 9. All boreholes where the till was encountered were located towards the east end of the site (i.e. east of Pier D). In addition, based on the anecdotal information, some piles, particularly at the east abutment, may have had problems seating into the bedrock due to the sloping nature of the bedrock profile. In general, the bedrock slopes towards the centre of the valley. The west abutment and pier A indicate bedrock sloping towards the south while Pier D and the east abutment indicate that the bedrock surface dips towards the north.

If the piles did not reach the bedrock surface, the axial capacity of the piles would be governed by the geotechnical capacity of the sandy silt till (encountered in the recent boreholes) to silty clay till material (encountered in the 1955 boreholes) and not the structural capacity. As noted above, this material was encountered in Borehole 55-13 and Boreholes 5, 8 and 9; measured 'N' values indicate the deposit is compact to very dense. The factored axial resistance of the steel 14BP73 piles terminated in the lower till may be taken as 1100 kN for Ultimate Limit States (ULS) and 750 kN for Serviceability Limit States (SLS) for 25 mm of settlement.

The additional boreholes drilled at the east pier (Pier D) and the south side of the east abutment confirm that there is till present but that the thickness of the deposit is less than 1 m. This thickness is not considered large enough to allow the piles to 'hang-up' within the deposit.

At most borehole locations, where the lower till deposit was not encountered, the clayey silt soil directly overlying the bedrock is of a very soft to stiff consistency. At Pier A, a nominal thickness of very loose silty sand was encountered in the boreholes above the bedrock. Based on the consistency of these deposits, it is expected that there would have been nominal resistance to driving the H-piles and the piles would have been driven to the bedrock surface. As such, the structural capacity of the pile will govern the design; however, there may have been difficulties in seating the piles on the sloping bedrock which would normally result in a lower capacity. In this regard, a factored axial resistance at ULS of 1800 kN should be assumed.

5.2.1.2 New Piles

For HP 310 x 110 piles driven to refusal in the shaley limestone bedrock encountered under the abutment widening locations, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value takes into account the structural capacity limitation of the pile, and potential difficulties that the pile may have seating into the bedrock surface that may be variable and inclined. Rock points should be provided to the pile tips to ensure penetration and adequate seating as per current MTO practice (Standard OPSD 3301.00 and OPSS 903.07.02.05).

A Seviceability Limit States (SLS) value is not provided because the limestone bedrock is considered to be an unyielding material. Under these conditions, and for HP 310 x 110 pile lengths of less than 35 m, the SLS values (for 25 mm of settlement) do not govern design because the SLS value is higher than the ULS value.

Pile installation should be in accordance with SP903S01. For this site, the piles will essentially be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be equipped with rock points and driven to bedrock. The pile termination or set criteria will be 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, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

5.2.2 Downdrag Load (Negative Skin Friction)

Due to the additional load from the widening of the approach embankments at the abutments, consolidation settlement of the underlying clayey silt to clay deposits will take place. The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clay deposit will take place. Where the piles are end-bearing on bedrock, a small amount of settlement of the clay relative to the pile will result in the development of negative skin friction acting on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments. Downdrag loads will be induced on the new piles as well as on the existing piles at the abutments. The abutment pile design should be based on the full downdrag load acting on the piles.

The estimated unfactored downdrag loads acting on a single pile over the length of pile within the native soils are summarized in the following table. All of the new piles and the outer piles within the existing pile group will experience downdrag loads since the embankment widening will also induce consolidation settlement under the existing abutment pile cap. Therefore, all existing piles should include the full downdrag load in the analysis. The loads given are the estimated unfactored downdrag loads acting on the new HP 310 x 110 steel piles and HP 360x108 steel piles for the existing structure.

	Abutment Foundation Location	**Unfactored Downdrag Load (kN)**
Proposed HP310x110 Piles	Northeast	200
	Southeast	150
	Northwest	255
	Southwest	350
Existing 360x108 Piles	East	240
	West	300 to 420

The load calculated in this manner is a unfactored load. 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* for ULS conditions.

Downdrag loads could be reduced if lightweight fill was used in construction of the widened embankments. However, given the relatively small amount of widening it would require considerable sub-excavation and replacement of the existing embankment fill in order to obtain sufficient benefit from lightweight fill.

5.2.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the existing piles subjected to lateral loads (eg. ice loads) should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The pile should be modelled as a beam-column supported by springs equivalent to the passive soil reaction distributed along the shaft. The passive resistance developed for lateral deformations typical of bridge foundations is generally much less than the passive pressure associated with a full passive resistance. This full passive resistance is calculated from earth pressure theories assuming unlimited deformation of the soil. The lateral resistance of the pile may be limited by the factored structural flexural resistance of the pile rather than the resistance of the soil.

Therefore, in order to develop the full passive resistance, the pile would have to deflect a ‘large’ amount. For piles ‘fixed’ within the pile cap, the magnitude of possible deflection is further reduced and the horizontal geotechnical resistance of the pile is some fraction of the full passive resistance occurring at relatively small horizontal displacements.

It can be assumed based on the shear strength of the soil, that the pile can be considered a laterally supported compression member. The horizontal load capacity of vertical piles may be limited in three different ways:

- The capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
- The bending moments may generate excessive bending stresses in the pile material, resulting in structural failure of the piles; or
- The deflections of the pile heads may be too large to be compatible with the superstructure.

CFEM (1992) gives two methods by which to assess the lateral capacity of a pile. The first is Brom's Method (1964), which examines failure criteria (i.e ultimate horizontal resistance) for two types of piles - 'short piles' where the lateral capacity of the soil adjacent to the pile is fully mobilized and 'long piles' where the bending resistance of the pile is fully mobilized. In the case of the Salmon River Bridge, the long piles are the governing case. Figure 20.10 in the manual, for cohesive soils, gives the graphical solution for the ultimate lateral resistance of the pile based on the yield moment of the pile and shear strength of the soil.

The second method examines the lateral deflections of the pile by using the horizontal subgrade reaction theory where the soil around a pile is modelled using a series of springs. The spring constant is called the coefficient of horizontal subgrade reaction, k_h (kN/m^3). The value of k_h is used as an input parameter into your elastic soil-structure interaction model. For cohesive soils, the coefficient of horizontal subgrade reaction to a vertical pile can be estimated using the following formula:

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

where:

k_h = coefficient of horizontal subgrade reaction (kPa/m)
s_u = undrained shear strength of the soil (kPa)
d = pile diameter (m)

Based on the above discussion, it is considered that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case. For both the existing piles (HP 360x108) and the proposed piles (HP310x110) driven to bedrock through the soft to stiff clayey silt to clay at this site, the horizontal resistance at Ultimate Limit States (ULS) will be controlled by structural limitations such as the yield moment (M_{YIELD}) of the pile (i.e. Brom's 1964 method). At Serviceability Limit States (SLS), the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil.

The recent undrained shear strengths measured in the recent boreholes completed to date at the abutments and the approaches are shown on Figure 1. As is evident from this figure, there is considerable scatter in the strength profile. In this regard, the lateral capacity analysis should be carried out using the following design strength profiles.

Location		***Borehole Number***	***Undrained Shear Strength, s_u***	***Elevation (m)***
West Abutment	North Side	2	100 kPa 25 kPa	above 75.0 m between 75 m and 69.5 m (bedrock surface)
	South Side	1	100 kPa 30 kPa	above 71.5 m between 71.5 m and 65 m (bedrock surface)
Pier A	North Side	7	100 kPa 35 kPa	above 71.6 m between 71.6 m and 65.5m (bedrock surface)
	South Side	6	100 kPa 35 kPa	above 71.9 m between 71.6 m and 64.9m (bedrock surface)
Pier D	North and South Sides	9	100 kPa	to bedrock surface
East Abutment	North Side	4	100 kPa 50 kPa	above 72.5 m between 72.5m and 67.5 m (bedrock surface)
	South Side	8	100 kPa	to 73.4 m (bedrock surface)

The upper zone of soil (down to a depth below the pile cap equal to about 1.5 x d after Brom's 1964, where d = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

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

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

5.2.4 Frost Protection

The pile caps should be provided with a minimum of 1.6 m of soil cover for frost protection.

5.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the 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 walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. 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 1.6 m behind the back of the wall stem (Case I in Figure C6.9.1(l) 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 II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- 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	0.27	0.31
At rest, K_o	0.43	0.47

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

- 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 the National Building Code of Canada, this site is located on the border between Seismic Zones 1 and 2. For design purposes, the bridge site can be considered to be in Seismic Zone 1. The site-specific zonal acceleration ratio for the Napanee area is 0.05. Based on experience, for the subsurface conditions at this site, up to 100 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.10g to between 0.05g and 0.10g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.10.

- 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, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.05$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.15$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

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

obtained using the k_h and three values of k_v as described above. 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.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.33	0.28	0.31
Non-yielding wall	0.42	0.35	0.39

Note: These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta = \varphi'/2$).

- 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 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:

$$K_a \gamma' d + (K_{AE} - K_a) \gamma' H$$

where:

K_a is the static active earth pressure coefficient;
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3) as given on page 23;
d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

5.4 Approach Embankment Design and Construction

The widening of the Salmon River Bridge approach embankments will require additional fill placement. The current embankment height is between 3.0 m and 6.0 m at the east and west approaches, respectively. It is assumed that the current highway grade at the approach embankments will remain the same. The proposed widening will be between 2.0 m and 2.8 m on both the north and south sides in order accommodate traffic staging during rehabilitation of the structure.

5.4.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from the existing embankment side slopes and below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent 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.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended. A large amount of seepage and softened embankment soils were noted during the minor excavation which was carried out for drill rig access on the southeast approach embankment (greater than 100 m east of the structure). It is recommended that provision be made in the contract to install drainage measures (such as a filter cloth, granular blanket and/or French drains) where seepage is encountered during preparation for widening.

Since the design high water level encroaches on the embankment side slopes near the river, rip-rap should be placed on the slopes to at least 0.5 m above the design flood level.

5.4.2 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed approach embankment height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 3 m to 6 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability. Static slope stability analyses for this embankment configuration were carried out using the following parameters, based on field and laboratory test data and accepted correlations:

Soil Deposit	***Bulk Unit Weight***	***Effective Friction Angle***	***Undrained Shear Strength***
Embankment Fill	22 kN/m^3	32°	–
Surficial Silty Sand to Sand and Gravel	20 – 22 kN/m^3	32°	–
Surficial Silty Clay	19 – 21 kN/m^3	–	75 – 100 kPa
Silty Clay to Clay	18 kN/m^3	–	100 kPa
Clayey Silt	20 kN/m^3	–	25 kPa

5.4.3 Approach Embankment and Settlement

Settlement of the approach embankment subgrade can be expected due to compression of the surficial sand to silty sand and sand and silt till strata, as well as consolidation of the firm to very stiff silty clay deposit that was encountered at the north abutment / approach embankment. Settlement analyses were carried out using the commercially available computer program Unisettle.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the new embankment fill itself is expected to be less than 25 mm. The use of granular fill for the embankment widening would reduce this settlement since the majority of settlement of granular fills will occur during construction – the majority of the settlement of cohesive fill would occur after construction. Given the relatively thin zone of embankment fill to be placed, it is recommended that granular fill be used particularly if the new edge of pavement straddles over the existing crest of the embankment. In order to minimize differential settlement between the existing and widened portion of the embankment, the newly placed embankment fill should be keyed into the existing embankment as per OPSD 208.01.

The settlement of the embankments as a consequence of consolidation settlement of the clayey silt to clay deposit is expected to be less than 50 mm. This maximum settlement will occur under the crest of the new widened embankment and represents the total differential settlement with respect to the existing embankment; however, the differential will be gradual due to the configuration of the widening and the anticipated pressure bulb.

5.5 Excavations and Temporary Cut Slopes

Excavations for construction of the pile cap at the west abutment, the excavation will typically extend through 0.6 m of loose sand to sand and gravel, overlying 0.7 m to 1.2 m of firm to hard silty clay containing organics. At the east abutment, the excavation for the pile cap will extend through about 1.2 m of loose to compact sand and gravel fill containing asphalt, overlying 1.4 m of soft to firm silty clay containing organics. At both locations, the base of the excavation will generally be formed within the underlying stiff to very stiff silty clay; however, loose brown sand was noted in Borehole 1 at the south limit of the west abutment at about the founding level. Above the original ground surface (about Elevation 80 m), the excavations for the abutments as well as the wing walls will extend back through the embankment fill. Excavations for the wing walls will extend through the embankment fill materials. The excavation for pile cap construction adjacent to the existing pile cap will be 2 m and 3 m deep below existing ground at the west and east abutments, respectively.

5.5.1 Open Cut

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 excavations are expected to be up to about 3 m below the original ground surface at the east and west abutments. The surficial sands, silty sands and silty clay soils are classified as Type 3 soil, according to the OHSA and the excavations through theses deposits will be below the groundwater table. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V) through the sands and 2H:1V through the silty clays. Steeper side slopes may be possible if dewatering can be achieved.

5.5.2 Excavation Support

Due to the configuration of the abutments and wing walls, space restrictions may not permit excavations for the widened approach embankments, wing walls and pile caps to be made within open cut. Therefore, it is anticipated that temporary roadway protection may be required. In addition, roadway protection may be required for the removal of the existing, pile-supported wing walls. Based on the subsurface conditions at the site and the likely excavation geometry, it is anticipated that a soldier pile and lagging system using anchors or rakers to provide lateral support would be suitable. Where the temporary shoring extends below the groundwater table, the lagging should be wrapped with filter cloth to prevent loss of fines.

The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

5.5.3 Groundwater and Surface Water Control

The groundwater level at the site is expected to be at relatively shallow depth below the existing ground surface at about Elevation 80 m; further monitoring of the piezometers is required to confirm the levels. Excavations to construct the pier cap extension, will require groundwater control. Given the available space and the required depth of excavation, it is likely that open-cut excavations with sufficient sumping from properly filtered sumps will adequately control the groundwater. In this case, however, the excavation side slopes will have to be maintained at about 3H:1V or flatter. Alternatively, consideration could be given to the use of a sheet piling wall to provide a cut-off for groundwater inflow.

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TABLE 1
RESULTS OF IN-SITU VANE STRENGTH TESTING
CLAYEY SILT TO CLAY DEPOSIT

Borehole Number	*Elevation/ Depth (m)*	*Undrained Shear Strength (kPa)*	*Remoulded Shear Strength (kPa)*	*Sensitivity*	*Location*
Silty Clay to Clay					
1	74.7/5.6	57	10	5.7	SW Abutment
1	74.4/5.9	>150	n/a	n/a	SW Abutment
1	73.1/7.2	>150	n/a	n/a	SW Abutment
1	71.6/8.7	77	19	4.0	SW Abutment
1	71.3/9.0	57	14	4.0	SW Abutment
2	75.0/5.6	62	19	3.3	NW Abutment
2	74.7/5.9	57	19	3.0	NW Abutment
4	72.6/7.2	110	24	4.5	NE Abutment
4	72.3/7.5	135	43	3.1	NE Abutment
4	71.1/8.7	48	26	1.8	NE Abutment
5	73.4/7.2	67	19	3.5	NE Approach
7	72.0/7.2	115	34	3.4	Pier A
7	71.7/7.5	115	53	2.2	Pier A
9	73.8/5.6	130	53	2.4	Pier D
9	73.5/5.9	96	53	1.8	Pier D
	Average	86	30	3.3	
Clayey Silt					
1	70.1/10.2	29	24	1.2	SW Abutment
1	69.8/10.5	29	10	2.9	SW Abutment
1	68.6/11.7	40	19	2.1	SW Abutment
1	68.3/12.0	38	17	2.2	SW Abutment
1	67.0/13.3	28	24	1.6	SW Abutment
1	65.6/14.8	40	24	1.7	SW Abutment
2	73.4/7.2	24	12	2.0	NW Abutment
2	73.1/7.5	24	14	1.7	NW Abutment
2	71.9/8.7	38	10	2.8	NW Abutment
2	71.6/9.0	29	10	2.9	NW Abutment
2	70.4/10.2	34	14	2.4	NW Abutment
2	70.1/10.5	29	19	1.5	NW Abutment
3	75.6/5.6	29	10	2.9	NW Approach
3	75.3/5.9	43	14	3.1	NW Approach
3	74.0/7.2	19	14	1.4	NW Approach
3	73.7/7.5	19	10	1.9	NW Approach
6	70.8/8.7	59	24	2.5	Pier A
6	70.5/9.0	38	19	2.0	Pier A
6	69.3/10.2	44	29	1.5	Pier A
6	69.0/10.5	37	19	1.9	Pier A
6	67.8/11.7	67	24	2.8	Pier A
6	67.5/12.0	38	24	1.6	Pier A
6	66.2/13.3	48	24	2.0	Pier A
6	65.9/13.6	86*	38	2.3	Pier A
7	70.5/8.7	38	14	2.7	Pier A
7	70.2/9.0	62	50	1.2	Pier A
7	69.0/10.2	48	29	1.7	Pier A
7	68.7/10.5	105*	57	1.8	Pier A
7	65.9/13.3	43	19	2.3	Pier A
7	65.6/13.6	53	24	2.2	Pier A
8	74.2/5.6	81*	21	3.9	SE Abutment
9	72.2/7.2	96*	29	3.3	Pier D
9	71.9/7.5	105*	24	4.4	Pier D
	Average	38	19	2.1	

***** Vane test taken at base of deposit above the sand deposit and may not be representative of the clayey silt.

TABLE 2
RESULTS OF ATTERBERG LIMITS TESTING
CLAYEY SILT TO CLAY DEPOSIT

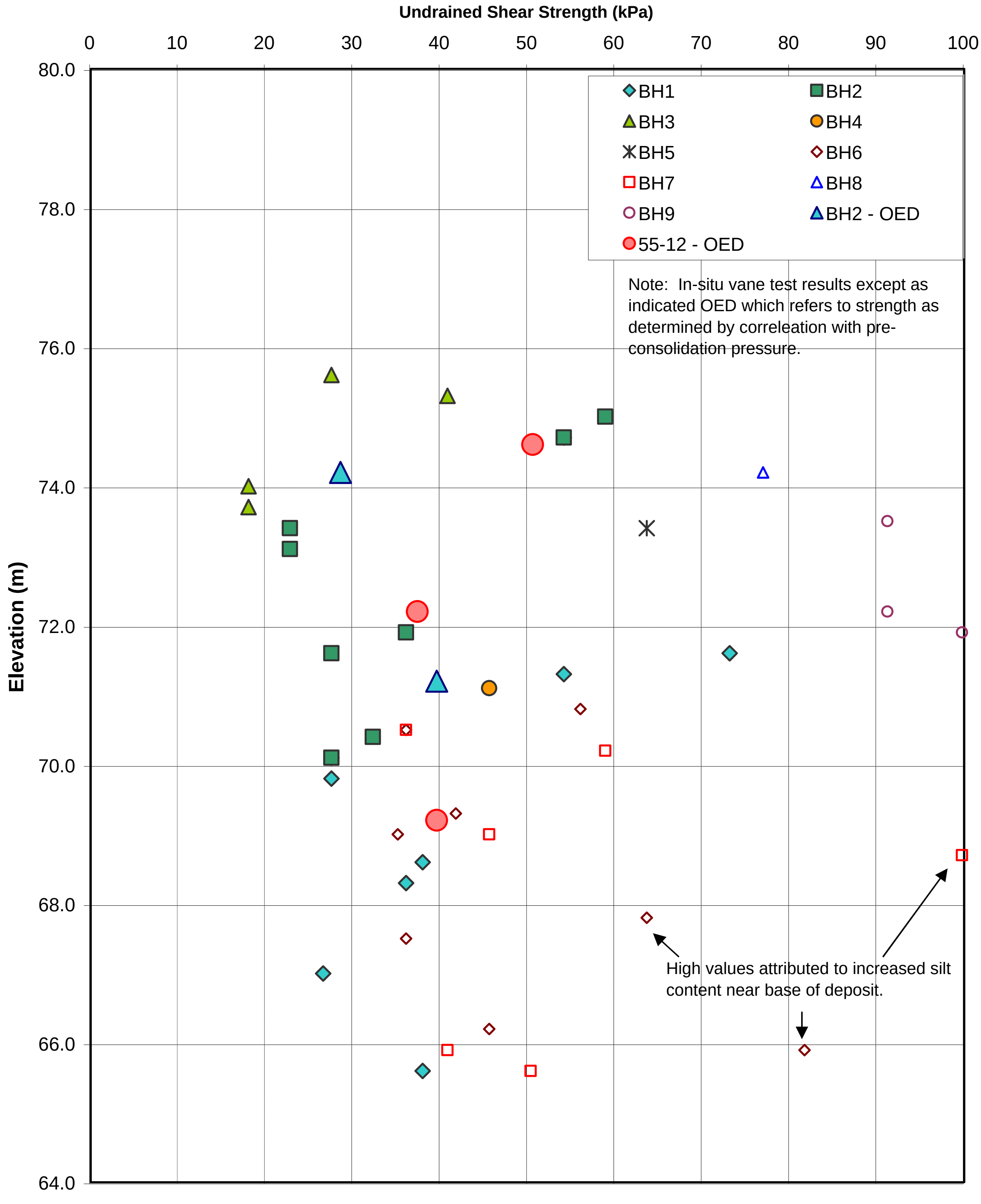
Borehole	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*	*Location*
Silty Clay to Clay						
2	7	76.0 – 75.4	68.3	24.1	44.2	NW Abutment
3	6	77.4 – 76.8	77.2	24.9	52.3	NW Approach
5	6	76.8 – 76.2	48.6	19.5	29.1	NE Approach
8	4	77.5 – 76.9	52.6	19.8	32.8	SE Abutment
55-6	SS6	75.5	47.5	18.2	29.3	Pier B
55-6	SS9	73.2	54.0	18.6	35.4	Pier B
55-12	SS3	73.7	49.5	17.4	32.1	Pier A
55-13	SS2	75.0	48.5	15.8	32.7	Pier D
Clayey Silt						
1	10	71.2 – 70.6	20.9	13.6	7.3	SW Abutment
2	8	74.5 – 73.9	25.3	16.1	9.2	NW Abutment
6	10	70.4 – 69.7	24.4	12.7	11.7	Pier A
7	9	71.6 – 71.0	23.8	14.6	9.2	Pier A
9	7	74.8 – 74.2	22.2	12.8	9.4	Pier D
55-6	SS11	71.0	20.5	13.3	7.2	Pier B
55-6	SS13	67.9	17.0	11.5	5.5	Pier B
55-12	SS5	70.6	25.5	13.7	11.8	Pier A
55-12	SS7	67.6	20.5	7.2	13.3	Pier A

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES

Footing Option	*NF*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread Footings on Silty Clay	NF		Low design bearing capacity. Dewatering required.	Costs may be lower relative to piling option.	Differential settlement between existing footing on piles and widened portion.
Piles driven to Limestone Bedrock		Differential settlement between existing and widened foundations minimized.		Same relative costs as piles on till.	Vibration during driving could have adverse affect on existing structure. Difficulty may be encountered seating piles on sloping bedrock surface.
Caissons on Limestone Bedrock		Drilling operation minimizes vibration in general. Differential settlement minimized.	Groundwater control likely to be required, temporary liners required. Could be significant vibrations during liner advancement/removal and socketting into bedrock.	Cost may be more than driven piles due to liners etc that may be required.	May be difficult to socket liner into limestone bedrock to seal off water; downhole inspection may not be possible. Significant vibrations during caisson socketting may impact existing bridge and pile foundations.

NF: Not considered a feasible founding alternative for this project.

FIGURE 1
Undrained Shear Strength Profile vs Elevation



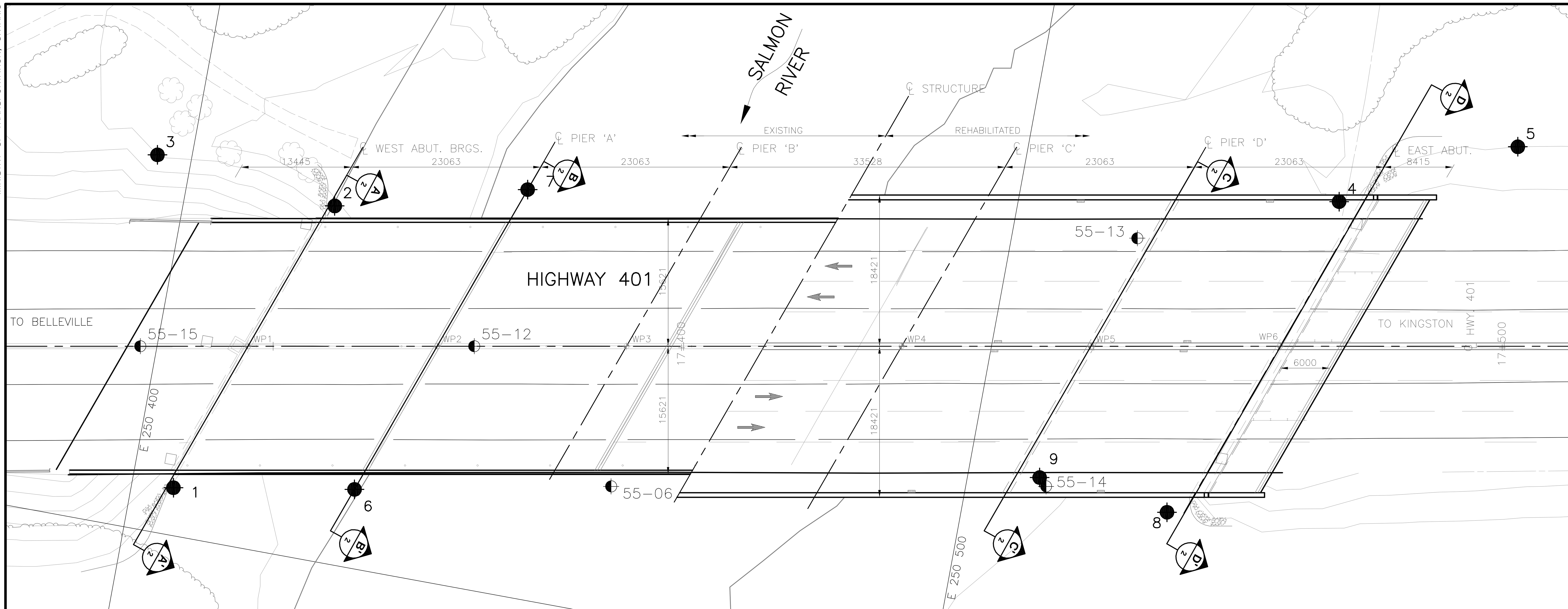
DIST. 8 HWY. 401
CONT No.
WP No.82–98–00

HWY 401 OVER SALMON RIVER
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

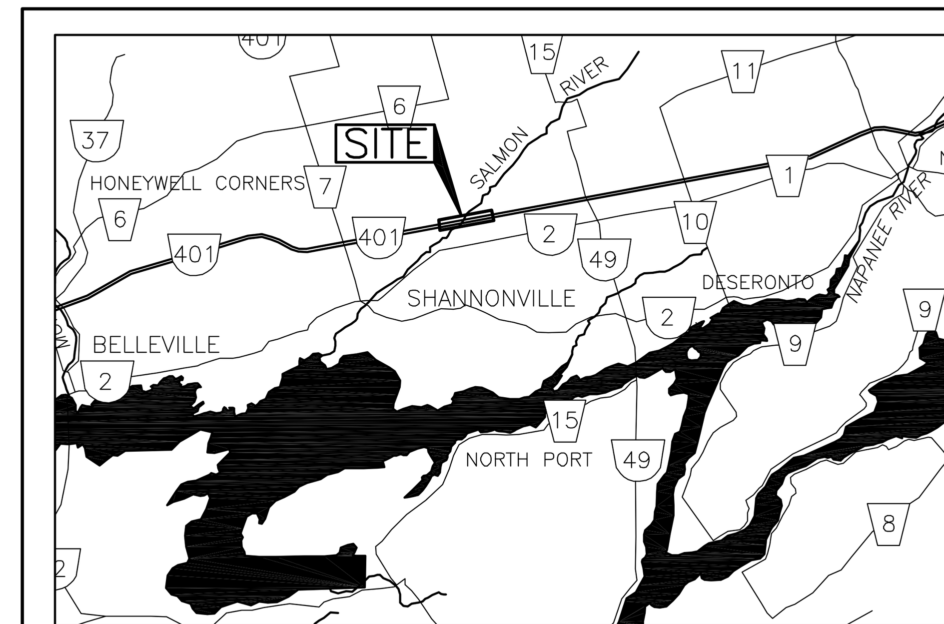
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



PLAN

6 0 6 12
SCALE METRES

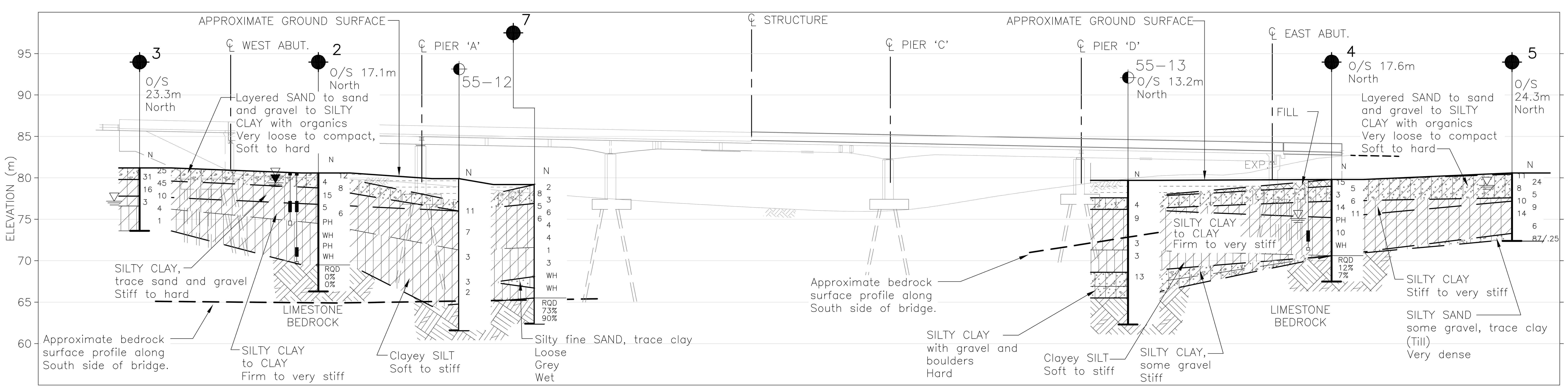


KEY PLAN

LEGEND

- Borehole – Current Investigation
- Borehole – 1955 Investigation by Racey, MacCallum and Associates Ltd.
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on MMM DD, YYYY
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
1	80.3	4898905.8	250405.1
2	80.6	4898943.1	250418.2
3	81.2	4898945.3	250395.8
4	79.8	4898965.8	250538.5
5	80.6	4898976.3	250558.7
6	79.5	4898909.6	250426.8
7	79.2	4898949.3	250441.0
8	79.8	4898924.7	250524.7
9	79.4	4898926.1	250508.7
55–6	80.1	4898915.6	250457.5
55–12	79.9	4898929.4	250438.0
55–13	79.7	4898956.9	250515.1
55–14	79.5	4898925.2	250509.6
55–15	80.3	4898922.0	250398.0



NORTH SIDE PROFILE

6 0 6 12
SCALE METRES

NOTE: Soil Stratigraphy shown based on the North side of bridge structure.

NOTES

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

REFERENCE

Base plan provided in digitial format by Lea Associates Ltd. drawing titled "HWY 401 OVER SALMON RIVER BRIDGE REHABILITATION", drawing file no. RI, received June II, 2003.

NO.	DATE	BY	REVISION

Geocres No. 31C–168

HWY. 401		PROJECT NO. 03–1111–009	DIST. 8
SUBM'D.	CHKD. SEP	DATE: JUNE 2003	SITE: 11–207
DRAWN: JDR	CHKD. ASP	APPD. FJH	DWG. 1

DIST. 8 HWY. 401
CONT No.
WP No.82-98-00

HWY 401 OVER SALMON RIVER

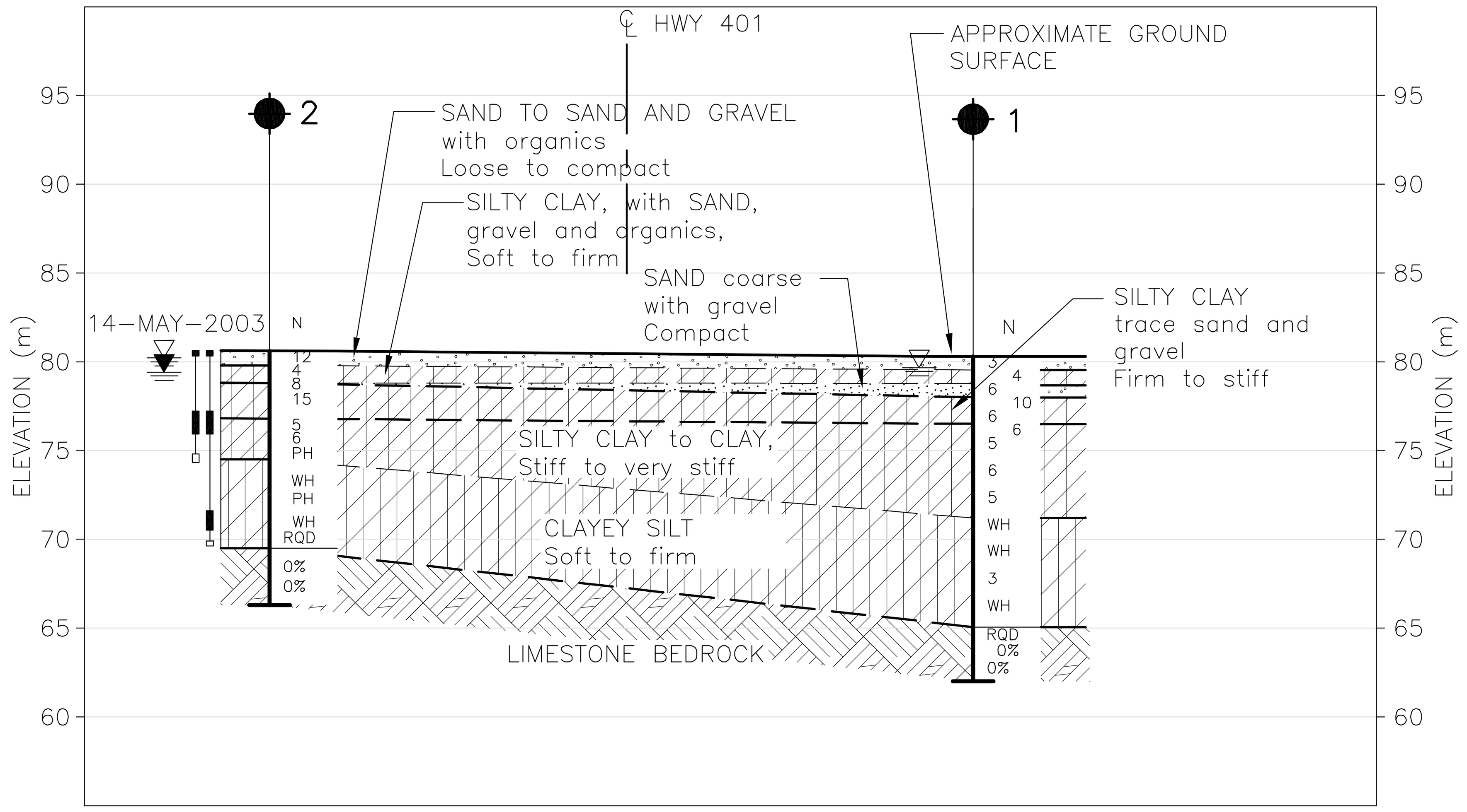
BRIDGE REHABILITATION

BOREHOLE SOIL STRATA

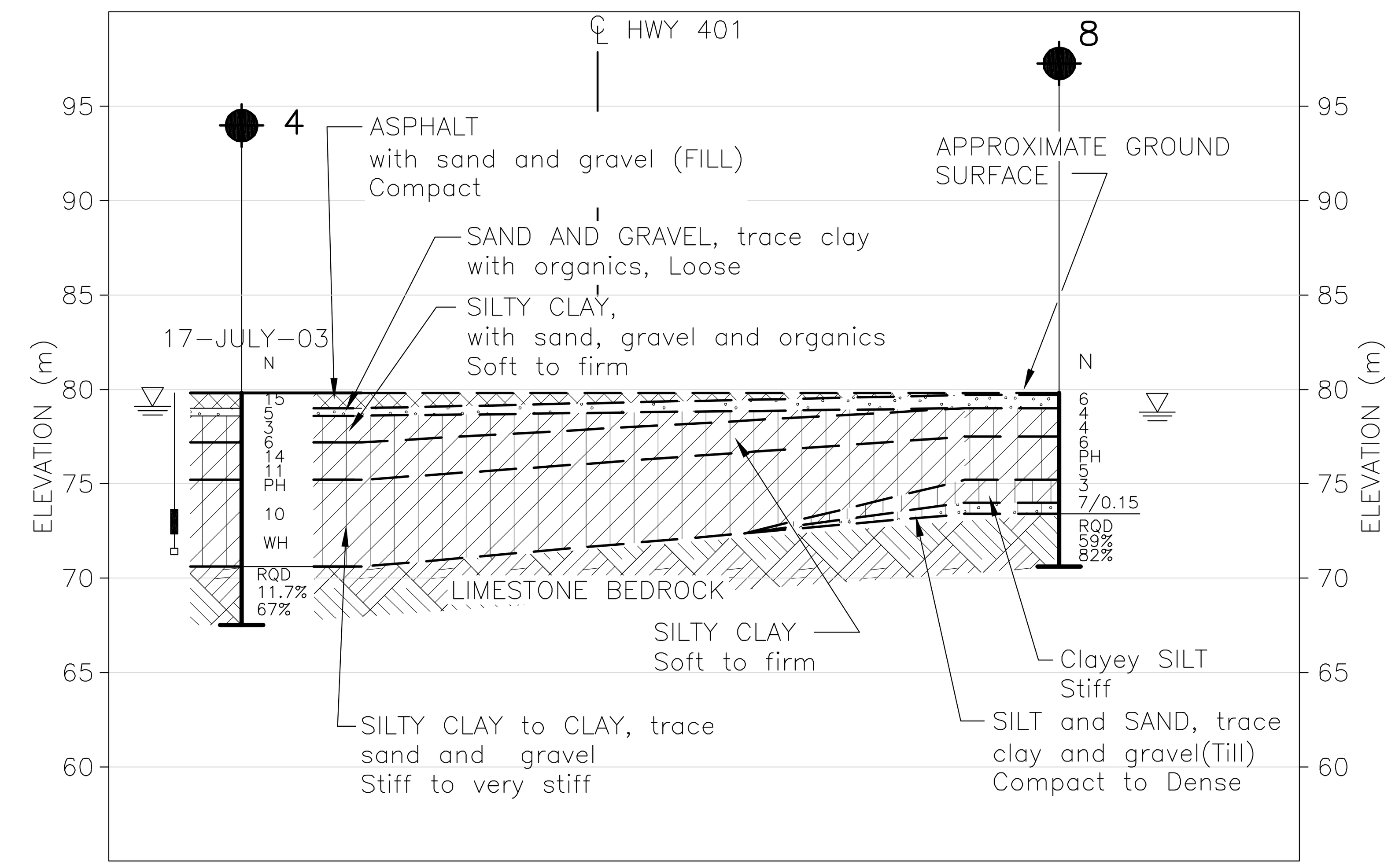
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

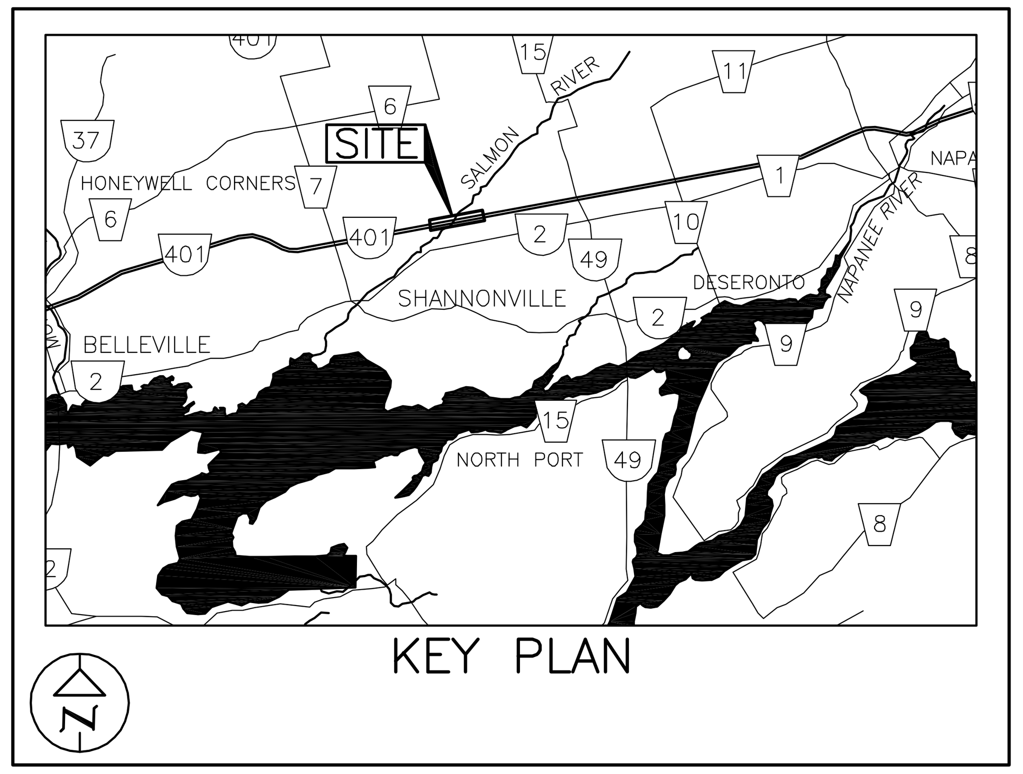
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

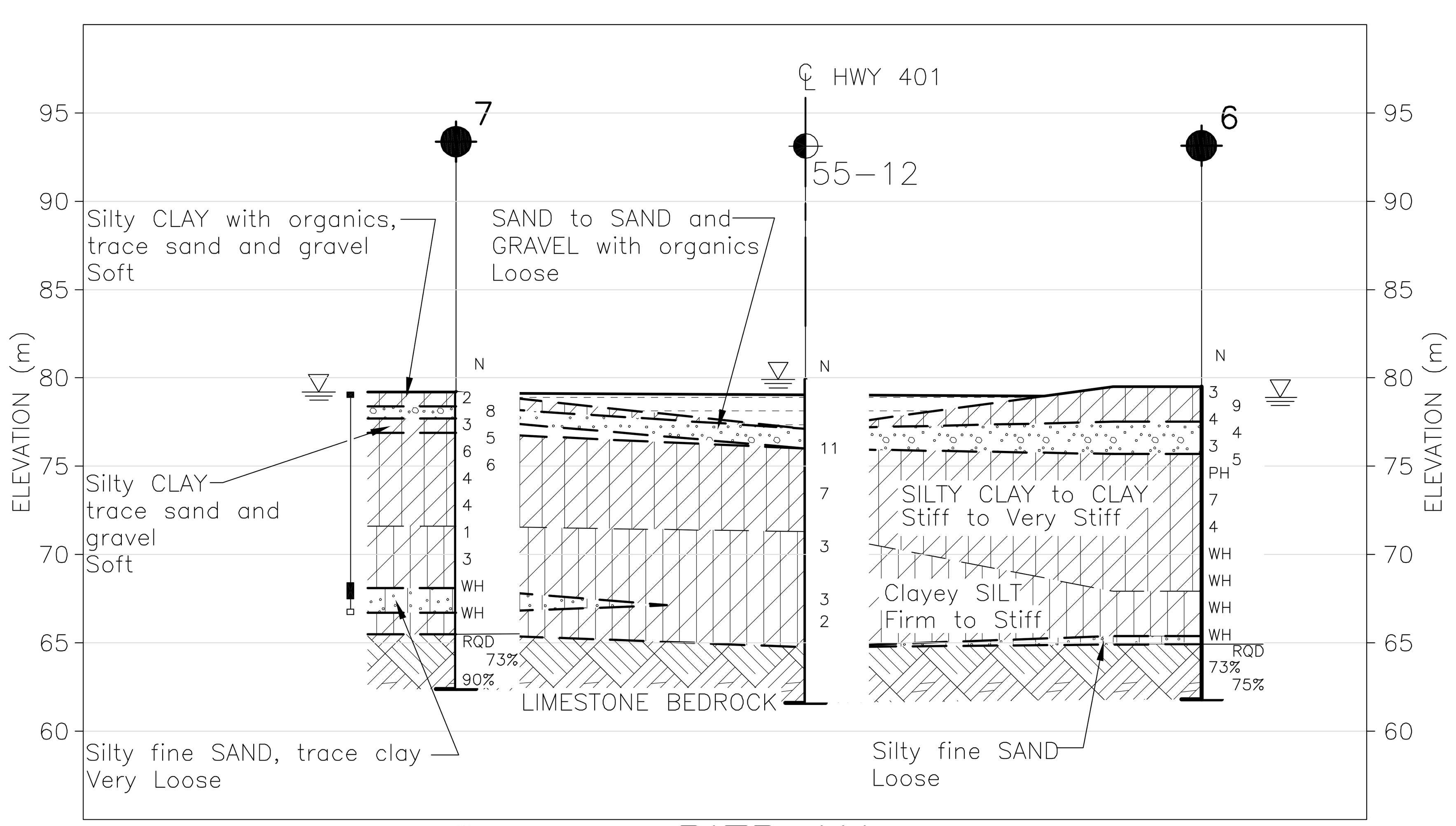


A-A' 1 WEST ABUTMENT
SCALE 6 0 6 12 METRES

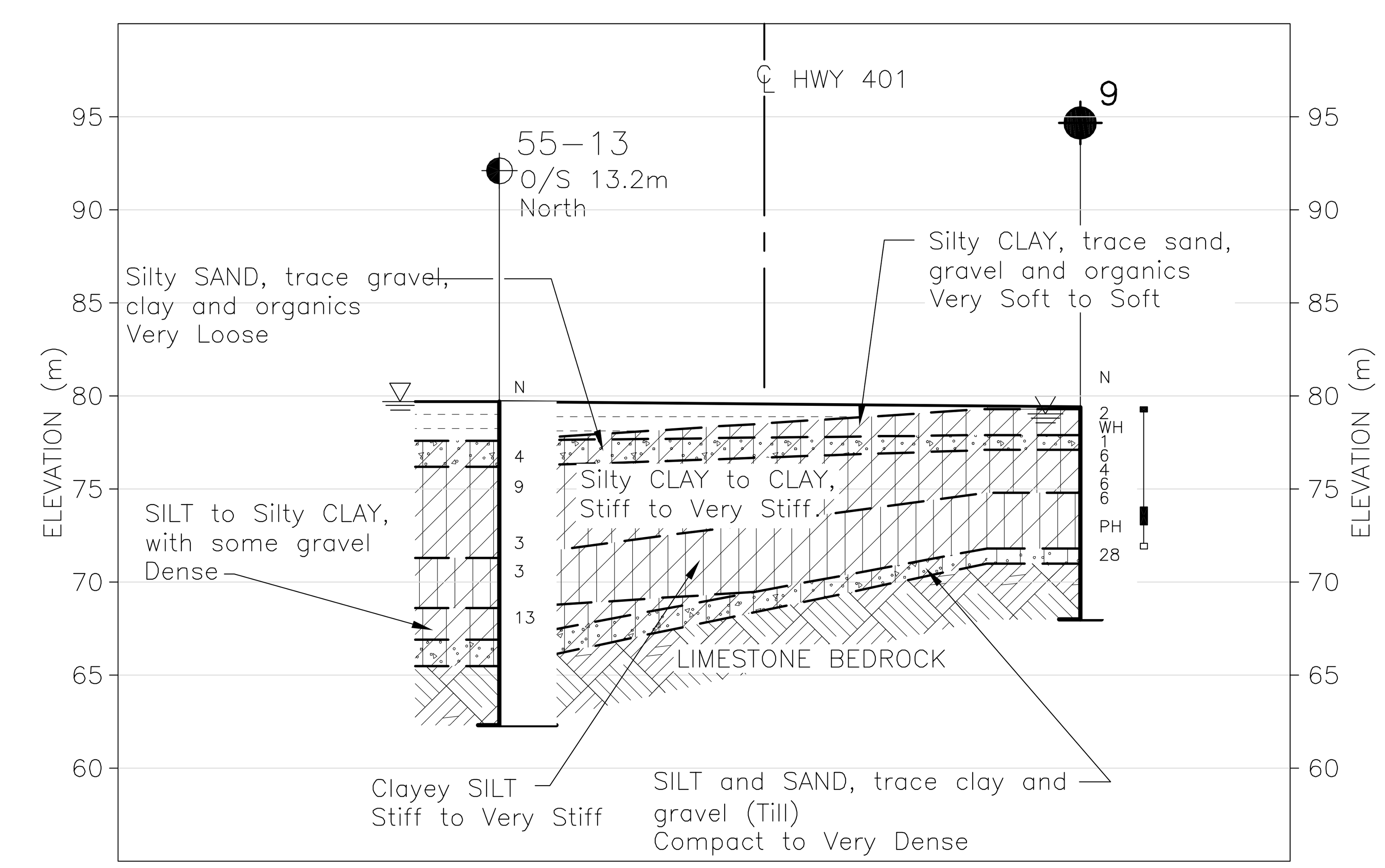


D-D' 1 EAST ABUTMENT
SCALE 6 0 6 12 METRES





B-B' 1 PIER 'A'
SCALE 6 0 6 12 METRES



C-C' 1 PIER 'D'
SCALE 6 0 6 12 METRES

LEGEND

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- Borehole – 1955 Investigation by Racey, MacCallum and Associates Ltd.
- Seal
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- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
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No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
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55-15	80.3	4898922.0	250398.0

NOTES

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

NO.	DATE	BY	REVISION

Geocres No.31C-168

HWY. 401		PROJECT NO. 03-1111-009	DIST. 8
SUBM'D.	CHKD. SEP	DATE: JUNE 2003	SITE: 11-207
DRAWN: JFC	CHKD. ASP	APPD. FJH	DWG. 2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_1	liquid limit
w_p	plastic limit
l_p	plasticity index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p)/I_p$
I_C	consistency index = $(w_1 - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

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

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LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

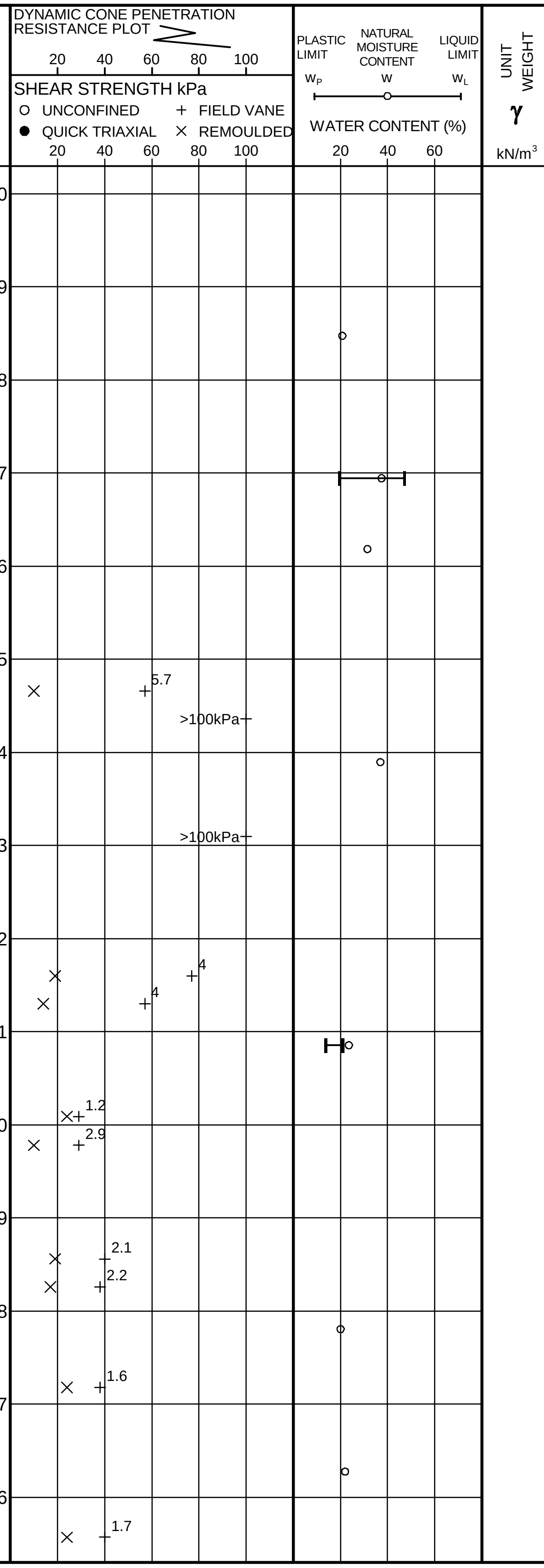
B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
$\parallel$ -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

RECORD OF BOREHOLE No 1

METRIC

PROJECT 03-1111-009			
W.P. 82-98-00	LOCATION N 4898905.8 ;E 250405.1		ORIGINATED BY SB
DIST 8 HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY SEP
DATUM Geodetic	DATE May 6, 2003		CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
80.3	GROUND SURFACE						
0.0	Topsoil					80	
0.2	SAND, trace clay and gravel Loose Grey Moist	1	SS	3	∇		
79.5 0.8	Silty CLAY, trace sand, gravel, organics, roots and pieces of decayed wood Soft Grey Moist	2	SS	4		79	
78.8 1.5	Coarse SAND with gravel Compact Grey Wet	3	SS	6			
78.0 2.3	Silty CLAY, trace to some sand and gravel, pockets of brown sand Stiff to firm Grey Moist	4	SS	10		78	
		5	SS	6		77	3 9 29 59
76.5 3.8	Silty CLAY to Clay Stiff to very stiff Grey Moist	6	SS	6		76	
		7	SS	5		75	
		8	SS	6		74	
						73	
		9	SS	5		72	
71.2 9.1	Clayey SILT Firm Grey Moist	10	SS	WH		71	
						70	
		11	SS	WH		69	
	Becoming siltier with pockets of silty fine sand below 12.2m depth	12	SS	3		68	
						67	
		13	SS	WH		66	



Continued Next Page

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

Golder Associates

PROJECT 03-1111-009	RECORD OF BOREHOLE No 1	2 OF 2	METRIC
W.P. 82-98-00	LOCATION N 4898905.8 ;E 250405.1		ORIGINATED BY SB
DIST 8 HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY SEP
DATUM Geodetic	DATE May 6, 2003		CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
65.1											
15.2	Grey slightly weathered to fresh, LIMESTONE to SHALEY LIMESTONE (Trenton and Black River Group) Bedrock cored from 15.2m to 18.3m depth For Bedrock coring details see Record of Drillhole 1						65 64 63				
62.0 18.3	End of Borehole Notes: 1. Spoon refusal at 15.2m depth (Elev. 65.1m) 2. Water level at 0.6m depth (Elev. 79.7m) upon completion of drilling						62				

MISS_MTO 031111009.GPJ ON_MOT.GDT 12/3/04

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

PROJECT: 03-1111-009

RECORD OF DRILLHOLE: 1

SHEET 1 OF 1

LOCATION: N 4898905.8 ;E 250405.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: MAY 6, 2003

DRILL RIG: D90 BOMBARDIER

DRILLING CONTRACTOR: WALKER DRILLING LTD

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
		.		65.06												
16, 17, 18	Power Auger; 108mm Hollow Stem Auger	Grey slightly weathered to fresh, strong massive to thinly laminated, fine to medium grained LIMESTONE to SHALEY LIMESTONE, some dark grey shaley partings which act as planes of weakness Core is very broken, recovered as strong rubble and solid core pieces less than 10cm length No joint orientations other than bedding observed. Bedding joints are closely spaced. Bedding surfaces are smooth and planar to uneven		15.24	1	0	100						BC BC BC BC BC BC			
					2	0	100						BC BC BC			
				62.00 18.30												
19, 20, 21, 22, 23, 24, 25		Grey slightly weathered to fresh, strong massive to thinly laminated, fine to medium grained LIMESTONE to SHALEY LIMESTONE, some dark grey shaley partings which act as planes of weakness Core is very broken, recovered as strong rubble and solid core pieces less than 10cm length No joint orientations other than bedding observed. Bedding joints are closely spaced. Bedding surfaces are smooth and planar to uneven														

MISS_ROCK 03111009-ROCK.GPJ GLDR_CAN.GDT 12/3/04 JFC/JDR

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: ASP

RECORD OF BOREHOLE No 2

METRIC

PROJECT 03-1111-009
W.P. 82-98-00
LOCATION N 4898943.1 ;E 250418.2
ORIGINATED BY SB
DIST 8 HWY 401
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
COMPILED BY SEP
DATUM Geodetic
DATE May 7, 2003
CHECKED BY ASP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
80.6	GROUND SURFACE
80.0 / 0.1	Topsoil
	SAND and GRAVEL, nodules of silty clay Compact Grey Moist
79.8 / 0.8	Silty CLAY with sand and gravel, trace organics Soft to firm Grey Moist
78.8 / 1.8	Silty CLAY, trace sand and gravel Stiff Brown mottled grey to grey Moist
76.8 / 3.8	Silty CLAY to CLAY Stiff to very stiff Grey Moist
74.5 / 6.1	Clayey SILT Soft to firm Grey Moist Becoming siltier below 7.6m depth
69.5 / 11.1	Grey slightly weathered to fresh, LIMESTONE to SHALEY LIMESTONE (Trenton and Black River Group) Bedrock cored from 11.3m to 14.3m depth For Bedrock coring details see Record of Drillhole 2
66.3 / 14.3	

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	12
2	SS	4
3	SS	8
4	SS	15
5	TO	PH
6	SS	5
7	SS	6
8	TO	PH
9	SS	WH
10	TO	PH
11	SS	WH

ELEVATION SCALE: 80, 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67

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

Field vane sensitivities: 3.3, 3 (elev. approx. 75); 2, 1.7 (elev. approx. 73.5); 3.8, 2.9 (elev. approx. 72); 2.4, 1.5 (elev. approx. 70.5)

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³: 18.1 (sample 8)

REMARKS & GRAIN SIZE DISTRIBUTION (%)

	GR	SA	SI	CL
Sample 9	0	1	6	36

Continued Next Page

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

Golder Associates

PROJECT 03-1111-009	RECORD OF BOREHOLE No 2	2 OF 2	METRIC
W.P. 82-98-00	LOCATION N 4898943.1 ;E 250418.2		ORIGINATED BY SB
DIST 8 HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY SEP
DATUM Geodetic	DATE May 7, 2003		CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	End of Borehole Notes: 1. No recovery sample 2. Water level in piezometer after installation as follows; -Shallow - 0.4m above ground surface (Elev. 81.0m) -Deep - 1.2m below ground surface (Elev. 79.4m) 3. Water level in piezometer on July 14, 2003 as follows; -Shallow - 0.1m below ground surface (Elev. 80.5m) -Deep - 1.1m below ground surface (Elev. 79.5m)										

MISS_MTO 03111009.GPJ ON_MOT.GDT 12/3/04

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

PROJECT: 03-1111-009

RECORD OF DRILLHOLE: 2

SHEET 1 OF 1

LOCATION: N 4898943.1 ;E 250418.2

DRILLING DATE: MAY 6, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D90 BOMBARDIER

DRILLING CONTRACTOR: WALKER DRILLING LTD

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	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		.	69.30				
	Power Auger, 108mm Hollow Stem Auger	Grey slightly weathered to fresh, strong massive to thinly laminated, fine to medium grained LIMESTONE to SHALEY LIMESTONE, some dark grey shaley partings which act as planes of weakness Core is very broken, recovered as strong rubble and solid core pieces less than 10cm length No joint orientations other than bedding observed. Bedding joints are closely spaced. Bedding surfaces are smooth and planar to uneven	11.30		BC		
					BC		
12				1	BC		
					BC		
					BC		
					B, SM, PL-UE		
13					BC	•	
				2	BC		
					BC		
14					BC		
			66.30 14.30				
		End of Drillhole					
15							
16							
17							
18							
19							
20							
21							

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: ASP

MISS_ROCK 03111009-ROCK.GPJ GLDR_CAN.GDT 12/3/04 JFC/JDR

PROJECT 03-1111-009 — **RECORD OF BOREHOLE No 3** — 1 OF 1 — **METRIC**

W.P. 82-98-00	LOCATION N 4898945.3 ;E 250395.8		ORIGINATED BY SB
DIST 8	HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY SEP
DATUM Geodetic		DATE May 8, 2003	CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES
81.2	GROUND SURFACE			
0.0	Topsoil			
0.2	Silty CLAY, trace sand and gravel with organics, roots above 0.8m depth Stiff to hard Brown Moist	1	SS	25
		2	SS	31
79.8 / 1.4	Silty CLAY, trace sand and gravel Very stiff to hard Brown Moist	3	SS	45
		4	SS	16
77.8 / 3.4	Silty CLAY to CLAY Firm to stiff Grey Moist	5	SS	10
		6	SS	3
76.6 / 4.6	Clayey SILT Soft to firm Grey Moist	7	SS	4
		8	SS	1
73.6 / 7.6	End of Borehole Auger and spoon refusal Probable bedrock surface Notes: 1. Water level at 4.0m depth (Elev. 77.2m) upon completion of drilling			

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

Elevation scale: 81, 80, 79, 78, 77, 76, 75, 74

Field vane sensitivity labels: 2.9, 3.1, 1.4, 1.9

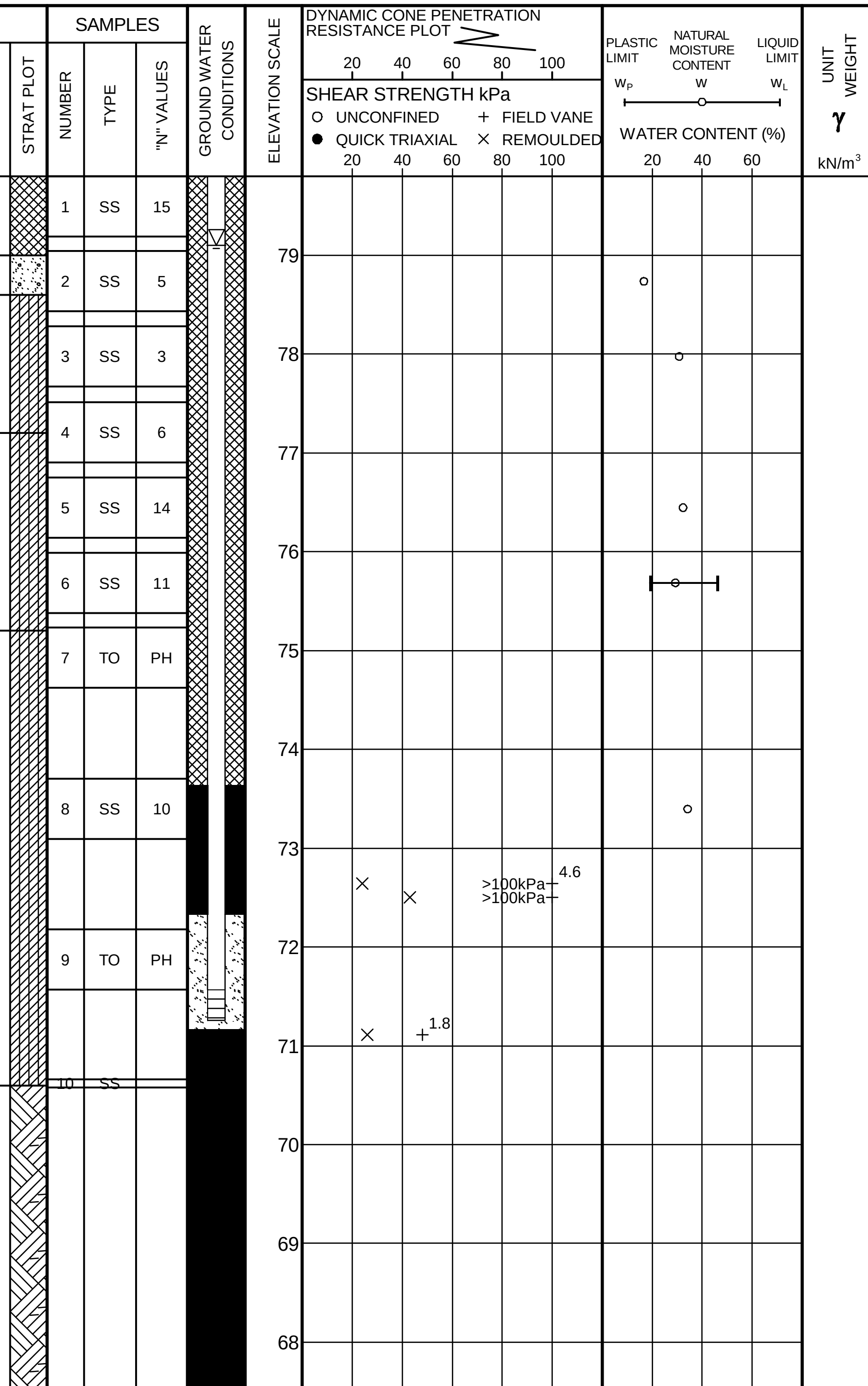
+3, ×3: Numbers refer to Sensitivity — O 3% STRAIN AT FAILURE

PROJECT 03-1111-009

RECORD OF BOREHOLE No 4 1 OF 1 **METRIC**

W.P. 82-98-00	LOCATION N 4898965.8 ;E 250538.5	ORIGINATED BY SB
DIST 8 HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY SEP
DATUM Geodetic	DATE May 8, 2003	CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.8	GROUND SURFACE				
0.0	Asphalt with roots and organics (FILL) Compact Black	1	SS	15	
79.0	SAND and GRAVEL with roots Loose Brown and grey	2	SS	5	
78.6 / 1.2	Silty CLAY, trace sand and gravel, trace organics and decayed wood Soft to firm Grey Moist	3	SS	3	
77.2 / 2.6	Silty CLAY, trace sand Stiff to very stiff Grey Moist	4	SS	6	
		5	SS	14	
		6	SS	11	0 1 40 59
75.2 / 4.6	Silty CLAY to CLAY Very stiff to firm Grey Moist	7	TO	PH	
	Slight layering at 6.1m depth	8	SS	10	
		9	TO	PH	
		10	SS		
70.6 / 9.2	Grey slightly weathered to fresh, LIMESTONE to SHALEY LIMESTONE (Trenton and Black River Group) Bedrock cored from 9.3m to 12.3m depth For Bedrock coring details see Record of Drillhole 4				
67.5 / 12.3	End of Borehole Note: Water level at 0.7m depth (Elev. 79.1m) on July 17, 2003				



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

PROJECT: 03-1111-009

RECORD OF DRILLHOLE: 4

SHEET 1 OF 1

LOCATION: N 4898965.8 ;E 250538.5

DRILLING DATE: MAY 9, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D90 BOMBARDIER

DRILLING CONTRACTOR: WALKER DRILLING LTD

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	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		.	70.50 / 9.30		
	Power Auger; 108mm Hollow Stem Auger	Grey slightly weathered to fresh, strong, thinly laminated, fine to medium grained LIMESTONE with shaley partings acting as planes of weakness Run 1 of core is very broken, recovered as strong rubble and solid core pieces less than 10cm length No joint orientations other than bedding observed.			BC
					BC
					BC
10				1	BC
					BC
					BC
					BC
11					B, SM, PL-UE
					B, SM, PL-UE
				2	BC
12					B, SM, PL-UE
					'0'
			67.50 / 12.30		
		End of Drillhole			

Other columns: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH – COLOUR / % RETURN; 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); DIP w.r.t. CORE AXIS (0 30 60 90); 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: 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: ASP

Golder Associates

RECORD OF BOREHOLE No 5

METRIC

PROJECT 03-1111-009			
W.P. 82-98-00	LOCATION N 4898976.3 ;E 250558.7		ORIGINATED BY SB
DIST 8	HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY SEP
DATUM Geodetic	DATE May 9, 2003		CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
80.6	GROUND SURFACE												
0.0	Topsoil												
0.2	Silty CLAY, trace sand and gravel, organics, wood, rootlets Stiff Brown to brown-black to grey Moist	1	SS	11		80							
		2	SS	24	∇	79							
		3	SS	8				o					
		4	SS	5		78							
77.6	SAND with wood Loose Brown Wet	5	SS	10									
77.2 3.4	Silty CLAY to CLAY Stiff to very stiff Grey Moist					77							
		6	SS	9		76		├o┤					
		7	SS	14		75							
	Becoming siltier below 6.1 m	8	SS	6		74		o					
							× ; >100kPa + 3.5						
73.3 7.3	Silty SAND with gravel, trace clay (Till) Very dense Grey Moist	9	SS	87/.25		73		o		22	48	22	8
72.4 8.2	End of Borehole Auger refusal Probable bedrock surface Notes: 1. Water level at 1.5m depth (Elev. 79.1m) upon completion of drilling												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates

RECORD OF BOREHOLE No 6 **METRIC**

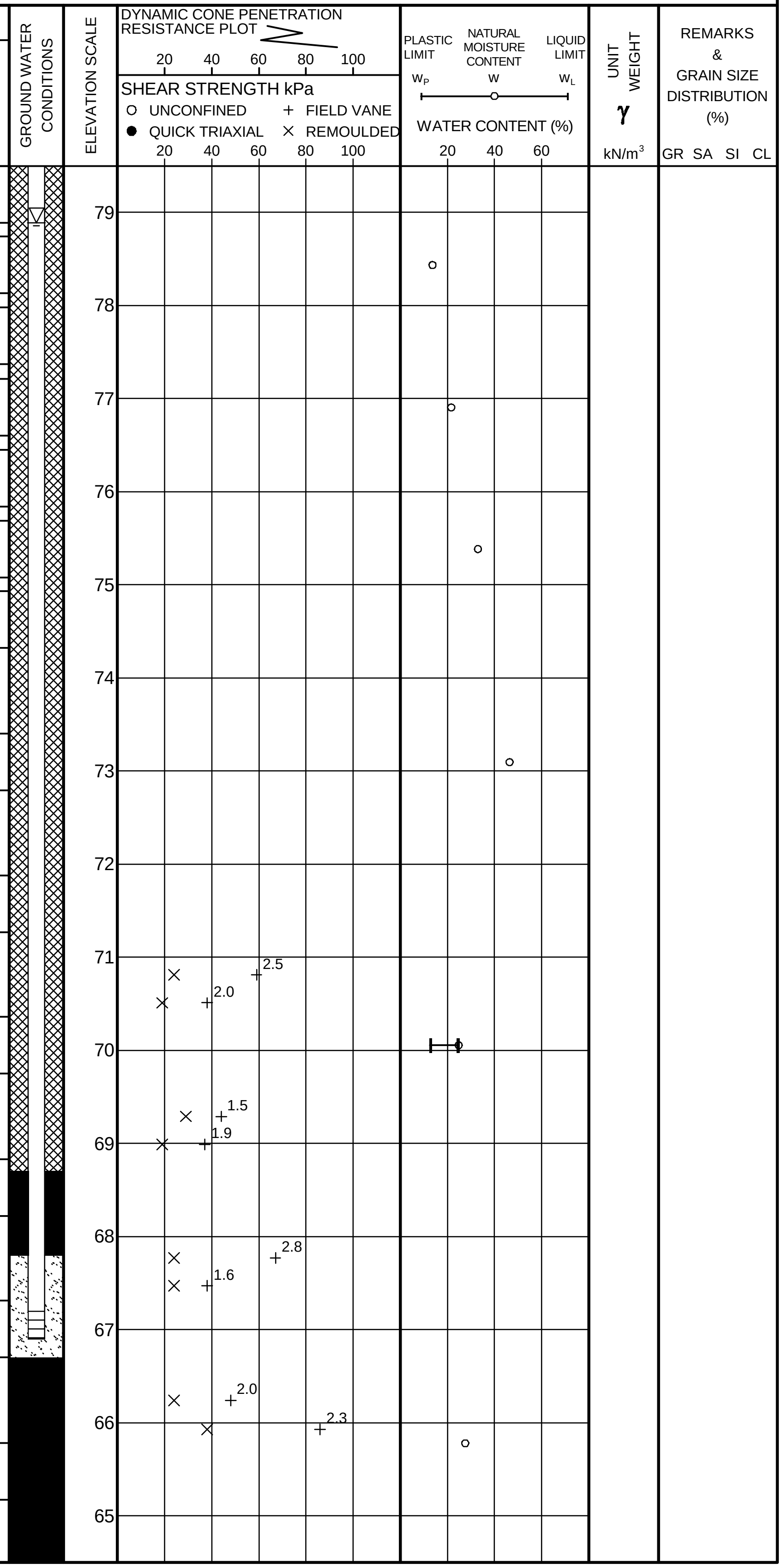
PROJECT 03-1111-009			
W.P. 82-98-00	LOCATION N 4898909.6 ;E 250426.8	ORIGINATED BY SB	
DIST 8 HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY JDR	
DATUM Geodetic	DATE July 14, 2003	CHECKED BY SP	

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
79.5	GROUND SURFACE
0.0	Silty CLAY, trace sand and gravel, roots, pieces of wood Soft to stiff Black/grey to grey Moist
77.5 / 2.0	SAND and GRAVEL, trace clay and silt, occ. roots and wood Loose Grey Wet
75.7 / 3.8	Silty CLAY to CLAY Stiff to very stiff Grey Moist
71.9 / 7.6	Clayey SILT Firm to stiff Moist Grey Varved above 8.2m depth
65.4 / 14.1	Silty fine SAND Loose Grey Wet
64.9 / 14.6	

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	3
2	SS	9
3	SS	4
4	SS	4
5	SS	3
6	SS	5
7	TO	PH
8	SS	7
9	SS	4
10	SS	WH
11	SS	WH
12	SS	WH
13A / 13 / 13B	SS	WH



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

PROJECT 03-1111-009	RECORD OF BOREHOLE No 6	2 OF 2	METRIC
W.P. 82-98-00	LOCATION N 4898909.6 ;E 250426.8		ORIGINATED BY SB
DIST 8 HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY JDR
DATUM Geodetic	DATE July 14, 2003		CHECKED BY SP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Grey, fresh, LIMESTONE (Trenton and Black River Group) with shaley laminations Bedrock cored from 14.6m to 17.7m depth For Bedrock coring details see Record of Drillhole 6						64 63 62				
61.8 17.7	End of Borehole Note: 1. Water level in piezometer at 0.7m depth (Elev. 78.8m) on July 16, 2003										

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

RECORD OF DRILLHOLE: 6

PROJECT: 03-1111-009
SHEET 1 OF 1

LOCATION: N 4898909.6 ;E 250426.8
DRILLING DATE: July 14, 2003
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG: CMB 55 BOMBARDIER
DRILLING CONTRACTOR: MARATHON DRILLING

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.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		.		64.90			
15–17.7	Power Auger, 108mm Hollow Stem Auger	Grey, fresh, strong, thin to finely laminated, fine to medium grained LIMESTONE with dark grey shaley lamina (styolitic). Fractures are closely spaced. One 5mm thick shale bed at 15.1m depth, partially washed out during drilling.		14.60	1	B, PL x3 J, SM x1 B, PL x4 J, R J, SM	
					2	B, R, W B, PL, SM J, PL, SM B, PL, SM	
		End of Drillhole		61.80 17.70			

Depth scale ticks: 15, 16, 17, 18, 19, 20, 21, 22, 23, 24

Other column headings: 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); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB
CHECKED: SP

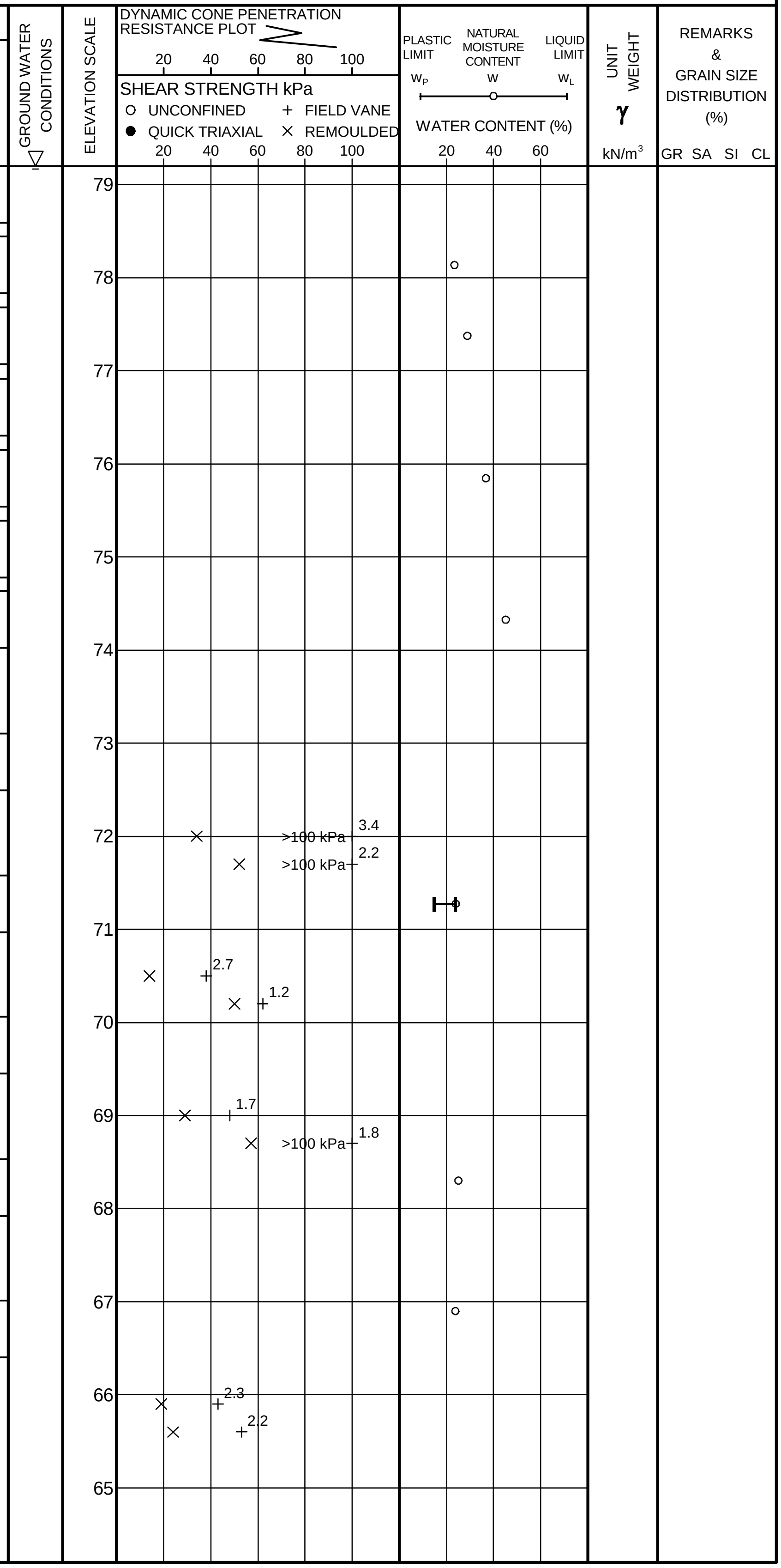
Golder Associates

RECORD OF BOREHOLE No 7

METRIC

PROJECT 03-1111-009			
W.P. 82-98-00	LOCATION N 4898949.3 ;E 250441.0	ORIGINATED BY SB	
DIST 8	HWY 401	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY JDR
DATUM Geodetic	DATE July 15, 2003	CHECKED BY SP	

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
79.2	GROUND SURFACE			
0.0	Silty CLAY, with roots and wood, trace sand and gravel; Soft; Blackish-brown; Moist	1	SS	2
78.4 / 0.8	SAND, medium to course, trace gravel; Loose; Brown; Wet	2	SS	8
77.7 / 1.5	Silty CLAY, trace sand and gravel; Soft; Grey; Moist	3	SS	3
76.9 / 2.3	Silty CLAY to CLAY; Stiff to very stiff; Grey; Moist	4	SS	5
		5	SS	6
		6	SS	6
		7	SS	4
		8	SS	4
71.6 / 7.6	Clayey SILT; Firm to stiff; Grey; Moist; Pockets of dark grey clay, pockets of silt in sample No. 10	9	SS	1
		10	SS	3
		11A / 11B	SS	WH
68.1 / 11.1	Silty fine SAND, trace clay; Very loose; Grey; Wet			
		12A / 12B	SS	WH
66.7 / 12.5	Clayey SILT, trace sand; Firm to stiff; Grey; Moist			
65.5 / 13.7				



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

Golder Associates

PROJECT 03-1111-009 | RECORD OF BOREHOLE No 7 | | METRIC

W.P. 82-98-00 | LOCATION N 4898949.3 ;E 250441.0 | ORIGINATED BY SB

DIST 8 HWY 401 | BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger | COMPILED BY JDR

DATUM Geodetic | DATE July 15, 2003 | CHECKED BY SP

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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Grey, fresh, LIMESTONE (Trenton and Black River Group) with shaley laminations Bedrock cored from 13.7m to 16.8m depth For Bedrock coring details see Record of Drillhole 7						64 63				
62.4 16.8	End of Borehole Note: Water level in open borehole at ground surface (Elev. 79.2m) upon completion of drilling.										

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

RECORD OF DRILLHOLE: 7

PROJECT: 03-1111-009

SHEET 1 OF 1

LOCATION: N 4898949.3 ;E 250441.0

DRILLING DATE: July 15, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CMB 55 BOMBARDIER

DRILLING CONTRACTOR: MARATHON DRILLING

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	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		.	65.50			
14	Power Auger; 108mm Hollow Stem Auger	Grey, fresh, strong, thin to finely laminated, fine to medium grained LIMESTONE with dark grey shaley lamina (styolitic). Fractures are closely spaced. One 10mm thick shale bed at 14.6m depth, very weak, partially washed out during drilling.	13.70	1	B, PL, SM/R B, PL-SM/R B, J, PL-SM B, PL-SM	
15–16				2	B, PL-SM B, PL-SM B, PL-SM B, PL-SM B, PL-SM	
		End of Drillhole	62.40 16.80			

Other column headings: SYMBOLIC LOG; 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); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6).

Depth scale: 14, 15, 16, 17, 18, 19, 20, 21, 22, 23

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: SP

RECORD OF BOREHOLE No 8

METRIC

PROJECT 03-1111-009

W.P. 82-98-00 | LOCATION N 4898924.7 ;E 250524.7 | ORIGINATED BY SB

DIST 8 HWY 401 | BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger | COMPILED BY JDR

DATUM Geodetic | DATE July 16, 2003 | CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.8	GROUND SURFACE					
79.8 / 0.0	Topsoil					
0.1	SAND and GRAVEL, trace clay, Loose, Grey, Moist	1	SS	6		
79.0 / 0.8	Silty CLAY, trace sand and gravel, roots and wood pieces, Soft to firm, Grey, Moist	2	SS	4	∇	
		3	SS	4		
77.5 / 2.3	Silty CLAY to CLAY, trace sand and gravel, Stiff to very stiff, Grey, Moist	4	SS	6		
		5	TO	PH		
	Layers of grey silt below 3.8m	6	SS	5		
75.2 / 4.6	Clayey SILT, Stiff, Grey, Moist	7	SS	3		
74.0 / 5.8	SILT and SAND, trace clay, trace gravel (Till), Compact to dense, Grey, Wet	8	SS	7/0.15		10 45 33 12
73.4 / 6.4	Grey, fresh, LIMESTONE (Trenton and Black River Group) with shaley laminations. Bedrock cored from 6.4m to 9.2m depth. For Bedrock coring details see Record of Drillhole 03-8					
70.6 / 9.2	End of Borehole. Note: Water level in open borehole at 1.0m depth (Elev. 78.8m) upon completion of drilling.					

Plot columns: 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 (%) (20 40 60); UNIT WEIGHT γ kN/m³; ELEVATION SCALE 79 78 77 76 75 74 73 72 71.

Plotted values: at approx. elev. 74.2: × (remoulded) near 20 kPa, + (field vane) near 80 kPa with sensitivity 3.9.

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

RECORD OF DRILLHOLE: 8

PROJECT: 03-1111-009
LOCATION: N 4898924.7 ;E 250524.7
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 16, 2003
DRILL RIG: CMB 55 BOMBARDIER
DRILLING CONTRACTOR: MARATHON DRILLING

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	ELEV. DEPTH (m)	RUN No.	FLUSH	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		.	73.50				
7	Power Auger, 108mm Hollow Stem Auger	Grey, fresh, strong, thin to finely laminated, fine to medium grained LIMESTONE with dark grey shaley lamina (styolitic). Fractures are closely spaced. Brocken core from 8.5m to 9.1m depth as result of joints parallel to core axis.	6.30	1		B, SM-R, PL B, SM-R, PL B, SM-R, PL B, SM-R, PL	
8–9				2	<<	BC, R, UE B, R, PL BC, R, UE BC, R, UE	
		End of Drillhole	70.60 9.20				

Depth scale: 7, 8, 9, 10, 11, 12, 13, 14, 15, 16

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB
CHECKED: SP

Golder Associates

RECORD OF BOREHOLE No 9

METRIC

PROJECT 03-1111-009
W.P. 82-98-00
LOCATION N 4898926.1 ;E 250508.7
ORIGINATED BY SB
DIST 8 HWY 401
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
COMPILED BY JDR
DATUM Geodetic
DATE July 18, 2003
CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.4	GROUND SURFACE				
0.0	Topsoil				
0.1	Silty CLAY, trace sand, gravel and organics, wood Very soft to soft Grey Moist	1 2	SS SS	2 WH	
77.9 / 1.5	Silty SAND, fine to medium, trace gravel, clay and organics Very loose Brown and black Moist	3	SS	1	
77.1 / 2.3	Silty CLAY to CLAY Stiff to very stiff Grey Moist	4 5 6	SS SS SS	6 4 6	
74.8 / 4.6	Clayey SILT, with layers of dark grey clay Stiff to very stiff Grey Moist	7 8	SS TO	6 PH	
71.8 / 7.6	SILT and SAND with gravel, trace clay, trace gravel (Till) Compact Grey Wet	9	SS	28	12 34 43 11
71.0 / 8.4	Grey, fresh, LIMESTONE (Trenton and Black River Group) with shaley laminations Bedrock cored from 8.3m to 11.4m depth For Bedrock coring details see Record of Drillhole 9				
68.0 / 11.4	End of Borehole Note: Water level in piezometer at 0.4m depth (Elev. 79.0m) on July 18, 2003.				

Plotted values:
- Elevation scale: 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68
- Shear strength (near elev. 73.8 to 73.5): >100kPa, 2.4, 1.8
- Shear strength (near elev. 72.2 to 71.9): 3.3, >100kPa, 4.4
- Water content (near elev. 78.3): 153

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

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

PROJECT: 03-1111-009

RECORD OF DRILLHOLE: 9

SHEET 1 OF 1

LOCATION: N 4898926.1 ;E 250508.7

DRILLING DATE: July 18, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CMB 55 BOMBARDIER

DRILLING CONTRACTOR: MARATHON DRILLING

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	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		.	70.90			
	Power Auger; 108mm Hollow Stem Auger	Grey, fresh, strong, thin to finely laminated, fine to medium grained LIMESTONE with dark grey shaley lamina (styolitic). Fractures are closely spaced. One 5mm thick shale bed at 9.6m depth, partially washed out during drilling.	8.30		B, SM, PL	
					B, SM, PL	
9				1	BC	
					B, SM, PL	
10					B, SM, PL	
					J, SM, PL	
				2	B, SM, PL	
11			67.80 11.40		J, SM, PL	
		End of Drillhole				
12						
13						
14						
15						
16						
17						
18						

Other column headings: SYMBOLIC LOG; 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); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

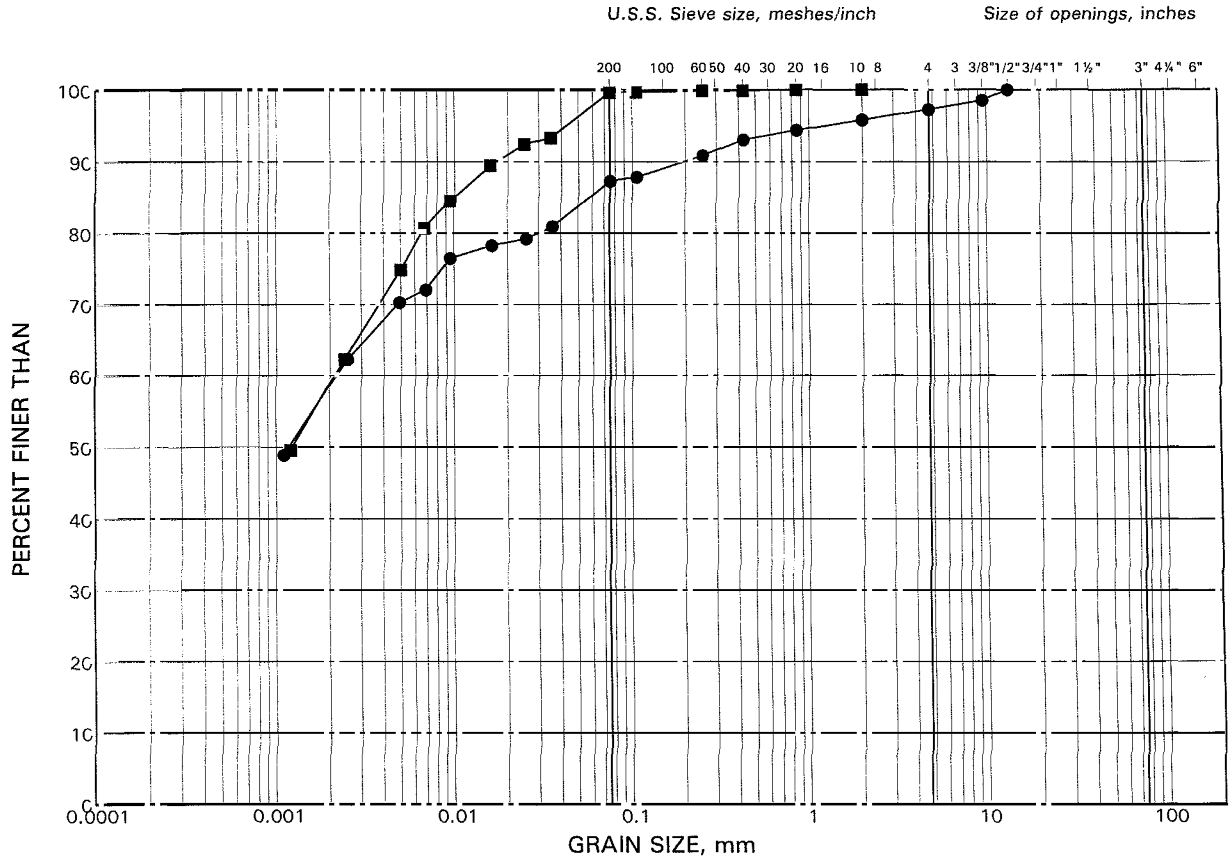
CHECKED: SP

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Silty Clay

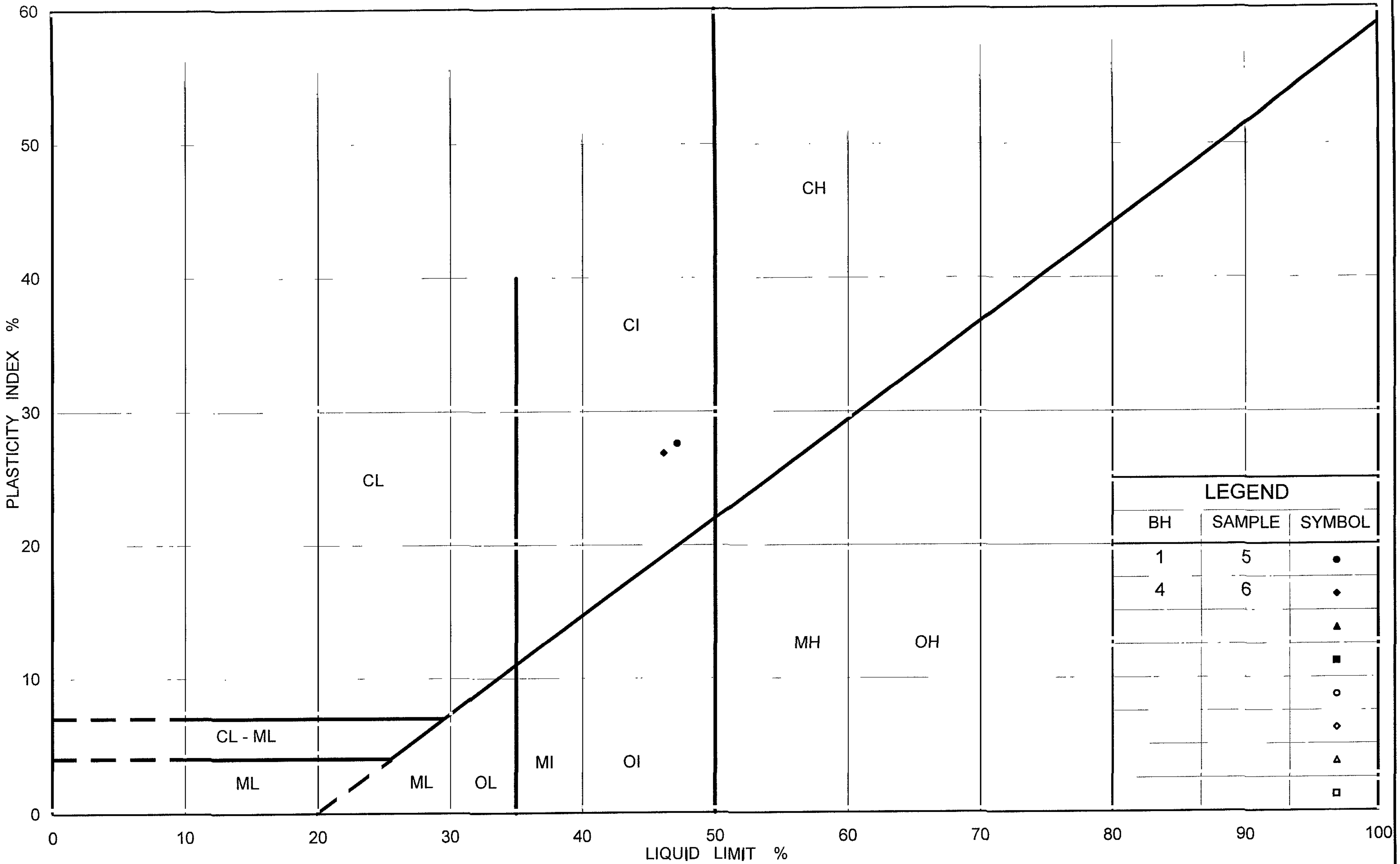
FIGURE A1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	1	5	76.9
■	4	6	75.7



LEGEND		
BH	SAMPLE	SYMBOL
1	5	●
4	6	◆
		▲
		■
		○
		◇
		△
		□

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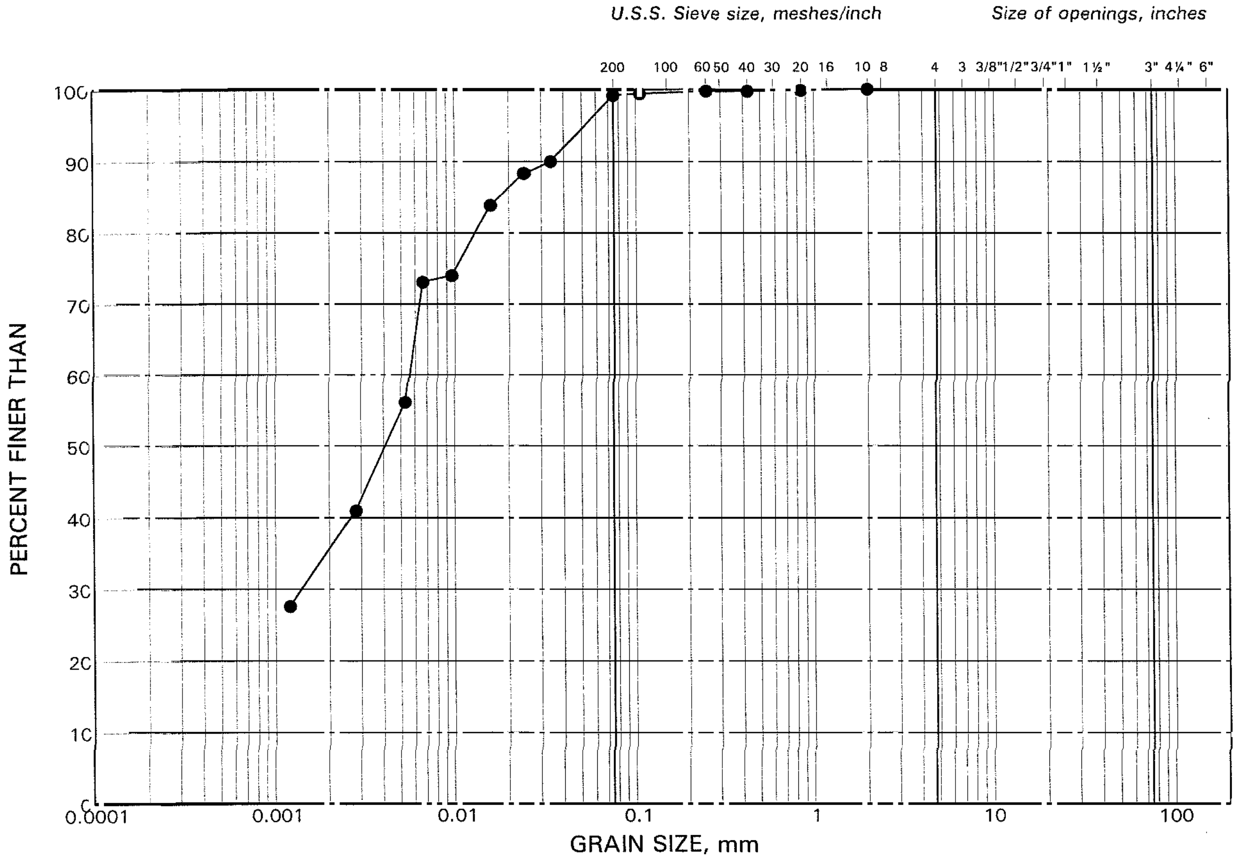
PLASTICITY CHART
Silty Clay

FIG No. A2

Project No. 031-111009

GRAIN SIZE DISTRIBUTION
Clayey Silt

FIGURE A3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	2	9	72.7

Project 031-111009

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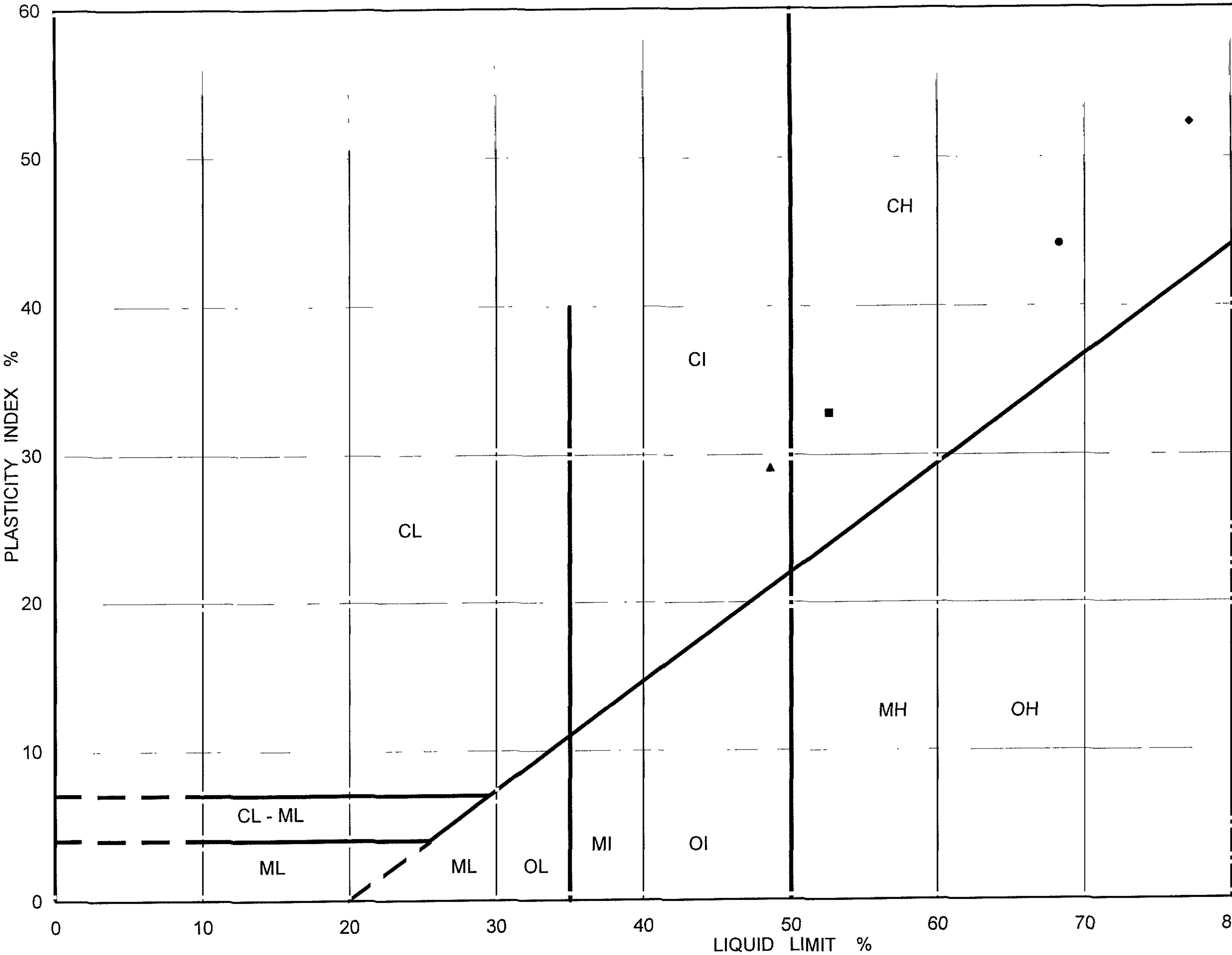
Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
2	7	●
3	6	◆
5	6	▲
8	4	■
		○
		◇
		△
		□

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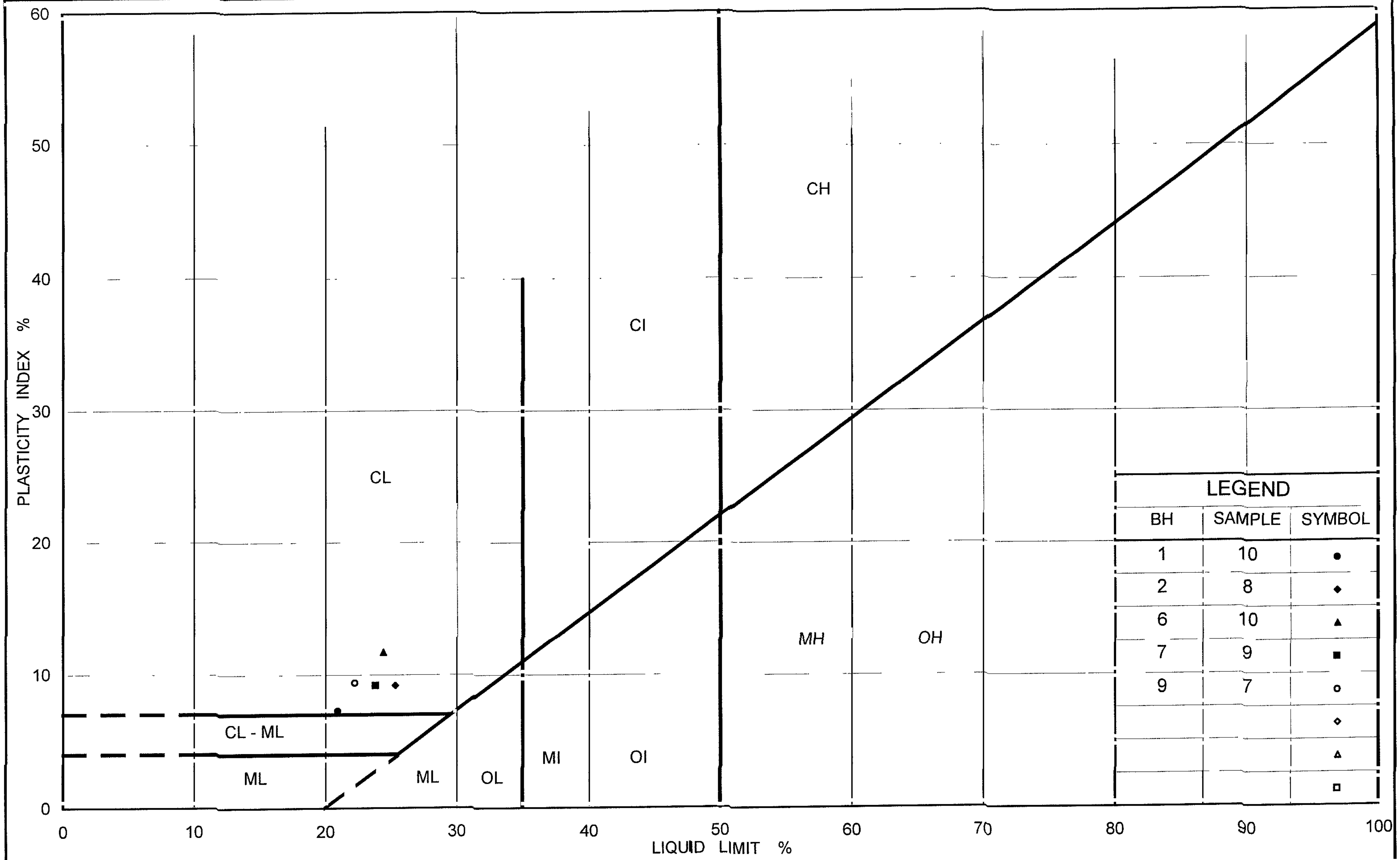
PLASTICITY CHART
Silty Clay to Clay

FIG No. A4

Project No. 031-111009

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
1	10	●
2	8	◆
6	10	▲
7	9	■
9	7	○
		◇
		△
		□

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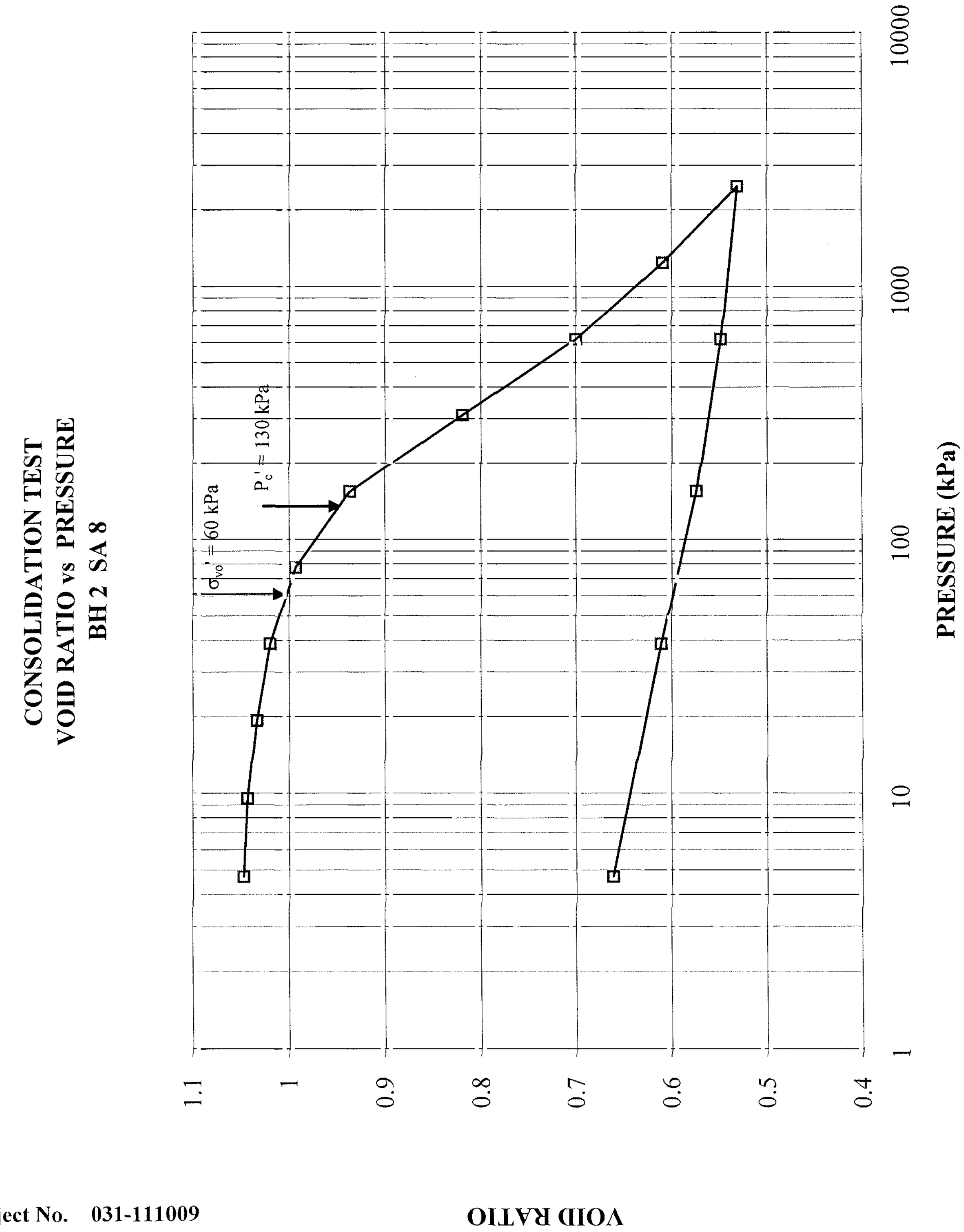
PLASTICITY CHART
Clayey Silt

FIG No. A5

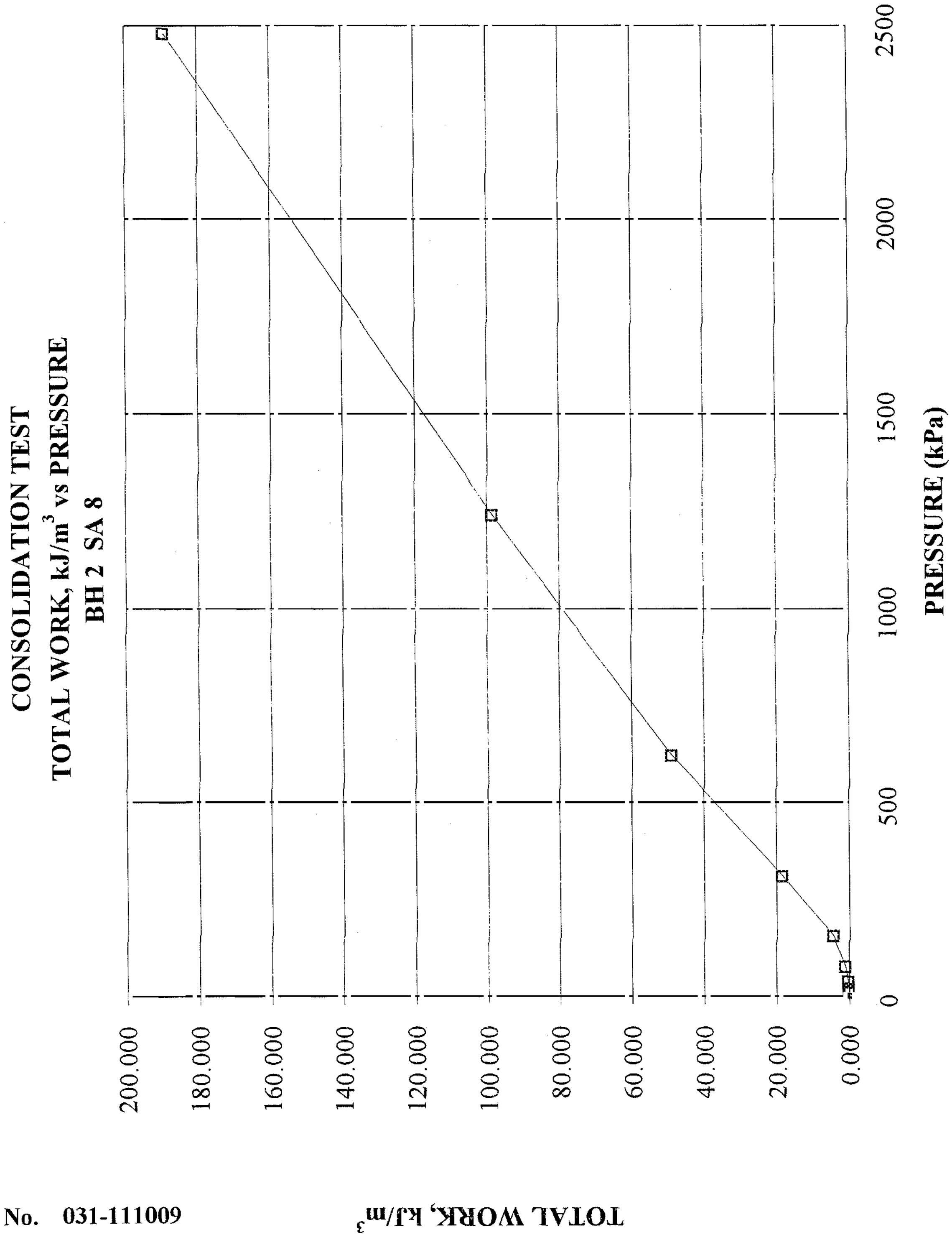
Project No. 031-111009

CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE	FIGURE A6



Project No. 031-111009

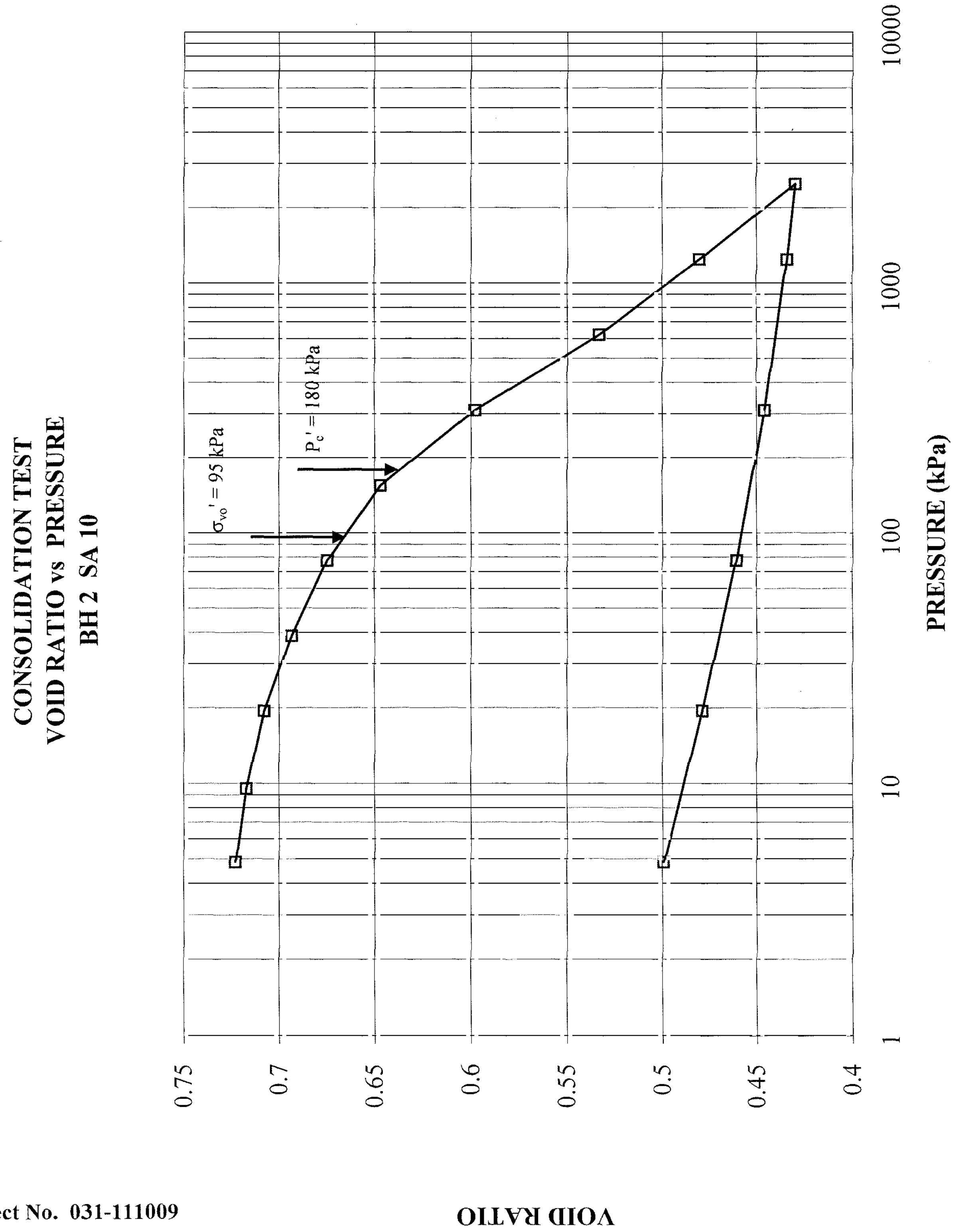
FIGURE A7



Project No. 031-111009

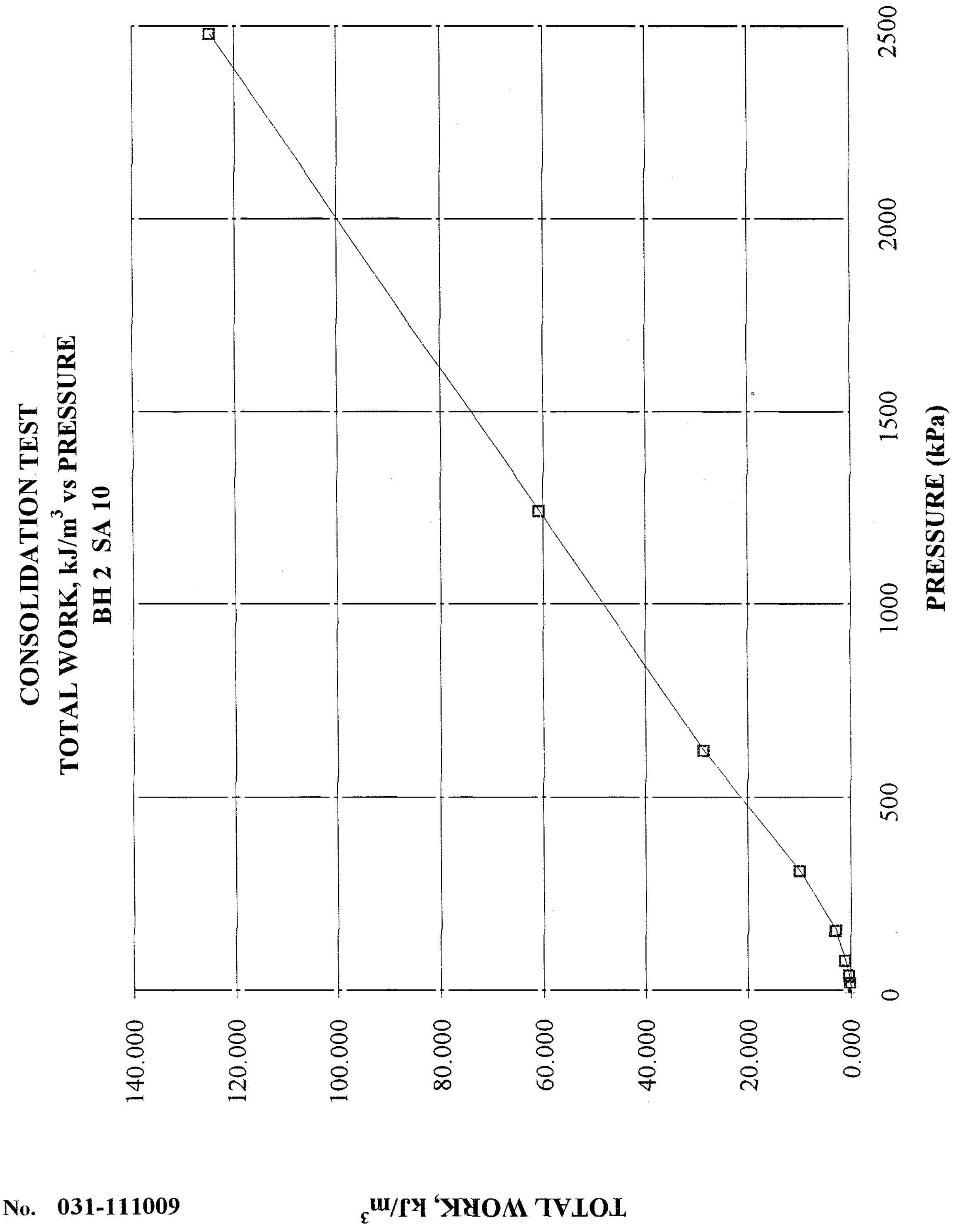
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A8



Project No. 031-111009

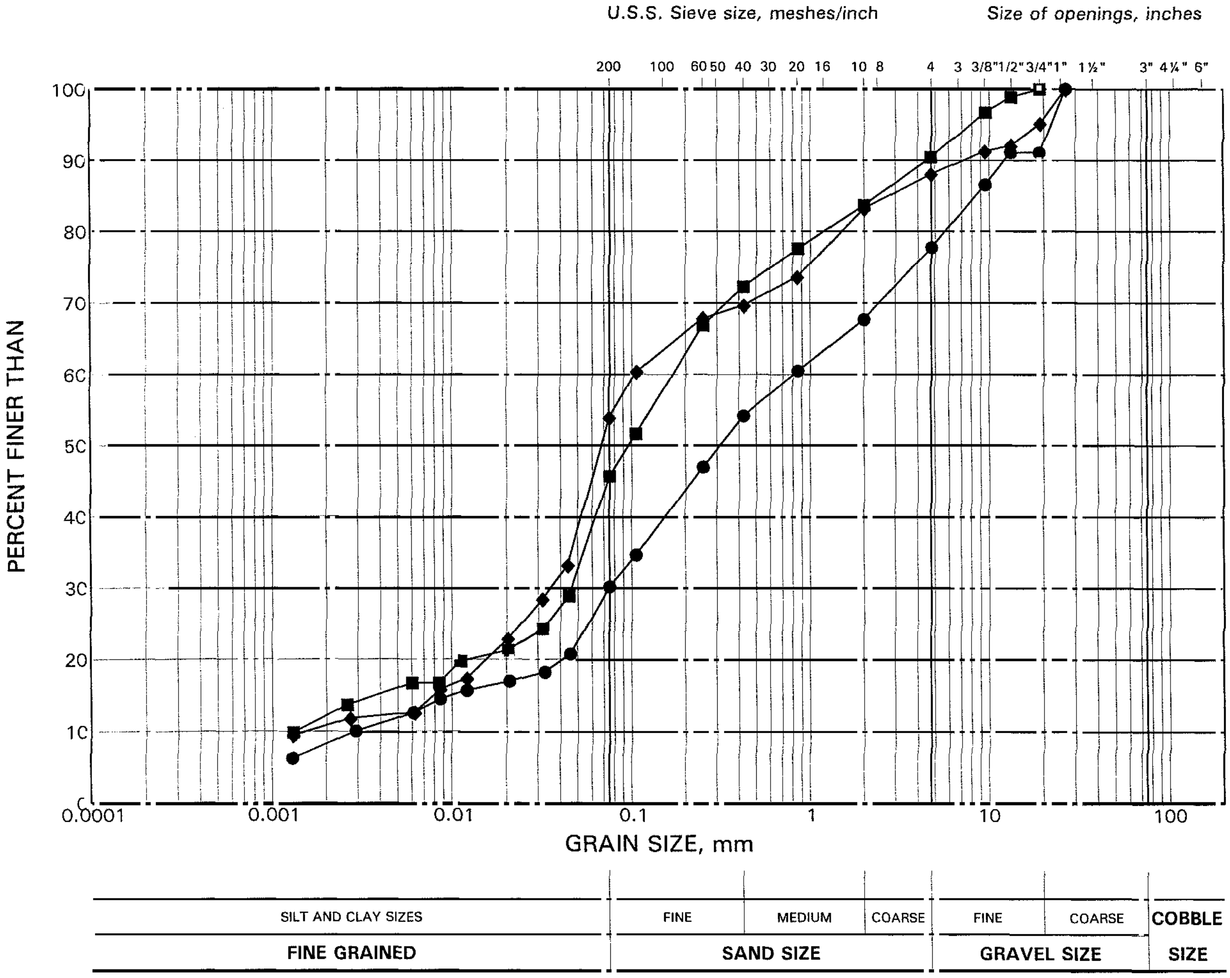
FIGURE A9



Project No. 031-111009

GRAIN SIZE DISTRIBUTION
Silty Sand (Till)

FIGURE A10



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	5	9	72.8
■	03-8	8	73.4
◆	03-9	9	71.2

Project 031-111009

Golder Associates

APPENDIX B

RECORD OF BOREHOLES FROM GEOCRES 31C-131 (1955)

BOREHOLES 6, 12 TO 15 AND RESULTS OF CONSOLIDATION TEST DATA

Order No.: S-500-501-55, T-66 [illegible], MacCALLUM AND ASSOCIATES Limited

Foundation Engineering Division

Engineering Data Sheet for Borehole: 6

Dated 16/2/55 (Day Month Year)

Hole Begun 28/2/55

Hole Ended 4/3/55

F. LUSK — Driller

W. LINTON — Helper

Job Name: TYENDINAGA TWP. ~~BRIDGE~~ No. 5 – SALMON RIVER

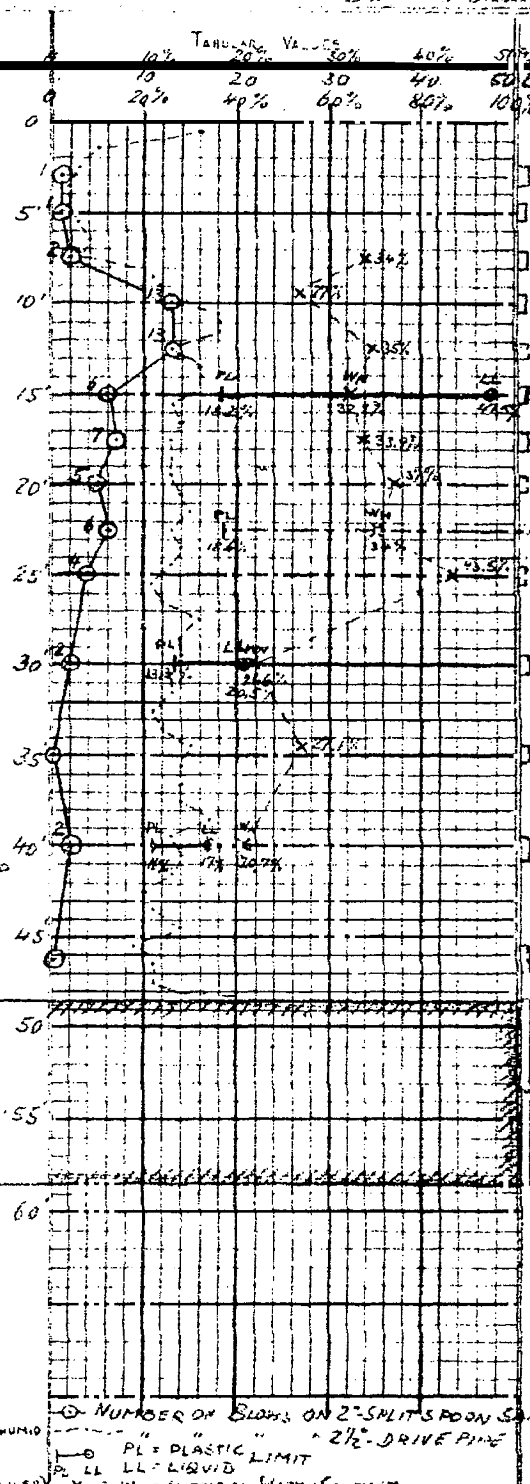
Job Located: PROP. HWY. No. 401 CROSSING SALMON RIVER; ~12 MILES E.N.E. OF BELLEVILLE, ONT.

Hole Located: AT NORTH END OF ISLAND IN SALMON RIVER; SEE ATTACHED SKETCH PLAN.

Hole Elevation: 263 Datum: M.S.L.

K. TUBBESING — Checked by 23/3/55 (Day Month Year)

Depth	El.	Thickness	W	Symbol	Narrative	Sampling
0	263				GROUND SURFACE	
		7.5'			VERY LOOSE, SLIGHTLY ORGANIC SILTY FINE TO MEDIUM SAND WITH COARSER LAYERS, SOME CLAY, CONTAINING SHELLS	SS-1, SS-2
7.5		1.5'			TRANSITION TO CLAY	SS-3
9'	254	~4.5'			STIFF DARK GREY CLAY	SS-4
					DO.	SS-5
~14.5		~13.5'			MEDIUM, CALCAREOUS, GREY, INORGANIC CLAY OF MEDIUM PLASTICITY	SS-6, SS-7, SS-8
					MEDIUM, CALCAREOUS GREY, INORGANIC CLAY OF HIGH PLASTICITY	SS-9, SS-10
~27'	236	~21.5'			TRANSITION TO	
					VERY SOFT CALCAREOUS, LIGHT GREY, FINE TO MEDIUM SILT (WATER CONTENT SLIGHTLY ABOVE LIQUID LIMIT)	SS-11, SS-12
					VERY SOFT CALCAREOUS, LIGHT GREY, MEDIUM SILT (NAT. WATER CONTENT EXCEEDING LIQUID LIMIT)	SS-13
					VERY SOFT SILT	SS-14
-48.5'	214.5				REFUSAL ON BEDROCK	Core Recovery:
					SOLID LIMESTONE, BEDDING FAIRLY HORIZONTAL. LAYERS ABOUT 2 TO 3" THICK. NO SHALES.	100%
-53.5'						100%
-58.5'						



Abbreviations: W WATER: S SATURATED, M MOIST, H HUMID, D DRY

S.S. SPLIT SPOON, T.W. THIN WALL SAMPLER

-⊙- NUMBER OF BLOWS ON 2" SPLIT SPOON SAMPLER

" " " 2½" DRIVE PIPE

PL = PLASTIC LIMIT, LL = LIQUID LIMIT

× = W_N = NATURAL WATER CONTENT

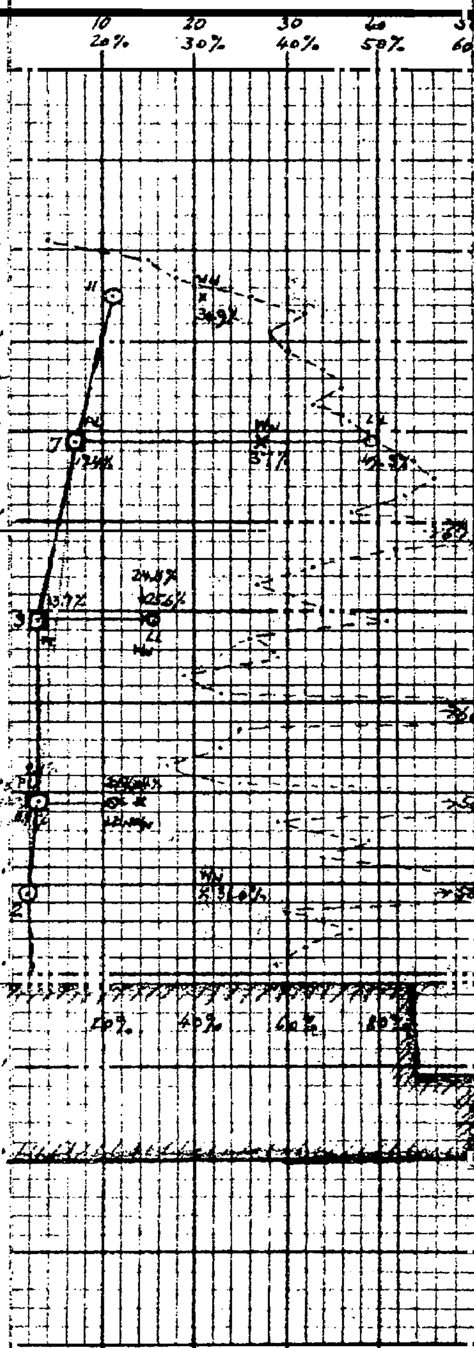
Order No.: S-500-501/55/T-66 RACEY, MacCALLUM AND ASSOCIATES Limited
Dated 16/2/55 (Day Month Year)
Foundation Engineering Division
Hole Begun 6/4/55
Hole Ended 16/4/55
Engineering Data Sheet for Borehole: 12

A. McCADDEN — Driller
D. McCURDY — Helper

Job Name: TYENDINAGA TWP. BRIDGE NO. 5 - SALMON RIVER
Job Located: PROP. HWY. NO. 401 CROSSING OVER SALMON RIVER; ~12 MILES E.N.E. OF BELLEVILLE
Hole Located: ON CENTRE LINE OF HWY., CENTRE OF MAIN BRANCH OF RIVER (ON RAFT), STA. 3632+75
Hole Elevation: ~262' (W.L.) Datum: M.S.L.

K. TUBBESING — Checked by 13/4/55 (Day Month Year)

Depth	El.	Thickness	Narrative	Sampling
0	262		▽ WATER LEVEL SALMON RIVER, 4/4/55	
		9'	WATER	
9'	253		SURFACE OF RIVER BED	
			SAND WITH WOOD, ORGANIC MATTER	
12.5			STIFF GREY CLAY	SS-1
		19'	STIFF GREY CLAY	TW-2
			MEDIUM GREY CLAY OF MEDIUM (TO HIGH) PLASTICITY	SS-3
			MEDIUM TO STIFF GREY CLAY	TW-4
28'	234			
			SOFT, LIGHT GREY CLAY OF LOW PLASTICITY	SS-5
		22.5'		TW6
			SOFT, LIGHT GREY SILT CLAY OF LOW PLASTICITY	SS 7
			VERY SOFT, LIGHT GREY SILT CLAY	SS 8, TW9
50.5	211.5		REFUSAL ON BEDROCK	SS-10
				CORE RECOVERY 87%
57'				100%
				100%
60'	202		END OF BOREHOLE	



ABBREVIATIONS
W. WATER; S. SATURATED; M. MOIST; H. HUMID; D. DRY
S.S. SPLIT SPOON; TW. THIN WALL (SAMPLER)

Order No.: 5-500-50/55/T66 RACEY, MacCALLUM AND ASSOCIATES LIMITED

Foundation Engineering Division

A. McCADDEN — Driller

Hole Begun 18/4/55

D. McCURDY — Helper

Hole Ended 20/4/55 **Engineering Data Sheet for Borehole:** 13

Job Name: TYENDINAGA BRIDGE No 5

Job Located: SALMON RIVER EAST OF BELLEVILLE

Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

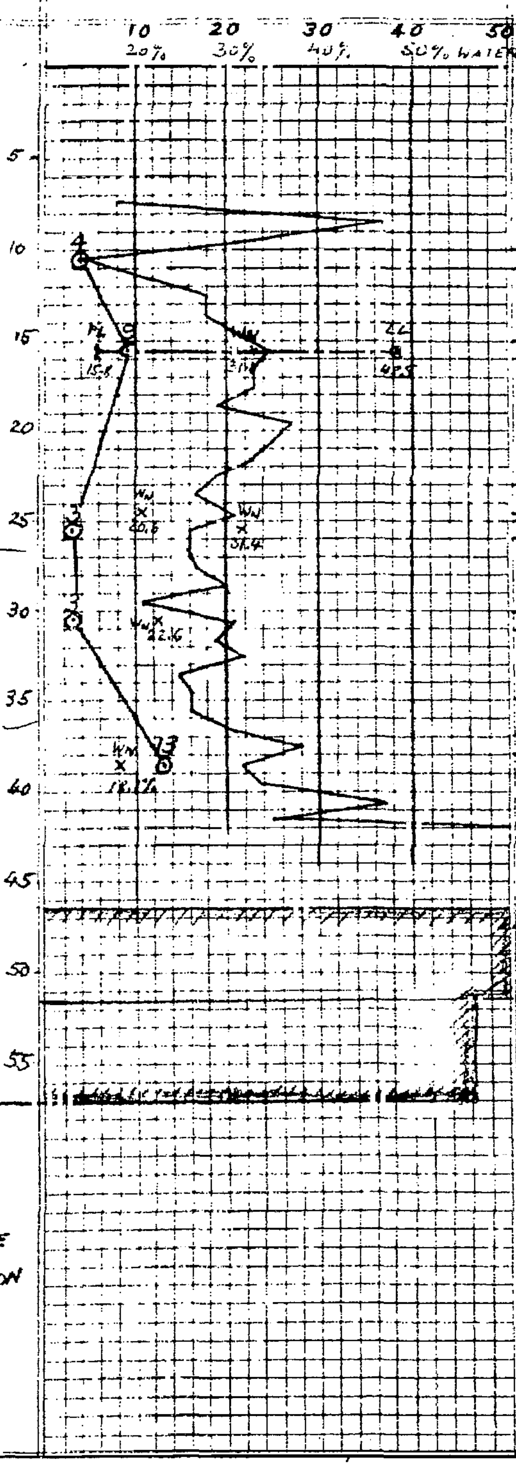
Hole Elevation: 261.5 (W.L.) Datum: M.S.L.

B.F WELSH & K.T. — Checked by

20 Day 4 Month 55 Year

DEPTH	EL	THICKNESS	DESCRIPTION	SAMPLING METHOD
0	261.5		WATER LEVEL	
		7.0	WATER	
7.0	254.5	4.5	LOOSE GRAVEL WITH CLAY AND ORGANIC MATTER	SS1
11.5		8.5	STIFF GREY SILTY CLAY	SS2
20.0		7.5	TRANSITION TO SOFT GREY SILTY CLAY	TW3, SS4
27.5		9.0	FINE TO MEDIUM SILT WITH SILT CLAY	SS5, TW LOST
36.5		5.5	SILT AND SILTCLAY WITH SOME GRAVEL	TW LOST, SS6
42.0		4.5	SILT AND SILTCLAY WITH GRAVEL AND SOME BOULDERS	168, 294, 331, 215, 446
46.5	215	10.5	SOLID GREY LIMESTONE	CORE RECOVERY: 100%, 93%
57'	204.5		END OF BOREHOLE	

TABULAR VALUES



- - - - BLOWS/FT. ON 3" Ø DRIVE PIPE

o—o—o BLOWS/FT. ON 2" Ø SPLIT SPOON

Order No.: S-500-50/55/?? TRACEY, MacCALLUM AND ASSOCIATES LIMITED

Foundation Engineering Division

A. McCADDEN — Driller

Hole Begun 20/4/55

D. Mc CURDY — Helper

Hole Ended 21/4/55 **Engineering Data Sheet for Borehole:** 14

Job Name: TYENDINAGA BRIDGE No. 5

B.F. WELSH & K.T. — Checked by

Job Located: SALMON RIVER E OF BELLEVILLE

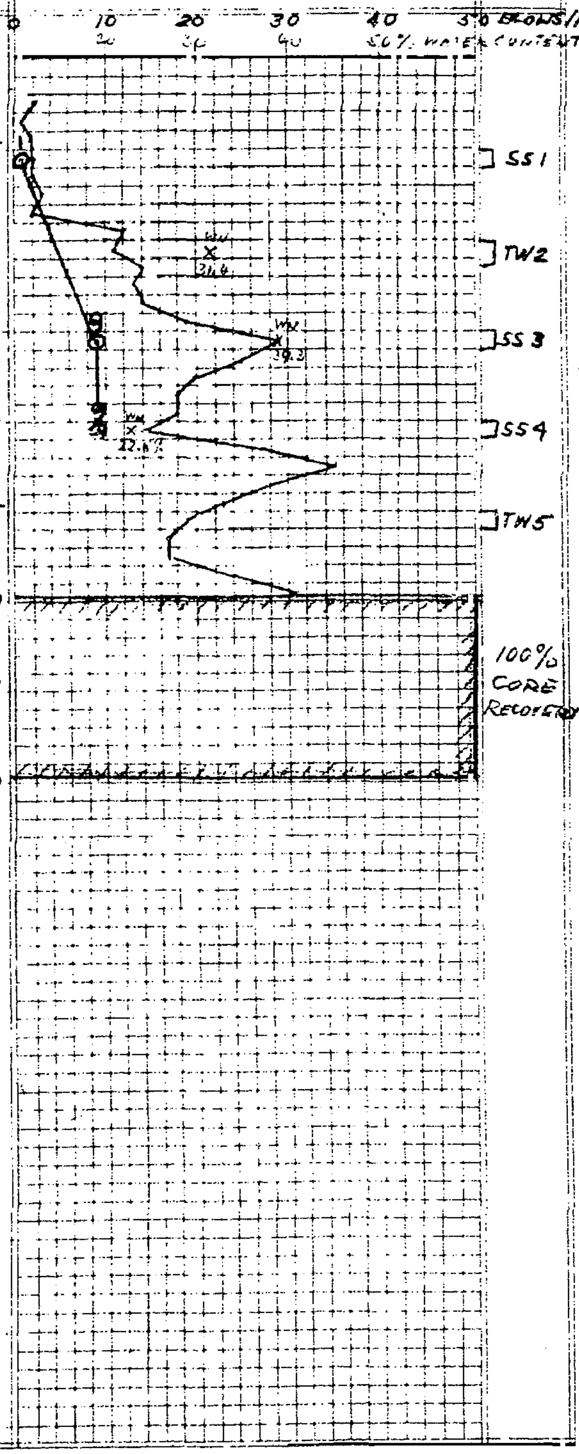
Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 261 Datum: M.S.L.

Day 22 Month 4 Year 55

DEPTH	EL	THICKNESS	SYMBOL	DESCRIPTION
0	261	1.7		WATER
1.7	259.3	7.3		VERY SOFT SANDY SILT WITH WOOD, TWIGS, ETC.
9.0		8.5		STIFF GREY CLAY
17.5		5.0		STIFF GREY SILTY CLAY
22.5		7.3		STIFF GREY SILT AND SILTCLAY
29.8	231	10		BEDROCK — GREY LIMESTONE (SOLID)
39.8	221			END OF BOREHOLE

TABULAR VALUES / SAMPLING METHOD



——— BLOWS/FT ON 3" DRIVE PIPE

o——o——o BLOWS/FT ON 2" SPLIT SPOON

Order No.: 5500-501/55/T-66 RACEY, MACCALLUM AND ASSOCIATES LIMITED — A. McCADDEN, Driller

Foundation Engineering Division

Hole Begun 22/4/55 — D. McCURDY, Helper

Hole Ended 23/4/55 — **Engineering Data Sheet for Borehole:** 15

Job Name: TYENDINAGA TWP. BRIDGE No. 5 - SALMON RIVER — B.E. WELSH & K.T., Checked by

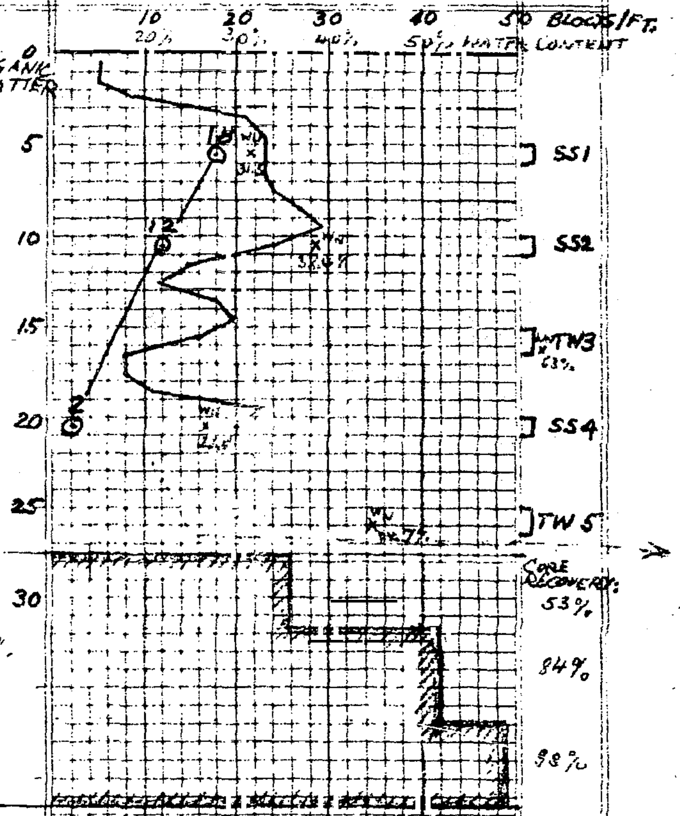
Job Located: PROP. HWY. # 401 CROSSING OVER SALMON RIVER, EAST OF BELLEVILLE, ONT.

Hole Located: ℄, STA. 36 31 + 50'

Hole Elevation: 263.5 Datum: M.S.L. — 25/4/55 (Day Month Year)

DEPTH	EL.	THICKNESS	SYMBOL	DESCRIPTION	SAMPLING METHOD
0	263.5	3'		VERY LOOSE SANDY SILT WITH ORGANIC MATTER W.L. AT 3.3' (23/4/55)	
3.0		12.5		TRANSITION FROM VERY STIFF BROWNISH GREY SILTY CLAY TO SOFT GREY SILTY CLAY	SS1 SS2
15.5				SOFT SILTCLAY WITH SILT	TW3 (W 63%) SS4 TW5
27.5				BEDROCK SOLID LIMESTONE, THICKNESS OF BEDS BY 2".	CORE RECOVERY: 53% 84% 98%
41.4				END OF BOREHOLE	

TABULAR VALUES



Enclosure No. 3

S-500-501/55/T-66
Tyendinaga Bridge #5
(Salmon River)