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## REPORT ON

### Preliminary Foundation Investigation and Design Cataraqui River Bridge Widening Structure Site No. 7-70 Highway 401, Kingston, Ontario W.P. 80-99-01

**Submitted to:**  
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REPORT



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Golder Associates Ltd., 2009



**PRELIMINARY FOUNDATION DESIGN REPORT  
CATARAQUI RIVER BRIDGE, HIGHWAY 401, KINGSTON, ONTARIO**

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# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
CATARAQUI RIVER BRIDGE UPGRADE  
HIGHWAY 401, KINGSTON, ONTARIO  
W.P. 80-99-01**





## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the Design-Build of bridge and culvert replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project. This report presents the results of a foundation assessment conducted for the upgrade of the Cataraqui River Bridge (Site 7-70) on Highway 401, located about 2.2 km east of Montreal Road in Kingston, Ontario.

As part of the current assignment, previously collected subsurface information pertinent to the site was reviewed and compiled. This existing subsurface information was contained in the following:

- Report prepared by Foundation of Canada Engineering Corporation Ltd. (FENCO) for the MTO (then the Ontario Department of Highways) titled *“Report to Ontario Department of Highways on Soil Conditions, Cataraqui River Bridge, Kingston, Ontario”*, dated April 21, 1954 (Geocres No. 54-F201C); and,
- Report prepared by Golder for the MTO titled *“Foundation Investigation and Design, Highway 401 Embankment Widening, Cataraqui Wetlands, Kingston, Ontario, G.W.P. 78-99-00”*, dated October 2012 (Geocres No. 31C-203).

The terms of reference and scope of work for the foundation engineering services are outlined in MTO’s Request for Proposal (RFP) for Assignment No. 4011-E-0017 dated January 2013, and in our proposal for services related to the Cataraqui River Bridge submitted to MMM, and dated February 21, 2013. No further field investigation was completed as part of the preliminary investigation phase.



## **2.0 SITE DESCRIPTION**

The site is located on Highway 401, about 2.2 km east of Montreal Road in Kingston, Ontario. The bridge was originally constructed in 1957 and consists of a three-span structure, approximately 93 m long and 28 m wide. It currently accommodates four travelled lanes. Based on the available General Arrangement structural drawings from 1954, prepared by FENCO, the piers and abutments are supported on concrete caissons founded on bedrock between about Elevation 68 and 73 m.

The natural ground surface within the lowland area surrounding the Cataraqui River valley is relatively flat at about Elevation 76 m based on the FENCO structural drawings. The base of the river channel is at approximately Elevation 71.9 m. To the west of the bridge, Highway 401 is elevated some 4 m above the lowland floodplains at about Elevation 80 m. The embankments that approach the existing bridge consist of embankment fill that is up to about 8 m high at the existing bridge structure. The pavement grade is at approximately Elevation 84.8 m at the bridge abutments. The highway embankment side slopes are oriented at about 2 horizontal to 1 vertical (2H:1V) in the immediate vicinity of the existing bridge. Based on visual observation at the time of the site investigation, the existing embankment slopes are relatively well vegetated and appear to be stable.

The Cataraqui River flows south through the site and into Lake Ontario. Based on the FENCO investigation report and structural drawings, the water level at the time of their investigation was at about Elevation 74.6 m. The high water level which is governed by the levels of Lake Ontario was indicated to be Elevation 76.2 m.



### **3.0 INVESTIGATION PROCEDURES**

The field work for the original investigation was carried out by FENCO between January 25 and February 11, 1954. At that time, a total of eight boreholes with dynamic penetration tests and three additional dynamic penetration tests were carried out at the site.

The borehole locations were measured relative to site features shown on the original site plan included in the FENCO report and their approximate locations relative to the current bridge deck and highway alignment are shown on Drawings 1 and 2. Boreholes BH 2, BH 3, and BH 5 were put down on the west side of the river. Boreholes BH 6, BH 7, BH 8, BH 9, and BH 12, as well as penetration tests PT 13, PT 14, and PT 15 were put down on the east side of the river.

The boreholes were drilled to depths ranging from 6.3 to 11.5 m below the ground surface at the time of the investigation. Soil samples were obtained using a split-spoon sampler driven during Standard Penetration Tests (SPTs) carried out in the overburden. Relatively undisturbed samples of the cohesive soil were also recovered in thin-walled Shelby tubes. Bedrock samples were recovered using rotary drilling techniques to obtain AXT-size rock core. The borehole logs from the FENCO investigation are provided in Appendix A.

Golder carried out a subsurface investigation west of the site in 2009 as part of the design stage for the proposed widening of Highway 401 to the west of the Cataraqui River Bridge. Boreholes E23 and E24, put down as part of that investigation, are relevant to the current assignment. The boreholes were put down near the toe of the existing embankment and advanced using portable/manual drilling equipment, to depths ranging from 2.1 to 3.2 m below the existing ground surface. Soil samples were obtained nearly continuously during SPT advancement. Borehole E23B was advanced adjacent to E23 to assess the probable shallow bedrock surface (i.e., refusal). Boreholes E23A, E24A, and E24B were extended beyond sampling using dynamic cone penetration techniques to assess the probable shallow bedrock surface (i.e., refusal). The borehole logs and laboratory test results from the Golder investigation are provided in Appendix B. Grain size distribution tests carried out on samples of the overburden are shown on Figures B1 and B2 in Appendix B. The results of Atterberg limit tests carried out on samples in boreholes west of the bridge site are shown on A-line plots on Figures B3 and B4 in Appendix B.

The FENCO report provided approximate borehole locations, relative to site features, and surveyed borehole elevations (relative to Geodetic datum). The locations and ground surface elevations of the boreholes advanced in 2009 were determined following drilling by Golder personnel at the site using a Trimble R8 GPS unit. The ground surface elevations at the time of drilling and approximate site locations are summarized in the table below, and are shown on Drawings 1 and 2.



## PRELIMINARY FOUNDATION DESIGN REPORT CATARAQUI RIVER BRIDGE, HIGHWAY 401, KINGSTON, ONTARIO

Borehole Number	Investigation	Adjacent Site Feature	Ground Surface Elevation (m)	Borehole Depth (m)
<b>West Bank</b>				
BH 2	FENCO, 1954	West Abutment (north)	76.1	8.2
BH 3	FENCO, 1954	West Abutment (south)	75.1	9.9
BH 5	FENCO, 1954	West Shoreline (south)	74.6	9.6
E23	Golder, 2009	West Embankment Toe (south)	76.6	3.2
E24	Golder, 2009	West Embankment Toe (south)	75.1	2.1
<b>East Bank</b>				
BH 6	FENCO, 1954	East Shoreline (north)	75.1	11.5
BH 7	FENCO, 1954	East Shoreline (south)	75.0	6.3
BH 8	FENCO, 1954	East Abutment (north)	75.6	11.0
BH 9	FENCO, 1954	East Abutment (south)	75.9	11.1
BH 12	FENCO, 1954	East Shoreline (central)	75.1	6.3
PT 13	FENCO, 1954	East Shoreline (south)	75.1	2.1
PT 14	FENCO, 1954	East Shoreline (south)	75.6	2.1
PT 15	FENCO, 1954	East Shoreline (north)	75.0	3.4

**Note:** Ground surface elevations were determined at the time of each geotechnical investigation.



## **4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Regional Geology**

This area of Highway 401 lies within the southern portion of the physiographic region known as the Napanee Plain, as delineated in *The Physiography of Southern Ontario*<sup>1</sup>.

The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping<sup>2</sup> indicates that the bedrock within the southern portion of the area consists of both granitic rock and crystalline limestone. In many areas bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the northern and southern portion of the Plain, and within and adjacent to river valleys throughout the Plain.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay.

In particular, the study area lies within the lowland area surrounding the Cataraqui River. The Cataraqui River is characterized by a number of lakes joined by the river. This river flows southerly towards Kingston and is one of two major rivers in the area.

### **4.2 Subsurface Conditions**

Eight boreholes with penetration tests and three additional penetration tests were carried out at the site as part of the original investigation in 1954. Two boreholes and associated penetration tests were advanced at the southern toe of the existing west approach embankment by Golder in 2009. The approximate borehole locations and surveyed ground surface elevations at the borehole locations are shown on Drawings 1 to 4. The interpreted stratigraphic conditions at the site are shown on Drawings 1 to 4, inclusive.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records from the 1954 investigation, included in Appendix A, and from the 2009 investigation, included in Appendix B. The stratigraphic boundaries shown on the borehole records, and on the interpreted stratigraphic sections included on Drawings 1 to 4, are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. Similarly, the ground surface conditions at these borehole locations may have changed since the time of drilling, particularly for those boreholes put down in 1954, prior to construction of the bridge.

In general, the subsurface conditions at the site consist of surficial layers of organic material or rock fill overlying very stiff to hard silty clay, which is underlain by a thin deposit of sand and gravel at some borehole locations. The soil deposits are underlain by syenite and granite bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

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

<sup>2</sup> Map 13-1965, Geological Survey of Canada, 1966.



#### **4.2.1 Surficial Deposits**

During the 1954 investigation, a surficial layer of topsoil was encountered at most borehole locations. The topsoil thickness generally ranged from 0.3 to 0.8 m thick. At Borehole BH 5, the topsoil was described as soft organic clay and contained wood pieces. The topsoil has a soft consistency, based on the penetration tests carried out at the borehole locations.

At Boreholes BH 7 and BH 12, rock fill was encountered at the ground surface. The thickness of the rock fill was about 1.6 m at Borehole BH 7 and 0.3 m at Borehole BH 12. Penetration test results indicated that the rock fill had a loose to very dense relative density.

Surficial fill was encountered at Boreholes E23 and E24 and consisted of soft clayey silt and fibrous peat, respectively.

#### **4.2.2 Silty Clay**

A deposit of silty clay was encountered beneath the surficial deposits at all borehole locations with the exception of Borehole BH 7 where no silty clay was encountered. The thickness of the silty clay deposit ranged up to about 6.1 m.

The upper portion of the silty clay has been weathered to a grey-brown crust and was observed to contain variable amounts of sand, gravel, and silt lenses. The thickness of the weathered zone is variable and ranges up to about 3.3 m in the boreholes put down at higher elevations. The bottom of the weathered zone was encountered between about Elevation 71.9 and 72.5 m.

The unconfined compressive strength of samples of the weathered crust recovered from the 1954 investigation ranged from about 180 to 280 kPa. SPT 'N' values for the weathered silty clay ranged from 29 to greater than 100 blows per 0.3 m of penetration, but were typically greater than about 35 blows per 0.3 m of penetration, indicating a generally hard consistency.

Atterberg limit tests carried out on samples of the weathered silty clay gave plasticity index values ranging from 20 to 28 percent and liquid limit values ranging from about 39 to 49 percent, indicating a moderate plasticity. The measured water content of the weathered silty clay ranged from approximately 27 to 33 percent. The results of grain size distribution testing carried out on samples of the silty clay from Boreholes E23 and E24 are included on Figure B1 in Appendix B.

The silty clay soil below the depth of weathering is a layered dark grey to lighter grey, and it varied in thickness up to about 3.8 m but was typically on the order of 3 m, where encountered.

The unconfined compressive strength of samples of the unweathered silty clay recovered from the 1954 investigation ranged from about 50 to 160 kPa, but was typically around 100 kPa. SPT 'N' values for the grey silty clay ranged from 28 to 78 blows per 0.3 m of penetration, indicating a generally very stiff to hard consistency.

An Atterberg limit test carried out on a sample of the grey silty clay from Borehole BH 9 gave a plasticity index value of 26 percent and a liquid limit value of 50 percent, indicating a moderate plasticity. The measured water content of the grey silty clay ranged from approximately 33 to 35 percent.



#### **4.2.3 Sand and Gravel**

A deposit of sand and gravel containing some silt and clay was encountered beneath the silty clay at Boreholes BH 3, BH 5, BH 6, and BH 9. The sand and gravel was encountered directly above the bedrock that underlies the site at locations where the bedrock is deepest between Elevation 68.5 and 70.2 m. The thickness of the deposit ranged from about 0.1 to 0.5 m.

The sand and gravel has a compact to dense relative density, based on one measured SPT “N” value of 30 blows per 0.3 m of penetration.

#### **4.2.4 Bedrock**

Bedrock consisting of hornblende syenite, syenite gneiss, or granite was encountered beneath the overburden soils at the site. Bedrock was encountered at depths ranging from 1.6 to 7.0 m below the ground surface at the time of drilling.

Near the eastern shoreline, the bedrock surface elevation generally increases from about 68.1 m at BH 6 (north of the highway), to about 73.4 m at BH 7 (south of the highway). At the boreholes put down near the eastern abutment, the bedrock was encountered between Elevation 69.2 and 69.4 m. On the west side of the river near the western abutment, the bedrock surface elevation decreases from 73.2 m at BH 2 (north of the highway) to about 69.8 m at BH 3 (south of the highway) and then increases to about 73.0 m at Borehole E24 (south of the embankment toe).

The bedrock cored as part of the 1954 investigation by FENCO is described in their 1954 report as “consistently fresh, sound and massive”. Further, the 1954 report noted that “generally excellent core was obtained of the rock. Some joints were noted, but they were consistently cemented with a chlorite deposit.” Hornblende syenite, syenite gneiss, and granite are igneous and metamorphic rocks that would typically be described as strong to extremely strong.

### **4.3 Groundwater Conditions**

Groundwater levels at the borehole locations were not recorded as part of the investigation. The water level within the Cataraqui River was measured to be Elevation 74.6 m on February 6, 1954. The water level in the Cataraqui River and adjacent Cataraqui Swamp encountered during Golder’s 2008/2009 investigation ranged from about Elevation 74.5 to 75.1 m. In general, the groundwater level within the soil at the site is anticipated to be at a similar elevation.

The groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.






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### 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.


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**PRELIMINARY FOUNDATION DESIGN REPORT  
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# **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
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## **6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides preliminary foundation design recommendations for the proposed widening of the existing Cataraqui River Bridge on Highway 401. The recommendations are based on interpretation of the factual data included in the original 1954 foundation design report (FENCO) and Golder's 2012 report addressing widening of the Highway 401 embankments to the west of the site. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the widened/upgraded structure. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, and scheduling.

The design options to accommodate the widening/rehabilitation of the existing Cataraqui River Bridge considered by MMM involve construction of a total of two additional lanes (one in each travelled direction) on the outside of the existing lanes, for a total of six lanes. The additional lanes would be supported on a widened bridge structure. During preliminary design, MMM narrowed the possible foundation options to the following alternatives:

- Support of the widened bridge structure on the existing pier and abutment caissons;
- Support of the widened bridge structure on new foundations constructed immediately adjacent to the existing pier and abutment caissons; or,
- Support of the widened bridge structure on new foundations constructed at an offset of approximately 4 m from the existing pier and abutment caissons.

Based on information provided by MMM, the overall increase in width of the approach embankments would be a maximum of about 10 m and would include up to an additional 5 m width on each side of the embankments; there is essentially no grade increase planned over the existing Highway 401 grade. If the widened bridge structure is to be supported on the existing caisson foundations, construction could be carried out from the bridge deck. If the widened bridge structure is to be supported on new foundations constructed beyond the existing caissons, consideration will have to be given to whether the proposed foundation locations in the river are to be accessed from floating equipment, from the nearby river bank, or from the existing bridge deck.

Further details and consideration of the proposed foundation options are discussed in the following sections.

### **6.2 Foundation Options**

The existing Cataraqui River Bridge is a three-span bridge originally constructed in 1957. The available structural drawings (by FENCO, originally dated August 16, 1954) indicate that the bridge is founded on concrete caissons bearing on the bedrock surface encountered at the following elevations.



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	Founding Elevation (m)	Depth Below Floodplain Grade (m)
West Abutment	70.1	5.9
Pier 1 (West)	71.0 – 72.8	5.0 – 3.2
Pier 2 (East)	70.4 – 73.2	5.6 – 2.9
East Abutment	68.6	7.4

**Note:** Floodplain elevation assumed to be about Elevation 76.0 m.

According to the 1954 structural drawings and the base and contour plans provided therein, the average floodplain grade and base of the river channel are at about Elevation 76.0 and 71.9 m, respectively.

Based on the subsurface conditions, deep foundation options have been considered for the widening/rehabilitation of the Cataraqui River Bridge. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and approximate costs is provided in Table 1 following the text of this report.

- **Use of existing caisson foundations:** Supporting the widened bridge structure on the existing caisson foundations is feasible from a geotechnical point of view. Provided this option is structurally feasible, no significant additional foundation construction would be required. Depending on the eccentricity and magnitude of the additional loads imposed on the existing foundations by the widened bridge structure, installation of rock anchors through the existing caissons and into the underlying bedrock may be required to provide a moment reaction with uplift resistance.
- **New caissons:** Caissons constructed adjacent to the existing caissons and founded on the granitic bedrock are feasible for this site, but founding depths would have to be designed to be compatible with those of the existing caissons. The new caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss during construction and would likely need to be nominally socketed into the bedrock. Since the existing caissons are assumed to be founded near the surface of the bedrock, socketing of new caissons to significant depth below the bedrock surface may result in a bearing capacity reduction of the existing caissons.
- **Driven steel H-piles:** Steel H-piles driven to refusal on the granitic bedrock are feasible and suitable for support of the widened bridge structure and would minimize the need for protection system requirements and groundwater control requirements. If the piles are driven, the use of driving shoes is recommended to minimize damage while driving to the granite bedrock. However, steel piles may not be suitable where the depth to bedrock is relatively shallow. If rock sockets are required to achieve a minimum pile length to meet structural requirements or to achieve pile toe fixity under seismic loading events, coring or use of a down-hole hammer would be necessary in the granite bedrock.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option, and this foundation option would have similar advantages to steel H-piles in terms of protection system requirements and groundwater control requirements. However, pipe piles are considered to have a slightly higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the possible presence of cobbles and/or boulders.



- **Micropiles:** Deep foundations consisting of drilled and grouted micropiles are feasible for support of the widened bridge structure, particularly where the variable bedrock surface at the site is relatively shallow. The construction equipment/techniques and risks associated with micropiling are similar to those required for drilled piles or for pre-drilled holes for H-piles to advance through cobbles and boulders or into the granitic bedrock. However, because micropiles require a smaller hole than pre-drilled H-pile holes or drilled piles, installation is generally faster and vibration-induced movements on adjacent structures would likely be less.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the widened bridge structure on the existing caisson foundations. However, if this is not considered to be feasible from a structural perspective, support of the bridge on either caissons or drilled micropiles, constructed at an offset from the existing caissons, is considered to be suitable.

## **6.3 Use of Existing Caissons**

If considered to be structurally feasible, the widened bridge may be supported on the existing caisson foundations. Based on the original 1954 geotechnical design report, the existing caissons are founded directly on the granitic bedrock. Though it is unclear whether or how far the caissons were set into the bedrock, it is assumed that they may have been founded a nominal depth (approximately 0.1 m) below the bedrock surface if weathered rock was encountered at the time of construction.

The existing caissons may be considered to derive their axial reaction entirely from end-bearing resistance. For preliminary design purposes, the bedrock may be considered to have a factored axial geotechnical resistance at Ultimate Limit State (ULS) of 8 MPa. Serviceability Limit State (SLS) resistances do not apply to footings founded in the granite bedrock as the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

### **6.3.1 Rock Anchors**

The widened bridge structure would extend away from the existing foundation footprint and would “hang off” the side of the foundations. The additional loading on the foundations would be applied eccentrically at some distance from the vertical axis of the existing, unreinforced concrete caissons. The resulting moment applied on the caissons would require uplift resistance at the base of each caisson, on the inner portion of the caisson footprint nearest to the centreline of the bridge.

The use of rock anchors, installed through the existing abutments/piers and caissons and into the underlying bedrock, may be considered to provide the required uplift resistance. The rock anchors would be installed from the existing bridge deck and could consist of either grouted or mechanical anchors.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes:

- i) Failure of the steel tendon or top anchorage;
- ii) Failure of the grout/tendon bond;
- iii) Failure of the rock/grout bond; and,
- iv) Failure within the rock mass, or rock cone pull-out.

Potential failure modes i) and ii) are structural and are best addressed by the bridge engineer.



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For potential failure mode iii), the factored bond stress of the anchor beneath the caisson at the concrete/rock interface may be taken as 1,000 kPa for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance is calculated based on the unit weight (undrained) of the potential mass of rock that could be mobilized by the anchor, and resistance to shear of the rock mass. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where:

$Q_r$	=	Factored uplift resistance of the anchor (kN);
$\phi$	=	Resistance factor (use 0.4);
$\gamma'$	=	Effective unit weight of rock (use 17 kN/m <sup>3</sup> below the groundwater level);
$D$	=	Anchor length in m; and,
$\theta$	=	one-half of the apex angle of the rock failure cone (use 30 degrees).

For a group of anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. Further guidelines can and must be provided for assessing the anchor resistance for these conditions during detail design.

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained during supplementary investigations carried out during the detailed design.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out to 1.3 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

Qualified geotechnical personnel must be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids, particularly for installations of significant length through the existing caissons. Confirmation of sufficient embedment into the rock beneath the caissons should be carried out by qualified geotechnical personnel to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock.

Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.



## 6.4 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

### 6.4.1 Founding Elevations

New abutments and piers constructed for the widened bridge structure may be supported on steel H-piles or steel pipe (tube) piles driven to found on the granitic bedrock. Additional borehole investigation will be required at the detail design stage to confirm the bedrock surface variability beyond the footprint of the existing structure. However, based on the borehole results from the original 1954 investigation, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, the following approximate pile tip elevations are recommended for preliminary design:

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
<b>Widening of Eastbound Lanes – South of Existing Structure</b>			
West Abutment	BH 3	69.8	69.7
Pier 1 (west)	BH 5	69.7	69.6
Pier 2 (east)	BH 7	73.4	66.9*
East Abutment	BH 9	69.4	69.3
<b>Widening of Westbound Lanes – North of Existing Structure</b>			
West Abutment	BH 2	73.2	71.0*
Pier 1 (west)	BH 2	73.2	66.9*
Pier 2 (east)	BH 6	68.1	68.0
East Abutment	BH 8	69.2	69.1

**Note:** \* Pile tip extended into bedrock by coring/churn drilling to achieve a minimum pile length of 5 m; shorter pile lengths with higher founding elevations may be feasible from a structural/seismic perspective.

Where bedrock is encountered at relatively shallow depth, the piles may have to be extended into the bedrock to achieve a minimum pile length to satisfy structural/pile toe fixity requirements; for preliminary purposes, a minimum pile length of 5 m has been assumed. Based on the existing subsurface information, this may be required at the eastern pier (south of the bridge) as well as at the west abutment and western pier (north of the bridge). At these locations, coring or use of a down-hole hammer would be necessary to form a socket in the strong granitic bedrock.

The abutment pile caps should be constructed at a minimum depth of 1.5 m below the ground surface for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).



Due to the potential for sloping bedrock at the site (based on the results from the 1954 borehole investigation), the piles should be reinforced at the tip with rock points to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 (*Deep Foundations*). If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II (*Steel Tube Pile Driving Shoe*).

#### **6.4.2 Axial Geotechnical Resistance**

For preliminary design of HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the granitic bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.

Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). The drawings should incorporate the appropriate note stating that the piles should be equipped with bearing points and should be driven to bedrock. For piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity of 100 mm per second is recommended at the existing abutments and piers. The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the pile driving procedures for the remaining piles.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the site.

### **6.5 New Caissons**

As an alternative to steel H-piles or pipe piles, construction of supplementary caissons could be considered for support of the new or widened abutments if new foundations are required. Due to the proximity to the river, temporary or permanent liners would be required during caisson construction.

#### **6.5.1 Founding Elevations**

If caisson foundations are adopted, the abutment pile caps should be constructed with a minimum founding depth of 1.5 m below the lowest surrounding grade, to provide adequate protection against frost penetration.

It is recommended that the caissons be socketed nominally (approximately 0.2 m) into the bedrock to allow for some weathering/fracturing of the upper portion of the bedrock, and to minimize the potential for loss of soils at the soil-bedrock interface during caisson construction. Since the existing caissons are assumed to be founded near the bedrock surface, the new caissons should not be founded significantly deeper than the existing caissons.





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Additional borehole investigation will be required at the detail design stage to confirm the bedrock surface variability beyond the footprint of the existing structure and within that of the proposed caissons. However, based on the borehole results from the preliminary investigation, the following caisson founding levels are recommended for preliminary design:

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)	Design Caisson Base Elevation (m)
<b>Widening of Eastbound Lanes – South of Existing Structure</b>			
West Abutment	BH 3	69.8	69.6
Pier 1 (west)	BH 5	69.7	69.5
Pier 2 (east)	BH 7	73.4	73.3
East Abutment	BH 9	69.4	69.3
<b>Widening of Westbound Lanes – North of Existing Structure</b>			
West Abutment	BH 2	73.2	73.0
Pier 1 (west)	BH 2	73.2	73.0
Pier 2 (east)	BH 6	68.1	67.9
East Abutment	BH 8	69.2	69.0

The granitic bedrock is anticipated to be strong. Therefore, the sockets would have to be advanced into the rock by churn drilling or rock coring.

### 6.5.2 Axial Geotechnical Resistance/Reaction

Caissons socketed approximately 0.2 m or greater into the bedrock should be designed based on end-bearing resistance, using a factored axial geotechnical resistance at ULS of 8 MPa for preliminary design purposes. For a 0.9 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of about 5,000 kN. Serviceability Limit States (SLS) resistances do not apply to caissons founded nominally within the granitic bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

## 6.6 Micropiles

Based on the original 1954 foundation design report (FENCO), the existing caissons are assumed to be founded near the surface of the bedrock. To minimize disturbance of the existing foundations and reduce the influence of sloping and shallow bedrock during construction of the new foundations, drilled and grouted micropiles may be considered for support of the new abutments and piers.

Micropiles are small diameter (typically less than 300 mm) drilled and grouted replacement piles that are typically reinforced with high-capacity steel (typically threaded bars or reinforced steel) to resist a high proportion of the design load. Micropiles are often cased through overburden deposits and then socketed and grouted directly





into bedrock at depth. The casing may or may not be removed, depending on the structural stiffness and axial/lateral capacities required. The special drilling and grouting methods used in micropile installation allow for high grout-to-ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. Theoretically, micropiles can have equal capacities in both tension and compression.

The design capacity of a micropile is based on the bonded surface area of the cylinder of grout in contact with the ground in the bond zone. The total pile length (i.e., embedment into the underlying bedrock) is determined by detailed pile design calculations. Larger diameter micropiles have the advantage of requiring a shorter bond length to accommodate the required design loads and therefore will need less drilling into the bedrock (i.e., overall shorter length pile). Larger diameter micropiles are also stiffer and provide more buckling resistance over the free length of the pile. However, drilling with the larger diameter through the strong to extremely strong granitic bedrock may be more time-consuming and poses a potentially greater risk of vibration, ground loss or settlement and movement of the existing structure. Smaller diameter micropiles may mitigate some of these risks, but would require a longer bond zone (i.e., overall longer length of pile) to achieve the same capacity.

The effects of a smaller cross-section on the lateral stiffness and buckling potential of the pile must be considered from a structural point of view.

### **6.6.1 Axial Geotechnical Resistance**

The axial geotechnical resistance of the micropiles will be primarily developed within the bond zone or the uncased lower section of the micropile socketted into the granitic bedrock. The grout-to-ground bond strength is estimated based on the results of uniaxial compressive strength (UCS) testing of the bedrock and from values recommended in state-of-practice design manuals.

UCS test results were not included as part of the original investigation. However, a preliminary estimate of the factored axial geotechnical resistance has been developed assuming typical values for the strong to extremely strong granitic bedrock at the site and based on recommended values found in *Recommendations for Prestressed Rock and Soil Anchors* (PTI 2004) and *Micropile Design and Construction Reference Manual* (FHWA/NHI 2005).

For a typical micropile consisting of an HSS 273 x 13 (10-3/4" x 1/2" wall) casing and a minimum diameter of 0.229 m (9") in the bond zone socketted into good quality granitic bedrock, a factored axial geotechnical resistance at ULS of 1,000 kN could be achieved with one #18 (2-1/4") bar, a 30 MPa grout, and a minimum bond length of 2.0 m. The axial resistance could be increased with a greater bond length (longer pile), larger diameter socket, and/or larger diameter bars. Using a #24 bar (3"), a 35 MPa grout, and a minimum bond length of 2.5 m, a factored axial geotechnical resistance at ULS of 1,600 kN could be achieved. For drilled micropiles founded in bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

The axial geotechnical resistances given above were calculated for typical micropiles with assumed casing and socket dimensions and are based on values of bedrock strength interpreted from material descriptions included in the original 1954 foundation design report. At the detail design stage, bedrock samples should be recovered for UCS testing to provide the information required to carry out a complete micropile design.



As described in Section 6.4.2, vibration monitoring should be carried out during installation of the micropiles to ensure that the vibration levels at the existing structure are maintained below tolerable levels.

## **6.7 Approach Embankments**

### **6.7.1 Subgrade Preparation and Embankment Construction**

It is recommended that all topsoil/organic material or existing surficial fill present within the footprint of the widened embankments be stripped prior to placement of embankment fill. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the widened approach embankment areas.

The approach embankment fill for the bridge widening should be placed and compacted in accordance with MTO's Special Provision 206S03 (*Earth Excavation and Grading*) and 105S21 (*Amendment to OPSS 501 – Compacting*). Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials, in accordance with OPSS 208.010 (*Benching of Earth Slopes*).

To reduce erosion of the widened embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 572 (*Seed and Cover*).

### **6.7.2 Global Stability**

Preliminary slope stability analyses have been performed for the proposed embankment widening using the commercially available slope stability analysis software package SlopeW<sup>TM</sup> Version 7.17 by GeoSlope International Ltd., to verify that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses were carried out considering embankment side slopes of 2H:1V. The soil stratigraphy used in the analyses was selected to represent soil conditions with the greatest thickness of overburden soil that may be expected at the site and was based on the information available. Preliminary analyses were carried out considering both short-term (undrained) and long-term (effective stress) conditions. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations.

<b>Soil Conditions</b>	<b>Bulk Unit Weight (kN/m<sup>3</sup>)</b>	<b>Effective Friction Angle</b>	<b>Undrained Shear Strength (kPa)</b>
Hard silty clay (weathered crust)	18	29°	200
Very stiff to hard silty clay	18	29°	100
Compact to very dense lower sand and gravel	21	34°	-
Granitic bedrock	23	-	-



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It was assumed in the analyses that the topsoil encountered at the ground surface in the boreholes put down within the embankment footprints in 1954 boreholes was stripped prior to construction of the approach embankments.

The preliminary stability analysis results indicate that an approximately 8 m high embankment with side slopes oriented no steeper than 2H:1V will have a factor of safety of 1.3 or better against static global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

### 6.7.3 Settlement

Preliminary settlement analyses under the widened approach embankments were carried out using both hand calculations and the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli and/or consolidation parameters as given in the table below, based on correlations with the SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)
Embankment fill	21	-
Very stiff to hard silty clay	18	20
Compact to very dense lower sand and gravel	21	100

Based on this preliminary assessment, the settlement of the foundation soils under an 8 m high, 5 m wide embankment widening (to match the existing approach embankments) is estimated to be a maximum of about 25 mm, decreasing to less than 10 mm near the new embankment toe. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. However, this estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the widened approach embankments as determined during the detail design, with emphasis on the thickness and properties of the unweathered silty clay deposit within the embankment widening footprint.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 percent of the height of the embankment, assuming approximately 95 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.



## **6.8 Construction Considerations**

The following sections identify future construction issues that should be considered at this stage as they may impact the planning and preliminary design.

### **6.8.1 Vibration Monitoring During Pile Driving**

The proposed widening construction is anticipated to be carried out adjacent to, or from the existing bridge structure. It is recommended that vibration monitoring be carried out during pile or caisson installation to assist in maintaining vibration levels within tolerable ranges for the existing and new portions of the bridge.

As discussed in Section 6.4.2, a maximum peak particle velocity of 100 mm per second is recommended at the existing piers and abutments. The piles or caissons furthest from the existing structure should be driven/cored first, in order to check the vibration level at the existing structures and, if necessary, alter the installation procedures for the remaining piles or caissons.

### **6.8.2 Obstructions**

The soils at this site as well as the existing approach embankment fill may contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in the next stage of investigation in support of the detail design.

### **6.8.3 Erosion and Scour Protection**

The near-surface soils at the site are expected to be susceptible to erosion and scour under the design flood/flow velocities. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g., rip-rap or granular sheeting) be provided on the creek banks to protect the foundations/pile caps from being exposed. The rip-rap should be consistent with the standard R-10 classification or granular sheeting classification in accordance with OPSS 1004 (Aggregates) but should be approved by the hydraulic design engineer.

## **6.9 Recommendations for Further Work in Detail Design**

Additional boreholes will be required during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, depending on the selected construction alternative. If construction of new foundations at the abutments and piers is selected, additional boreholes will be required at the proposed abutment and pier foundation locations to allow for the following:

- Confirmation of the bedrock surface elevation within the proposed foundation area, and the thickness of any weathered, fractured or otherwise weakened zone at the top of the bedrock, to confirm the founding elevation for piles or caissons and the strength of the competent bedrock. If micropiles are adopted, strength testing on bedrock core samples will be required to allow for detailed micropile design.
- Further assessment of the groundwater level and permeability of the site soils to refine dewatering estimates.



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## **PRELIMINARY FOUNDATION DESIGN REPORT CATARAQUI RIVER BRIDGE, HIGHWAY 401, KINGSTON, ONTARIO**

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Regardless of the method selected for support of the widened bridge structure, additional boreholes will be required for the widened approach embankments to allow for the following:

- Assessment of the depth and extent of stripping of topsoil/organics and fill materials within the footprint of the approach embankments.
- Further assessment of the thickness and consolidation/elastic compression properties of any loose or firm/stiff surficial soils within the footprint of the approach embankments, to confirm the settlement estimates.



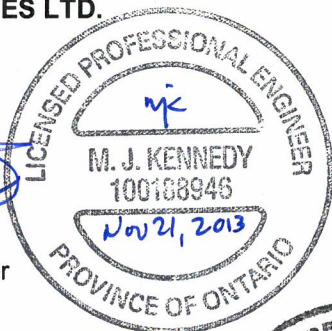
## PRELIMINARY FOUNDATION DESIGN REPORT CATARAQUI RIVER BRIDGE, HIGHWAY 401, KINGSTON, ONTARIO

### 7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.

#### GOLDER ASSOCIATES LTD.

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Geotechnical Engineer



Lisa Coyne, P.Eng.  
Senior Geotechnical Engineer, Principal

Fin Heffernan, P.Eng.  
Designated MTO Foundations Contact



MJK/LCC/FJH/lc/bg

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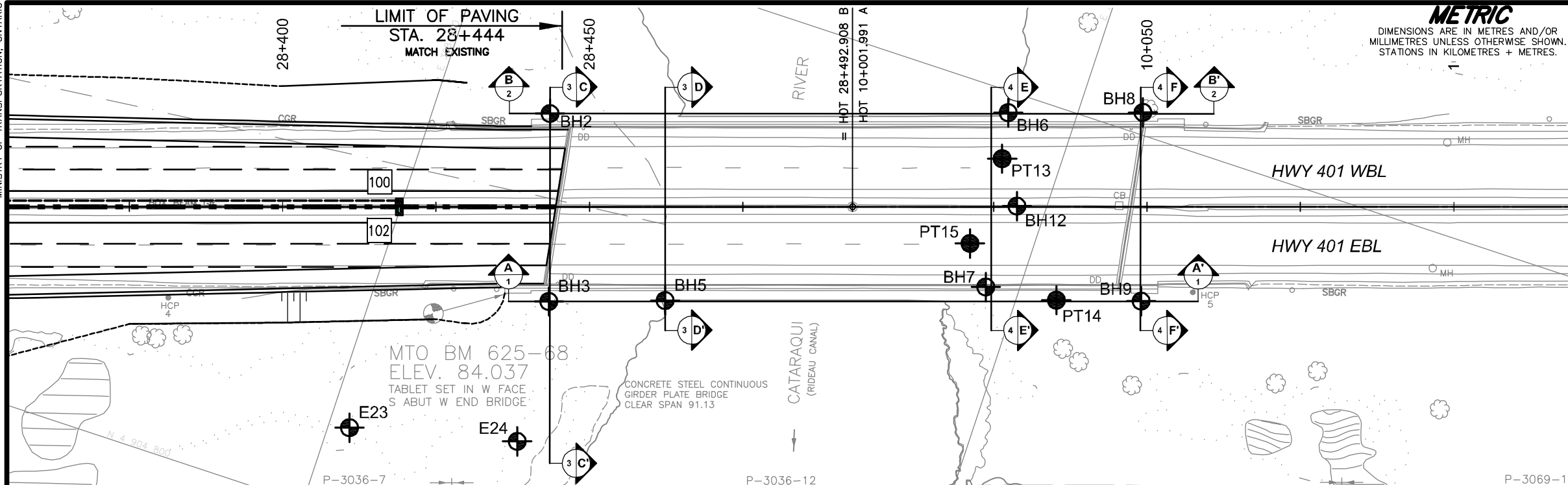


## PRELIMINARY FOUNDATION DESIGN REPORT CATARAQUI RIVER BRIDGE, HIGHWAY 401, KINGSTON, ONTARIO

**Table 1 – Comparison of Foundation Alternatives**

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Use of existing caisson foundations	<ul style="list-style-type: none"> <li>Feasible, preferred option</li> </ul>	<ul style="list-style-type: none"> <li>No considerable additional foundation construction required</li> </ul>	<ul style="list-style-type: none"> <li>Increased load on existing foundations</li> <li>Eccentric loading and uplift moments expected to require anchoring of existing caissons</li> </ul>	<ul style="list-style-type: none"> <li>Rock anchors or similar installed from bridge deck for uplift resistance</li> </ul>	<ul style="list-style-type: none"> <li>Lower cost</li> </ul>
New caissons	<ul style="list-style-type: none"> <li>Feasible</li> </ul>	<ul style="list-style-type: none"> <li>Construction from outside of travelled lanes (reduced impact on traffic)</li> <li>Structurally similar to existing foundations</li> </ul>	<ul style="list-style-type: none"> <li>May require construction of pier foundations from within the river</li> </ul>	<ul style="list-style-type: none"> <li>Rock socketing into granite bedrock may be required</li> <li>Pier foundation construction from floating equipment or (possibly) from constructed pad on shore</li> </ul>	<ul style="list-style-type: none"> <li>Moderate cost</li> </ul>
New H-piles/pipe piles	<ul style="list-style-type: none"> <li>Feasible, but would likely require rock socketing in some areas</li> </ul>	<ul style="list-style-type: none"> <li>Construction from outside of travelled lanes (reduced impact on traffic)</li> </ul>	<ul style="list-style-type: none"> <li>Shallow bedrock depth in some locations would require pre-drilling of bedrock to achieve minimum pile length</li> <li>May require construction of pier foundations from within the river</li> </ul>	<ul style="list-style-type: none"> <li>Rock socketing into granite bedrock may be required at some locations</li> <li>Pier foundation construction from floating equipment or (possibly) from constructed pad on shore</li> </ul>	<ul style="list-style-type: none"> <li>Moderate cost</li> </ul>
Drilled Micropiles	<ul style="list-style-type: none"> <li>Feasible, preferred option</li> </ul>	<ul style="list-style-type: none"> <li>Reduced vibration during construction, minimizes potential impact on existing bridge foundations</li> <li>Smaller equipment footprint,</li> <li>Requires smaller diameter socketing into strong to very strong bedrock</li> </ul>	<ul style="list-style-type: none"> <li>May require construction of pier foundations from within the river</li> <li>Requires specialist contractor</li> </ul>	<ul style="list-style-type: none"> <li>Pier foundation construction from floating equipment or (possibly) from constructed pad on shore</li> </ul>	<ul style="list-style-type: none"> <li>Moderate to high cost</li> </ul>



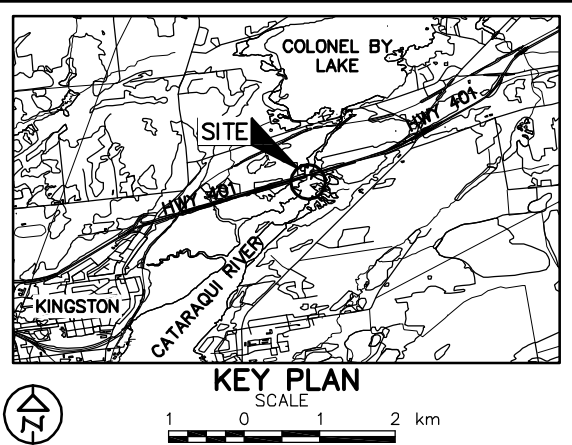


CONT No.  
GWP No. 80-99-01

SHEET

CATARAQUI RIVER BRIDGE  
SITE 7-70  
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.  
OTTAWA, ONTARIO, CANADA



**LEGEND**

- Borehole - Previous Investigation
- Penetration Test - Previous Investigation
- Seal
- Piezometer
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of or during drilling

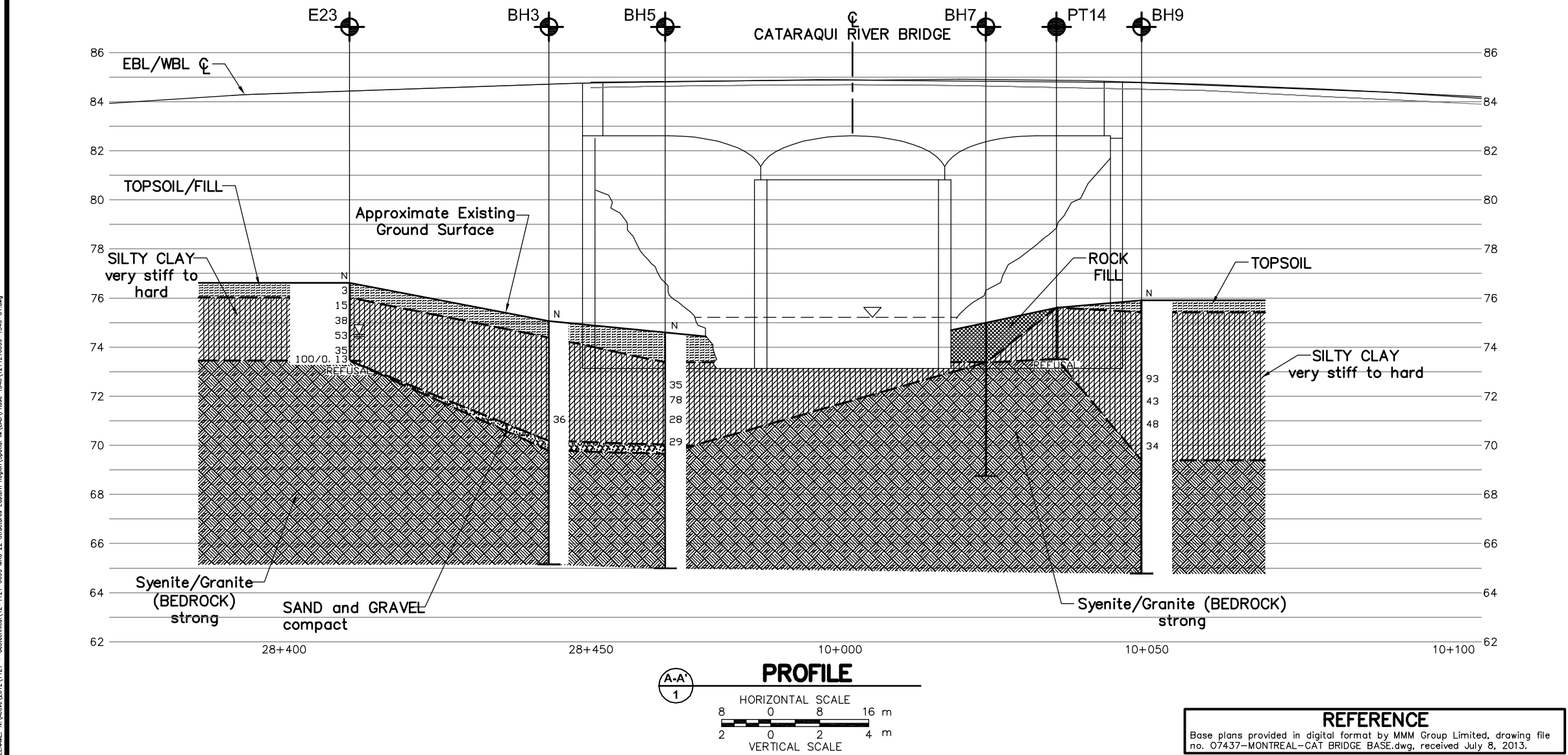
No.	ELEVATION	NORTHING	EASTING
BH2	76.1	4904873.0	308918.3
BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904870.0	309006.3
PT14	75.6	4904874.4	308990.0
PT15	75.1	4904889.2	308990.6
E23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1

**NOTES**

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

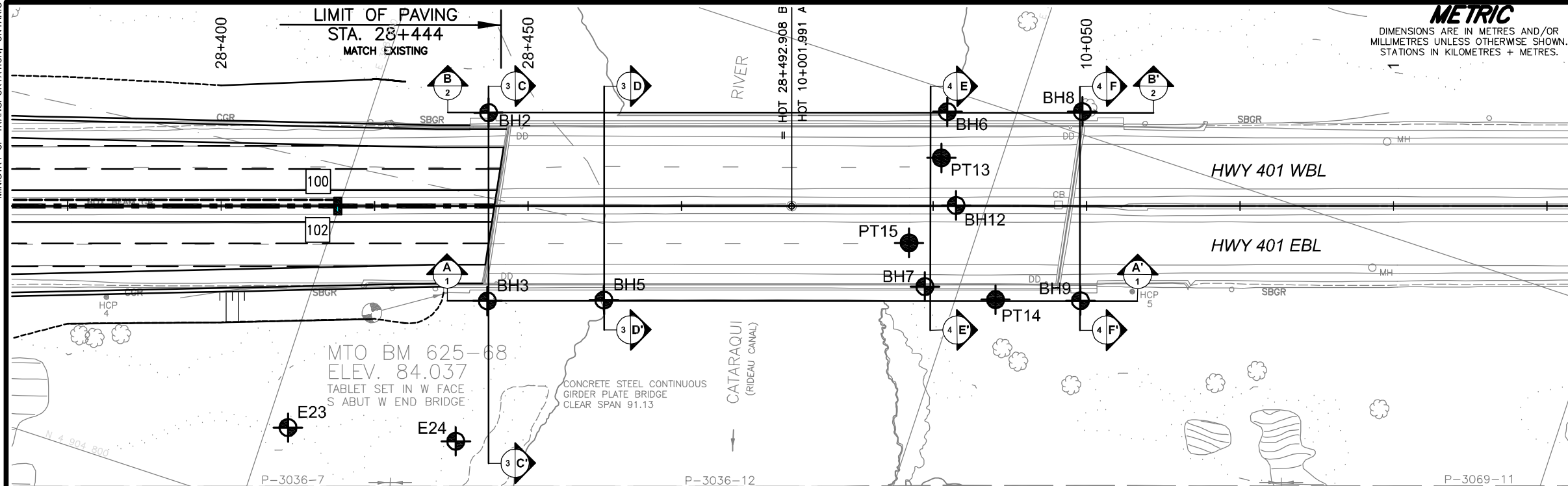


**REFERENCE**

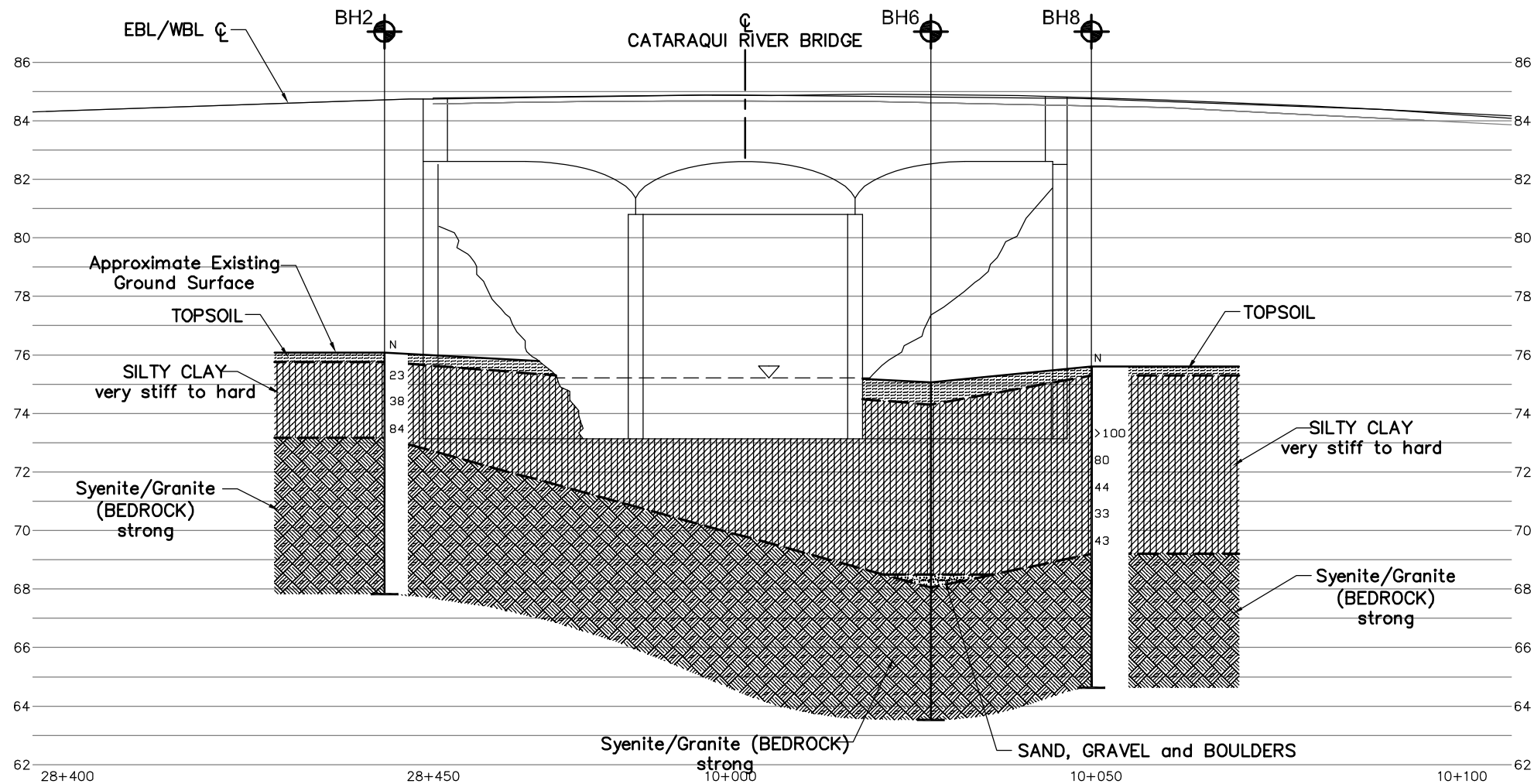
Base plans provided in digital format by MMM Group Limited, drawing file no. 07437-MONTREAL-CAT BRIDGE BASE.dwg, received July 8, 2013.

NO.	DATE	BY	REVISION
Geocres No. 31C-215			
HWY. 401			PROJECT NO. 12-1121-0099-1340 DIST. Eastern
SUBM'D. MJK	CHKD. FJH	DATE: Nov. 2013	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG.1





**PLAN**  
SCALE  
8 0 8 16 m



**PROFILE**  
HORIZONTAL SCALE  
8 0 8 16 m  
VERTICAL SCALE  
2 0 2 4 m

CONT No.  
GWP No. 80-99-01

CATARAQUI RIVER BRIDGE  
SITE 7-70  
BOREHOLE LOCATIONS AND SOIL STRATA

**Golder Associates Ltd.**  
OTTAWA, ONTARIO, CANADA

**KEY PLAN**  
SCALE  
1 0 1 2 km

**LEGEND**

Borehole - Previous Investigation

Penetration Test - Previous Investigation

Seal

Piezometer

Standard Penetration Test Value

Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)

No.	ELEVATION	NORTHING	EASTING
BH2	76.1	4904873.0	308918.3
BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904870.0	309006.3
PT14	75.6	4904874.4	308990.0
PT15	75.1	4904889.2	308990.6
E23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1

**NOTES**

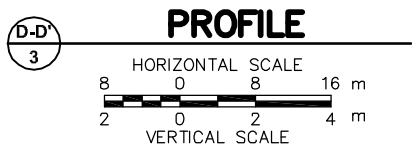
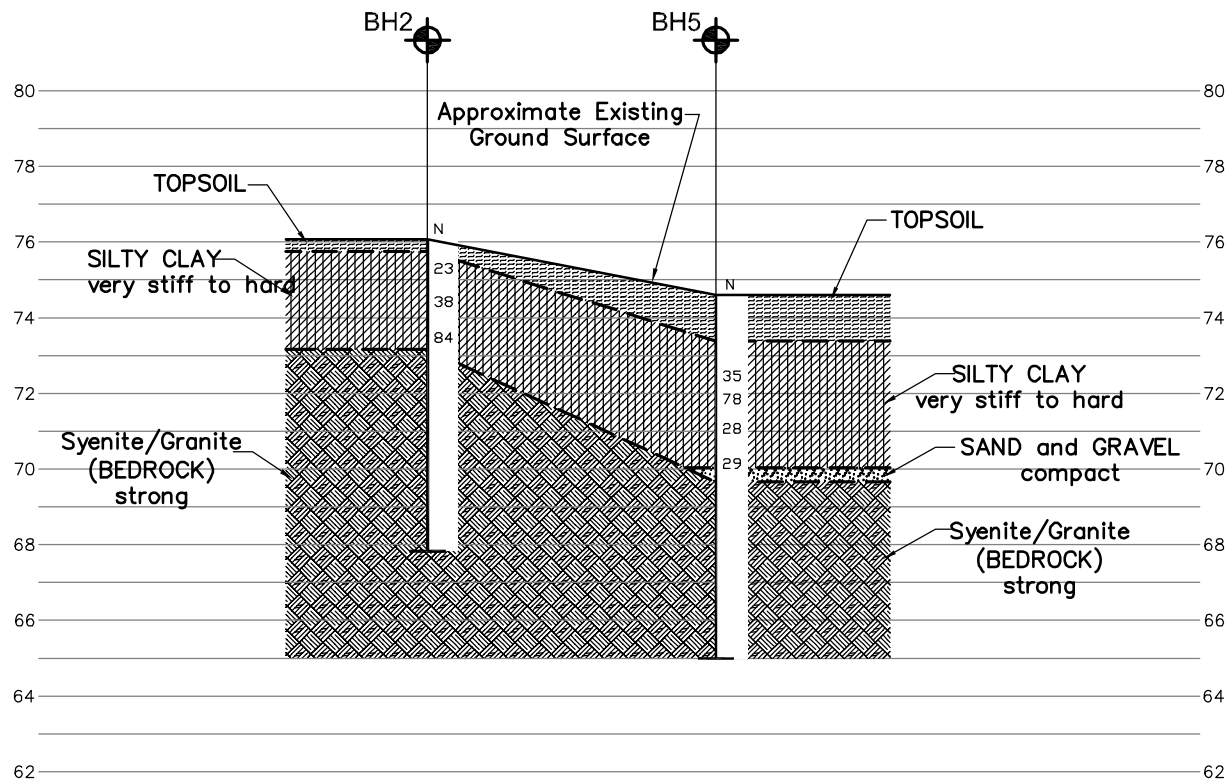
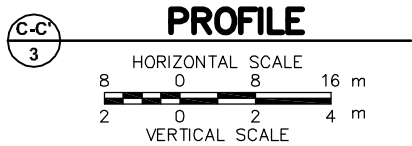
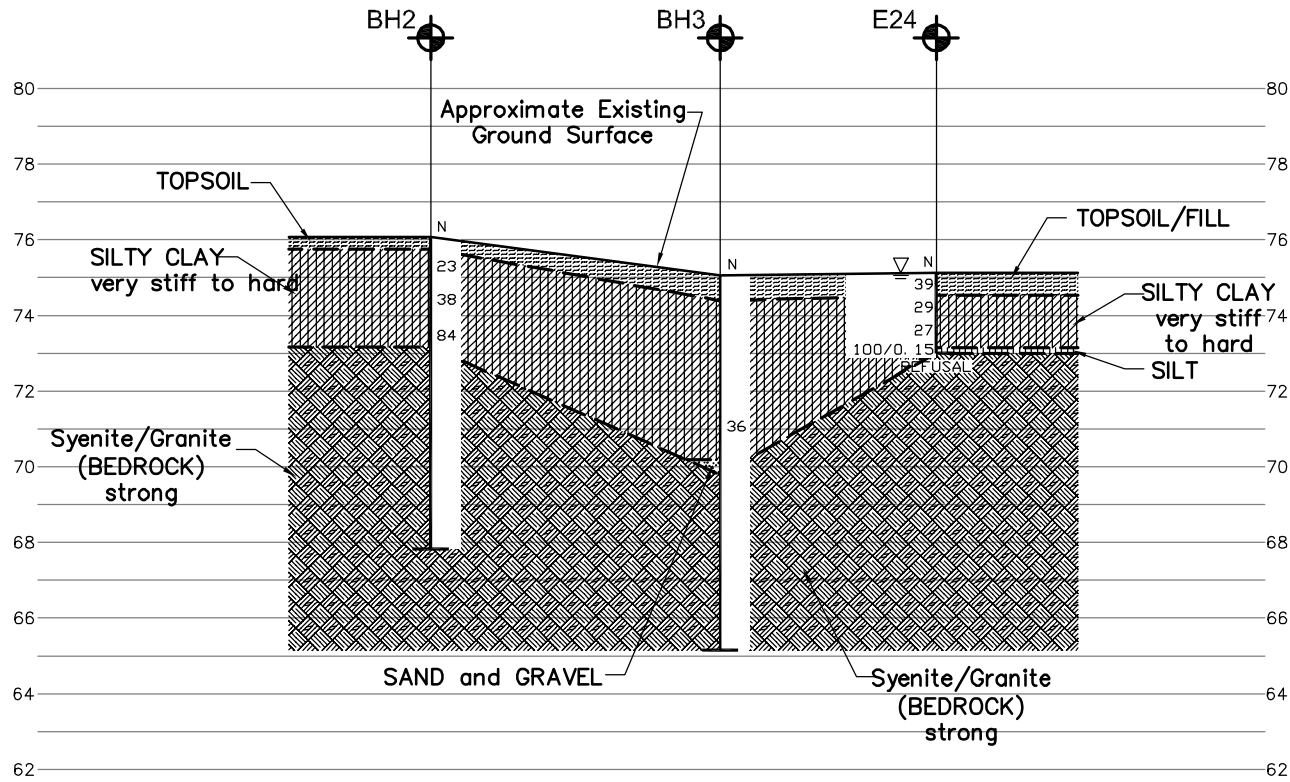
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

NO.	DATE	BY	REVISION
Geocres No. 31C-215			
HWY. 401		PROJECT NO. 12-1121-0099-1340 DIST. Eastern	
SUBM'D. MJK	CHKD. FJH	DATE: Nov. 2013	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 2

**REFERENCE**  
Base plans provided in digital format by MMM Group Limited, drawing file no. 07437-MONTREAL-CAT BRIDGE BASE.dwg, received July 8, 2013.



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

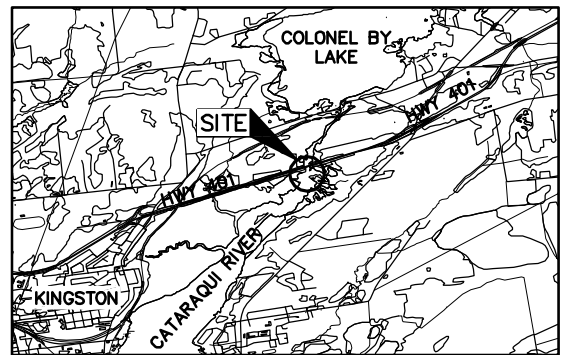
CONT No.  
GWP No. 80-99-01

CATARAQUI RIVER BRIDGE  
SITE 7-70  
SOIL STRATA

SHEET



**Golder Associates Ltd.**  
OTTAWA, ONTARIO, CANADA



KEY PLAN  
SCALE  
1 0 1 2 km

#### LEGEND

- Borehole - Previous Investigation
- Penetration Test - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)
- WL upon completion of or during drilling

No.	ELEVATION	NORTHING	EASTING
BH2	76.1	4904873.0	308918.3
BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904870.0	309006.3
PT14	75.6	4904874.4	308990.0
PT15	75.1	4904889.2	308990.6
E23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1

#### NOTES

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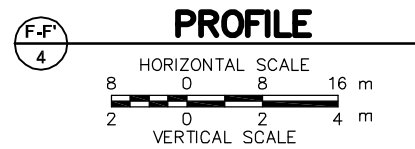
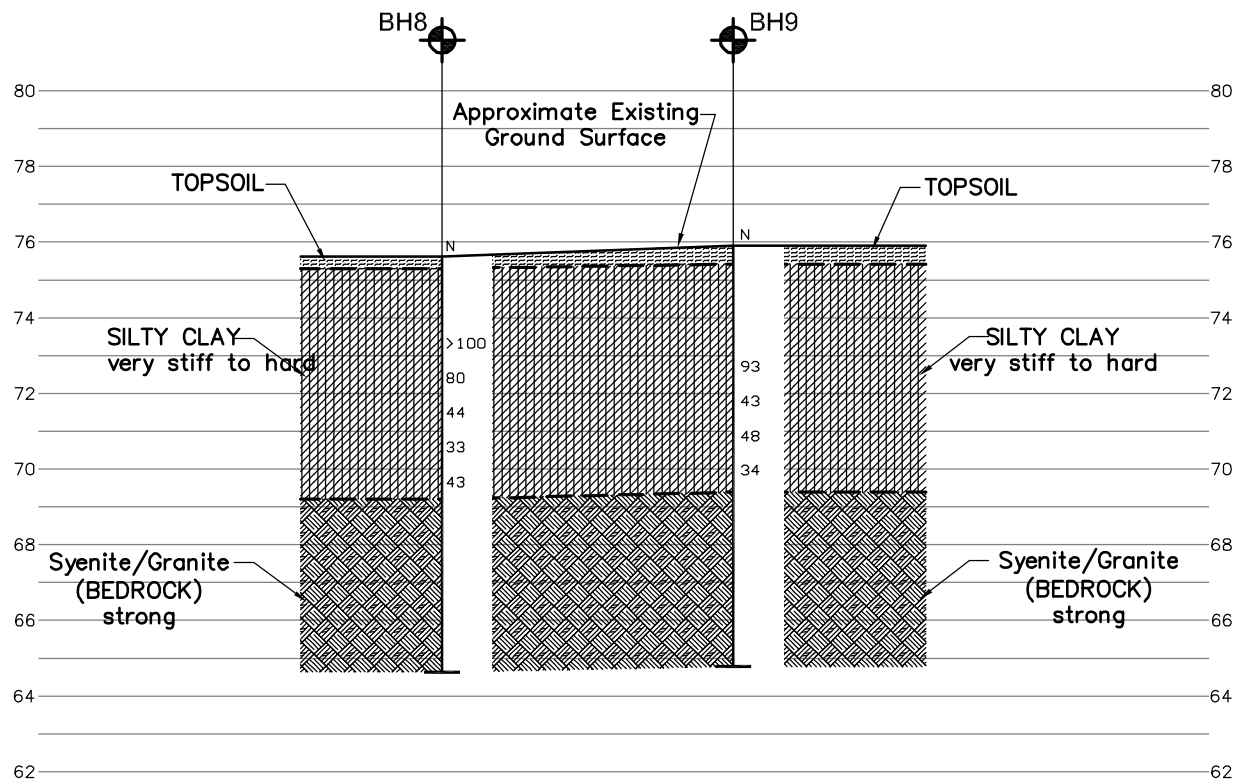
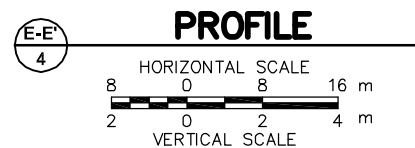
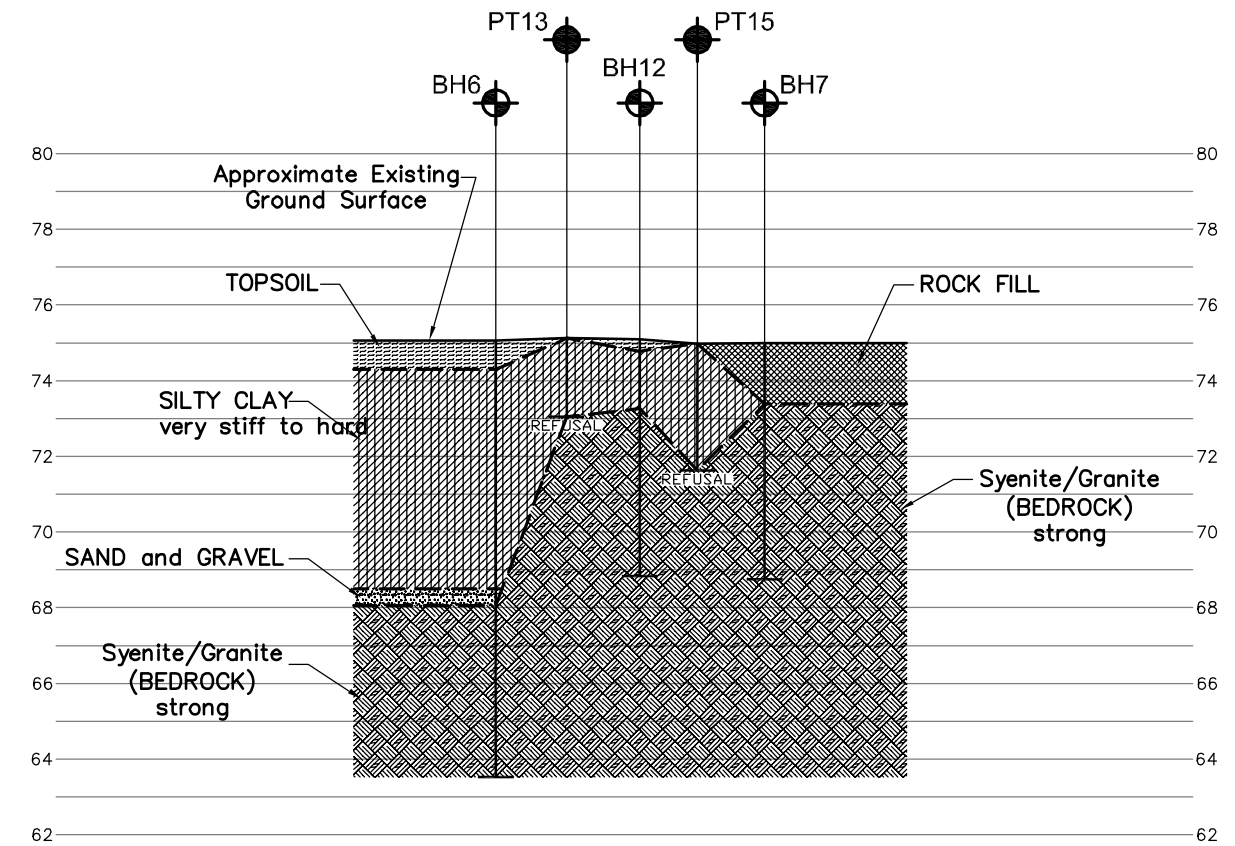
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

NO.	DATE	BY	REVISION
Geocres No. 31C-215			
HWY. 401	PROJECT NO. 12-1121-0099-1340		
SUBM'D. MJK	CHKD. FJH	DATE: Nov. 2013	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 3

#### REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. 07437-MONTREAL-CAT BRIDGE BASE.dwg, received July 8, 2013.



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

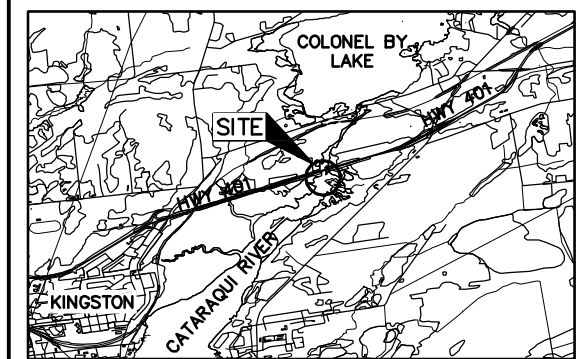
CONT No.  
GWP No. 80-99-01

CATARAQUI RIVER BRIDGE  
SITE 7-70  
SOIL STRATA

SHEET



**Golder Associates Ltd.**  
OTTAWA, ONTARIO, CANADA



**KEY PLAN**  
SCALE  
1 0 1 2 km

**LEGEND**

- Borehole - Previous Investigation
- Penetration Test - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)

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BH3	75.1	4904843.8	308927.8
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PT13	75.1	4904870.0	309006.3
PT14	75.6	4904874.4	308990.0
PT15	75.1	4904889.2	308990.6
E23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1

**NOTES**

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

NO.	DATE	BY	REVISION
Geocres No. 31C-215			
HWY. 401	PROJECT NO. 12-1121-0099-1340		
SUBM'D. MJK	CHKD. FJH	DATE: Nov. 2013	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 4

**REFERENCE**

Base plans provided in digital format by MMM Group Limited, drawing file no. 07437-MONTREAL-CAT BRIDGE BASE.dwg, received July 8, 2013.



# **APPENDIX A**

**Previous Foundation Investigation Report  
(Foundation of Canada Engineering Corp. Ltd., 1954)**

MONTREAL OFFICE  
1830 SHERBROOKE ST. W.  
MONTREAL, 25

FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED  
270 BLOOR STREET EAST  
TORONTO  
April 28, 1954.

BA 354  
CABLE ADDRESS  
"FOUNDAMENG"  
TORONTO  
54-F-201c

Ontario Department of Highways,  
Attention Mr. H. N. Lamont,  
Chief Bridge Engineer,  
Room 1422,  
Parliament Buildings,  
Toronto, Ontario.

SITE INVESTIGATION  
CATARAQUI RIVER BRIDGE

Gentlemen:

This letter accompanies our factual  
report covering the investigation carried out at  
the above site.

Yours very truly,  
FOUNDATION OF CANADA  
ENGINEERING CORPORATION LIMITED

  
R. W. Crudge, P. Eng.,  
ASSISTANT TO THE VICE-PRESIDENT.

RWC:MP  
Encl.  
1020.

FENCO

1020  
REPORT  
TO  
ONTARIO DEPARTMENT OF HIGHWAYS  
ON  
SOIL CONDITIONS  
CATARAQUI RIVER BRIDGE  
KINGSTON, ONTARIO

Distribution -

- 2 copies - Ontario Department of Highways, Toronto, Ontario.
- 2 copies - Foundation of Canada Engineering Corporation,  
Limited.
- 2 copies - Geotechnical Services.

April 21, 1954.  
**THE FOUNDATION** COMPANIES  
CANADA

## INDEX

	Page
Introduction	1
The Site	1
Procedure	1
Soil Conditions	2
Water Conditions	3
Personnel	3

Appendix I - Office Reports on Soil Exploration

Drawing in Pocket at rear.

1020-C-2 Plot Plan & Soil Stratigraphy.



## INTRODUCTION

The Foundation of Canada Engineering Corporation Limited has been retained by the Ontario Department of Highways (proposal dated January 11, 1954 and accepted January 14, 1954) to carry out a soil investigation at the site of the proposed bridge on the limited access highway crossing the Cataraqui River, near Kingston Mills, Ontario.

The object of the investigation is to examine the soil and rock conditions at the locations of the bridge piers and abutments and to investigate the stability of the approach embankments.

This work has now been completed and is here reported in detail.

## THE SITE

The proposed bridge crossing is located at a very narrow section of the Cataraqui River which is also a part of the Rideau Canal. It is located about one-half mile below Kingston Mills.

At this site, the canal passes over Precambrian rock of the Frontenac Axis. Broken rock from this formation is seen on both banks of the canal at the site. Apparently this has been dredged from the channel which is a portion of the original canal constructed in 1826. No construction records are available to show either the original rock surface or the final rock surface after excavation.

The bed rock is overlain by a lacustrine clay deposit of glacial origin.

## PROCEDURE

The field work was begun on January 25, 1954 and was completed on February 11, 1954. Eight exploratory borings with penetration tests and three separate penetration tests were made using a standard machine rig.

In addition, seven jet probings were made along the approximate line of each of the two piers which will be located in the water.

Drawing No. 1020-C-2 (in pocket at rear) shows the boring locations and the inferred soil stratigraphy. The elevations of the borings were determined by the Ontario Department of Highways. The Office Reports on Soil Exploration in Appendix I give the detailed log of each borehole and the laboratory test results.



PROCEDURE (Continued)

Laboratory tests have been carried out in the Geocon Ltd. laboratory and the remaining samples will be retained there for destruction after June 1st, 1954 unless other instructions are received.

SOIL CONDITIONS

The following are the principal soil strata which have been encountered:

Rock Fill

Rock Fill which has apparently been excavated to form the channel was encountered in Borehole No. 7. Borehole No. 5 is located just to one side of a ridge which is seen from the surface to be composed of rock fill.

Topsoil

One to two feet of clayey topsoil was encountered at most boreholes.

Soft Organic Clay

In Borehole No. 5 and in most of the jet probings some soft organic clay was encountered forming the river bottom. This is quite soft and contains a considerable amount of partially decomposed organic material.

Very Stiff Mottled Clay

The topsoil at most boreholes is underlain by 3 to 10 feet of very stiff mottled grey and brown clay. The unconfined compressive strength of this deposit is about 2 to 3 tons per square foot with a low sensitivity. The natural moisture content is about 30% and the liquid and plastic limits respectively about 40% and 20%.

The structure of the clay is generally somewhat chunky and crumbly. This together with the color and strength show that it has been preconsolidated by desiccation. It also contains some irregular but generally horizontal layers of silt or silty clay, at a frequency of one every 1/2 to 3 inches.

The clay contains a small amount of sand and small gravel.

Firm Varved Clay

The transition from the mottled to varved clay is a gradual one.

The varved clay contains alternating grey and dark grey layers. The thickness of a pair of layers is generally 1/4 to 1/2

### Firm Varved Clay (Continued)

inch. The unconfined compressive strength is generally in the range 1/2 to 1-1/2 tons per square foot with a low to moderate sensitivity. The natural moisture content is about 35% with liquid and plastic limits respectively of about 50% and 25%.

### Sand and Gravel

In several locations a small quantity - up to 2 feet - of sand and gravel was encountered just overlying bedrock. This sand and gravel is generally of low permeability containing enough silt and clay to fill most of the interstices.

### Bedrock

Bedrock was encountered at elevations varying between 223 and 240.

The bedrock is consistently fresh, sound and massive Precambrian igneous rock of the Frontenac Axis. It is composed of hornblende syenite or syenite gneiss except in Borehole Nos. 5 and 7 where there occurs some pink fine to coarse grained granite with a small hornblende content.

Generally excellent core was obtained of the rock. Some joints were noted, but they were consistently cemented with a chlorite deposit.

### WATER CONDITIONS

Due to the impermeable nature of the overburden, it is not believed that the ground water conditions at the site will cause any unusual difficulties during the proposed construction. Piezometers and slotted pipes, for long-time ground water table observations, were therefore omitted. It is believed that the permanent ground water table will be found near the average elevation of the water in the nearby Cataraqui River.

### PERSONNEL

The site exploration work has been carried out by the Geotechnical Services Division of Gecon Ltd. Field work was supervised by B.D. Benedict with driller J. Johnson. The report was written by B.D. Benedict and N.D. Lea.

BDB/jb

*B.D. Benedict*  
B.D. Benedict,  
P.Eng.

APPENDIX I  
OFFICE REPORTS ON SOIL EXPLORATION

## **EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"**

It is the practice of The Foundation Company of Canada Limited to report boring results on Form C-149, entitled "Office Report on Soil Exploration". The object of this form of record is to combine all of the information which is gathered through the boring on one sheet of paper so that it will be possible, by reference to this sheet, to make a careful and complete study of the soil as disclosed by that particular boring. An explanation of the various columns of the report follows. They will be referred to as Columns 1 to 12. Note that Columns 1 to 4 inclusive are entitled "Soil Profile", Column 5 is entitled "Elevation Scale", Column 6 is the main body of the form in which results are presented in graphical form and Columns 7 to 12 have the caption "Samples".

### **COLUMN 1 — ELEVATION AND DEPTH OF BOUNDARIES**

In this column are shown the elevation and the depth of important boundaries between the various soil strata. The elevation is shown above the line and the depth below the floor of the drilling platform is shown below the line. The datum to which the elevation refers is shown in the general heading of the form.

### **COLUMN 2 — WATER CONDITIONS**

In this column the water level or water table is indicated to scale by a horizontal line in the appropriate location with the symbol W.L. or W.T. above the line. A distinction is made between water level and water table. The water level merely refers to the level of the water standing in the boring or standpipe. If there is no indication of the date on the form, then it will be the date on which the boring is made. The depth of the porous stone, or perforated section of the observation well, or the depth of the boring and casing at the time of water level observation, whichever applies, is also shown. The water table refers to what is believed to be the true water table in the soil on the date shown, based on results of a series of water level observations. Where there are complicated groundwater conditions, some notation on this will also be made in this Water Conditions column.

### **COLUMN 3 — DESCRIPTION**

In the third column is shown a description of the soil. This description follows the standard terminology laid down in The Foundation Company of Canada Limited Circular No. 1102. Some of the most significant points of this terminology are as follows: "Clay" is always used to refer to material which displays an appreciable amount of plasticity. "Silt" is used to characterize a material finer than fine sand which shows very little or no plasticity but which shows a rapid response to the shaking test. "Till" is used to refer to a material which includes all grain sizes from clay size to boulder size. The term "Clay Till" is used when a till has considerable cohesion and "Sandy Till" or "Silty Till" when the till as a mass displays the properties of a sand or silt. Colours are referred to the colour standards of the U.S. Department of Agriculture.



Terms describing the consistency of cohesive soils are related to unconfined compressive strength in tons per square foot as follows:

Semi-liquid	— below 0.03
Very soft	— 0.03 to 0.25
Soft	— 0.25 to 0.5
Firm	— 0.5 to 1.0
Stiff	— 1.0 to 2.0
Very stiff	— 2.0 to 4.0
Hard	— over 4.0

Terms describing the relative density of non-cohesive soils are related to the penetration resistance of the 2'' drive sampler as follows:

Very loose	— 0 to 4
Loose	— 4 to 10
Compact	— 10 to 30
Dense	— 30 to 50
Very dense	— over 50

#### **COLUMN 4 — STRATIGRAPHIC PLOT**

The stratigraphic plot, Column 4, follows the standard symbols of the National Research Council, Canada.

#### **COLUMN 5 — ELEVATION SCALE**

The information in all columns is plotted to a true scale of elevation which is shown in this Column (No. 5). It is usually made so that one small square of Column 6 represents either a foot or half a foot.

#### **COLUMN 6 — GRAPHS**

The main body of the report forms a graph which is used to plot to correct depth scale the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

The first scale shown there is not filled in on the printed form but is usually used for the strength of the soil as determined either through unconfined compression, vane, or other strength tests.

The next scale is water content which is given in percent by weight of the dry solids in the soil. Using different symbols, there is shown on the graph the natural moisture content by an open circle, the liquid limit by an open square and the plastic limit by an open triangle. A line is drawn between the liquid and the plastic limits, as the length of this line represents the plasticity index.

The lowest scale is the penetration test resistance in blows-per-foot. The penetration test is performed using a conical point of 2'' diameter with a sixty degree cone. The point

is fastened to a diamond-drill rod and driven into the ground several feet from the location of the boring. The blows-per-foot are recorded and each record is shown on the plot by a cross. A dotted line is drawn between these crosses. The hammer used for this test may weigh either 50, 140, 380 or 520 pounds. Nevertheless, whatever the weight of the hammer, the number of blows per foot is converted to an equivalent number of blows of 4200 inch-pounds of energy. It is this equivalent number of blows which is plotted on the graph. It has been our experience that, except for soft or very loose soils, the weight of the hammer used influences the results obtained only to a minor extent.

#### **COLUMN 7 — OTHER TESTS**

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted graphically in Column 6. The symbols used for this column are shown at the head of the form under the abbreviations.

#### **COLUMNS 8, 9 AND 10 — SAMPLES**

These columns describe the samples obtained from the boring. Column 10 gives the number of the sample, Column 9 the type and Column 8 the condition. The meaning of the symbols used for type and condition is shown at the head of the report. The symbols for condition of sample are made to correspond in a vertical scale with the location of the sample.

#### **COLUMN 11 — PENETRATION RESISTANCE**

In Column 11 is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. These blows-per-foot have been converted in the same way as those of the penetration test.

In the field, the penetration resistance is recorded for every 6 inches on the 2-inch Drive Sampler. The figure which is recorded in Column 11 is usually the sum of the resistances for driving the second and third increments of the sampler into the ground. For the thin-walled sampler, the penetration resistance is recorded in the field for each foot. Thus a single figure shown in Column 11 is usually an average.

When greatly different results are obtained for driving different increments of the same sample, then two or more figures are shown in Column 11. Such a series of figures does not necessarily represent the blows for successive feet but it does indicate the variation in penetration resistance measured as a rate of number of blows per foot.

The symbol "R" for refusal means that under 20 or more blows the advance was less than  $\frac{1}{8}$  inch.

#### **COLUMN 12 — ELEVATION AND RECOVERY**

In this column is shown for each sample the elevation of the top of the sample and the percentage recovery.



## OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG MACHINIC JOB 1020 BORING # 2  
 CASING 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT MAR. 4/54  
 SAMPLER HAMMER, WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY M.A. & M. BORING DATE FEB 9/54

## SAMPLE CONDITION



DISTURBED  
FAIR  
GOOD  
LOST

## SAMPLE TYPES

C.S. - CHUNK  
D.O. - DRIVE-OPEN  
D.F. - DRIVE-FOOT VALVE  
D.P. - DRIVE PISTON  
T.O. - THIN WALLED OPEN  
T.P. - THIN WALLED PISTON

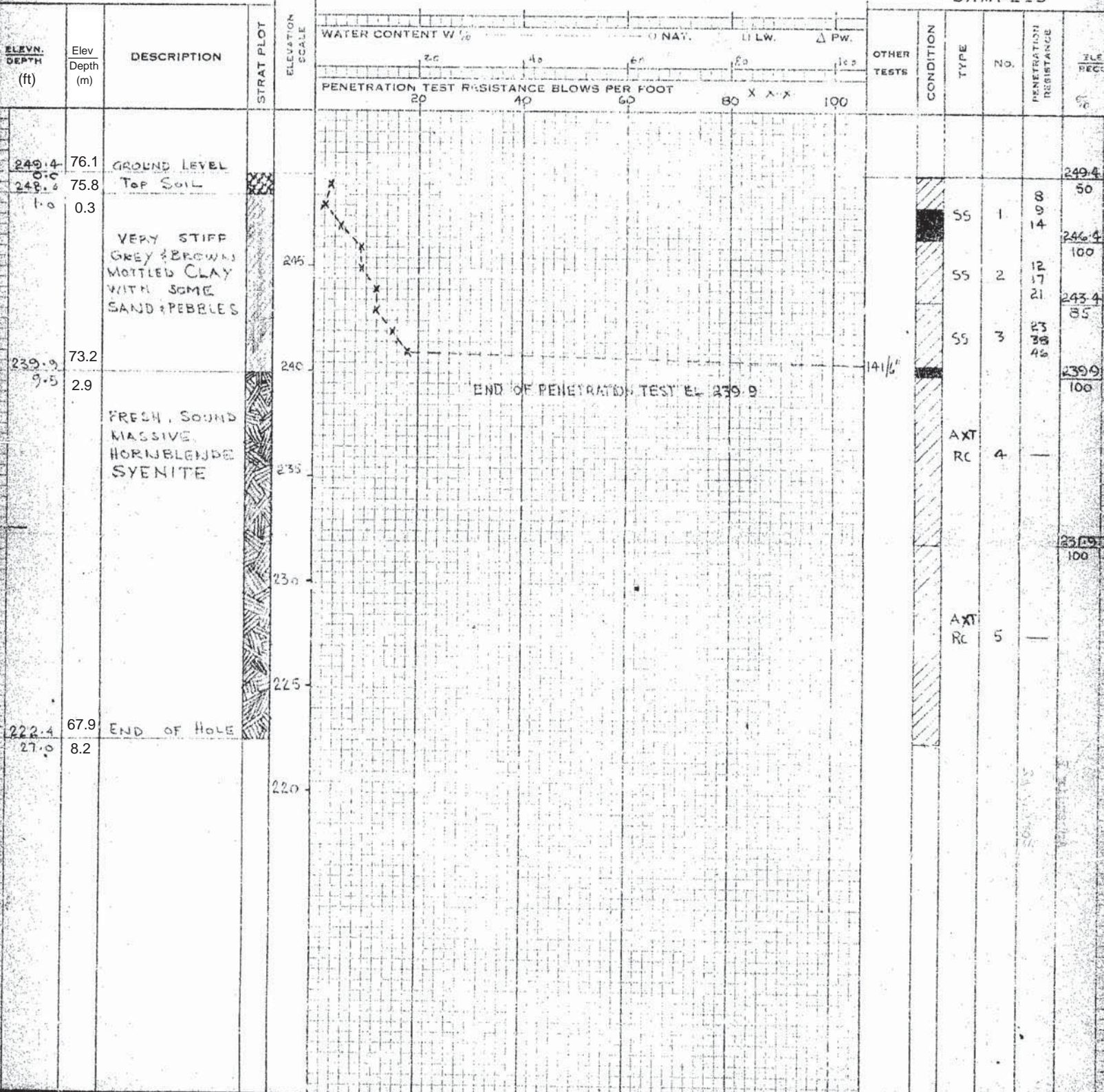
F.S. - FOIL SAMPLE  
B.A. - BARREL AUGER  
S.A. - SPIRAL AUGER  
W.S. - WASHED SAMPLE  
R.C. - ROCK CORE  
S.S. - SLEEVE SAMPLE

## ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST  
M. - MECHANICAL ANALYSIS  
U. - UNCONFINED COMPRESSION  
QC. - TRIAXIAL CONSOLIDATED QUICK  
Q. - TRIAXIAL QUICK  
S. - TRIAXIAL SLOW  
γ. - UNIT WEIGHT  
K. - PERMEABILITY  
C. - CONSOLIDATION  
CA. - CASING  
WL. - WATER LEVEL IN CASING  
WT. - WATER TABLE IN SOIL

## SOIL PROFILE

## SAMPLES





**OFFICE REPORT ON SOIL EXPLORATION**
**APPENDIX I**

DRILL RIG. MACHINE JOB 1020 BORING 3  
 CASING 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT MARCH 4/54  
 SAMPLER HAMMER WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY M.A.M. BORING DATE FEB 8/54

**SAMPLE CONDITION**

 DISTURBED  
 FAIR  
 GOOD  
 LOST

**SAMPLE TYPES**

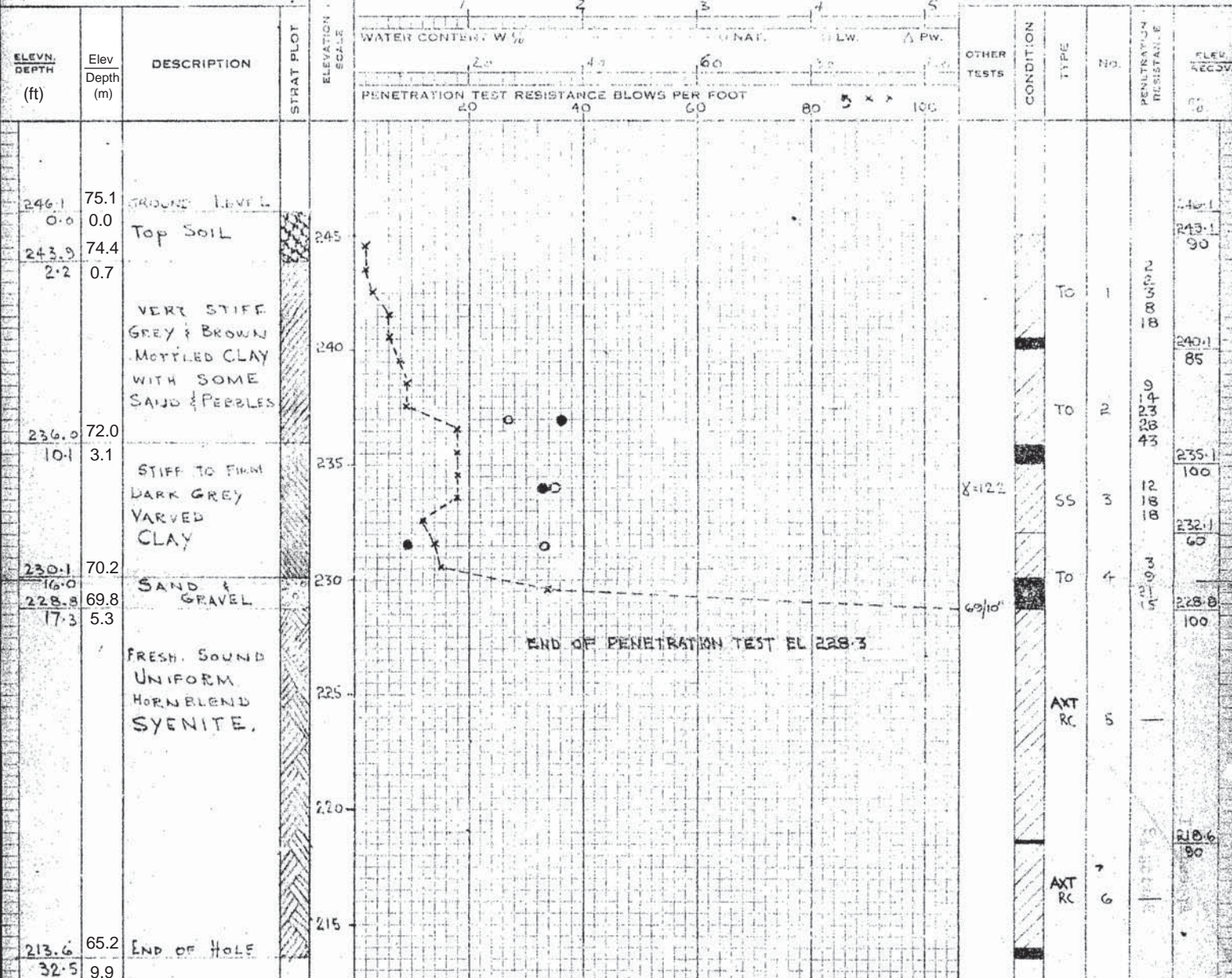
C.S. - CHUNK  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 D.P. - DRIVE PISTON  
 T.O. - THIN WALLED OPEN  
 T.P. - THIN WALLED PISTON  
 F.S. - FOIL SAMPLE  
 B.A. - BARREL AUGER  
 S.A. - SPIRAL AUGER  
 W.S. - WASHED SAMPLE  
 H.C. - ROCK CORE  
 S.S. - SLEEVE SAMPLE

**ABBREVIATIONS**

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 Y. - UNIT WEIGHT  
 K. - PERMEABILITY  
 C. - CONSOLIDATION  
 CA. - CASING  
 WL. - WATER LEVEL IN CASING  
 WT. - WATER TABLE IN SOIL

**SOIL PROFILE**
**UNCONFINED COMPRESSIVE STRENGTH**

Ton./Sq. Ft.

**SAMPLES**




## OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG: MACHINE

JOB: 1020

BORING # 5

CASING: 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED)

DATUM: GEODETIC

DATE REPORT: MAR 4/54

SAMPLER HAMMER WT. 430 DROP 15 INCHES

COMPILED BY: JA CHECKED BY: M.J.M. BORING DATE: FEB 5/54

## SAMPLE CONDITION



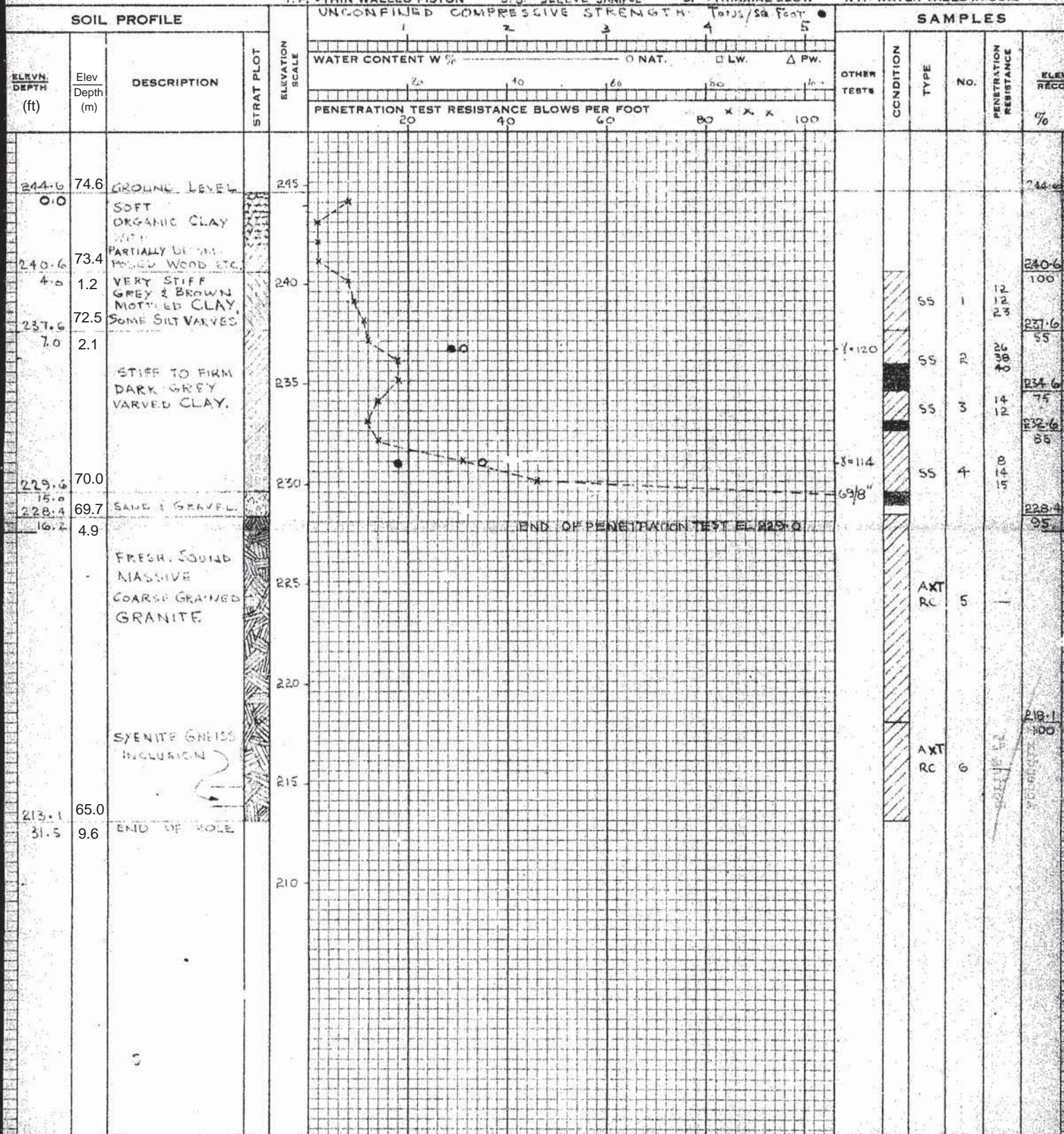
DISTURBED  
FAIR  
GOOD  
LOST

## SAMPLE TYPES

C.S. - CHUNK  
D.O. - DRIVE-OPEN  
D.F. - DRIVE-FOOT VALVE  
D.P. - DRIVE PISTON  
T.O. - THIN WALLED OPEN  
T.P. - THIN WALLED PISTON  
F.S. - FOIL SAMPLE  
B.A. - BARREL AUGER  
S.A. - SPIRAL AUGER  
W.S. - WASHED SAMPLE  
R.C. - ROCK CORE  
S.S. - SLEEVE SAMPLE

## ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST  
M. - MECHANICAL ANALYSIS  
U. - UNCONFINED COMPRESSION  
QC. - TRIAXIAL CONSOLIDATED QUICK  
Q. - TRIAXIAL QUICK  
S. - TRIAXIAL SLOW  
Y. - UNIT WEIGHT  
K. - PERMEABILITY  
C. - CONSOLIDATION  
CA. - CASING  
WL. - WATER LEVEL IN CASING  
WT. - WATER TABLE IN SOIL





## OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. MACHINE

JOB \_\_\_\_\_ 1020

APPEX I  
BORING # 6

**CASING** 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED)

DATUM 56045710

DATE REPORT March 4, 54

SAMPLER HAMMER, WT. 430 DROP. 15 INCHES

COMPILED BY JA CHECKED BY MAM BORING DATE JAN 25 21 54

BORING DATE JAN 25-27-54

### SAMPLE CONDITION



DISTURBED  
 FAIR  
 GOOD  
 LOST

## SAMPLE TYPES

C. S. - CHUNK  
D. O. - DRIVE-OPEN  
D. F. - DRIVE-FOOT VALVE  
D. P. - DRIVE PISTON  
T. O. - THIN WALLED OPEN  
T. P. - THIN WALLED PISTON

F. S. - FOIL SAMPLE  
B. A. - BARREL AUGER  
S. A. - SPIRAL AUGER  
W. S. - WASHED SAMPLE  
R. C. - ROCK CORE

## ABBREVIATIONS

V. -IN-SITU VANE SHEAR TEST      7. -UNIT WEIGHT WET  
 M. -MECHANICAL ANALYSIS      K. -PERMEABILITY  
 U. -UNCONFINED COMPRESSION      C.-CONSOLIDATION  
 QC. -TRIAxIAL CONSOLIDATED QUICK CA.-CASING  
 Q. -TRIAxIAL QUICK      WL.-WATER LEVEL IN CASING  
 S. -TRIAxIAL SLOW      WT.-WATER TABLE IN SOIL

[illegible]







# OFFICE REPORT ON SOIL EXPLORATION

APPEX I 8  
BORING!

DRILL RIG. MACHINE  
CASING 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED)  
SAMPLER HAMMER. WT. 430 DROP 15 INCHES

JOB 1020 BORING # 8  
DATUM GEODETIC DATE REPORT MAR 4/54  
COMPILED BY JA CHECKED BY MAJAL BORING DATE FEB 3/54

### SAMPLE CONDITION

DISTURBED  
FAIR  
GOOD  
LOST

## SAMPLE TYPES

C. S. - CHUCK  
D. O. - DRIVE-OPEN  
D. F. - DRIVE-FOOT VALVE  
D. P. - DRIVE PISTON  
T. O. - THIN WALLED OPEN  
T. P. - THIN WALLED PISTON

F.S. - FOIL SAMPLE

B. A. - BARREL AUGER  
S. A. - SPIRAL AUGER  
W. S. - WASHED SAMPLE  
R. C. - ROCK CORE  
S. S. - SLEEVE SAMPLE

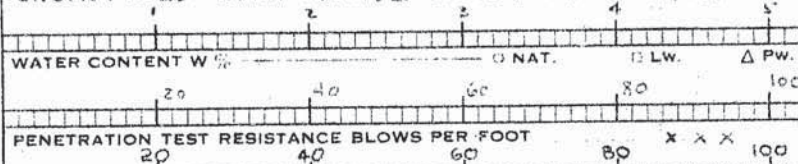
## ABBREVIATIONS

V. -IN-SITU VANE SHEAR TEST	Y. -UNIT WEIGHT
M. -MECHANICAL ANALYSIS	K. -PERMEABILITY
U. -UNCONFINED COMPRESSION	C. -CONSOLIDATION
QC. -TRIAxIAL CONSOLIDATED QUICK	CA. -CASING
Q. -TRIAxIAL QUICK	WL. -WATER LEVEL IN CASING
S. -TRIAxIAL SLOW	WT. -WATER TABLE IN SOIL

## SOIL PROFILE

ELEV. DEPTH (ft)	Elev Depth (m)	DESCRIPTION	STRAT. PLT
247.9 8.82	75.6	GROUND LEVEL	
246.9 1.0	75.3	TOP SOIL	
	0.3		
		VERY STIFF GREY & BROWN MOTTLED CLAY WITH SAND & FERRULES.	
236.4 11.5	72.1		
	3.5	STIFF GREY & DARK GREY VARVED CLAY	
226.9 21.0	69.2		
	6.4		
		FRESH, SOUND HORN BLENDED SYENITE	
211.9 31.0	64.6	END OF HOLE	
	11.0		

## UNCONFINED COMPRESSIVE STRENGTH TONS/SQ FT



## SAMPLES

OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECORDED
					%
					247.9
				17	246.9
		TO	1	18	65
				29	
				35	
				58	241.9
					50
		SS	2	31	
				49	238.9
				92	65
$\gamma = 119$		SS	3	26	
				34	
				46	235.9
					100
		SS	4	12	
				18	
				26	232.9
					65
$\gamma = 123$		SS	5	17	
				15	
				18	229.9
					100
$\gamma = 125$		SS	6	20	
$77/10^6$				21	
				22	226.9
					100
		AXT RC	7	—	
					221.9
					90
		AXT RC	8	—	



# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG. MACHINE

**JOB**

1020

BORING # 9

CASING 4 (STANDARD SAMPLERS TO FIT UNLESS NOTED)

DATUM GEOMETRY

DATE REPORT MARCH 4 1954

SAMPLER HAMMER WT. 430 DROP 15 INCHES

COMPILED BY JA CHECKED BY M.A.M. BORING DATE JAN 20 - FEB 1 1954

### SAMPLE CONDITION



**DISTURBED  
FAIR  
GOOD  
LOST :**

## SAMPLE TYPES

C.S. - CHUNK	F.S. - FOIL SAMPLE
D.O. - DRIVE-OPEN	B.A. - BARREL AUGER
D.F. - DRIVE-FOOT VALVE	S.A. - SPIRAL AUGER
D.P. - DRIVE PISTON	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE
T.P. - THIN WALLED PISTON	S.S. - SLEEVE SAMPLE

## ABBREVIATIONS

V. -IN-SITU VANE SHEAR TEST	7. -UNIT WEIGHT WET
M. -MECHANICAL ANALYSIS	K. -PERMEABILITY
U. -UNCONFINED COMPRESSION	C. -CONSOLIDATION
Qc. -TRIAxIAL CONSOLIDATED QUICK	CA. -CASING
Q. -TRIAxIAL QUICK	WL. -WATER LEVEL IN CASING
S. -TRIAxIAL SLOW	WT. -WATER TABLE IN SOIL

### SOIL PROFILE

ELEV DEPTH		DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W %	NAT.	LW.	PW.	OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECORDED
(ft)	Elev Depth (m)				20	40	60	80	100				%	
					PENETRATION TEST RESISTANCE BLOWS PER FOOT									
248.9 0.0	75.9	GROUND LEVEL												248.9
247.3 1.6	75.4 0.5	TOP SOIL												247.9 80
		VERY STIFF GREY & BROWN MOTTLED CLAY WITH SOME SAND & PEBBLES		245								TO	1	12 12 23 28 41 242.9 50
				240								TO	2	28 46 69 239.9 85
236.9 12.0	72.2 3.7	STIFF TO FIRM GREY & DARK GREY VARNED CLAY		235								SS	3	51 54 57 236.9 85
				230								SS	4	54 25 18 233.9 100
227.9 227.5	69.5	SAND & GRAVEL		225								SS	5	26 25 23 230.9 100
21.0 21.4	6.5	FRESH, SOUND MASSIVE SYENITE GNEISS.		220								SS	6	18 17 17 227.9 100
212.4 36.5	64.8 11.1	END OF HOLE		215								AKT RC	7	— 212.4 100
				210								AKT RC	8	—



## OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG. MACHINE JOB 1020 BORING # 12  
 CASING BX (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT MAR 4 / 1954  
 SAMPLER HAMMER WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY MAJ M BORING DATE FEB 2 / 1954

## SAMPLE CONDITION

## SAMPLE TYPES

## ABBREVIATIONS



DISTURBED  
FAIR  
GOOD  
LOST

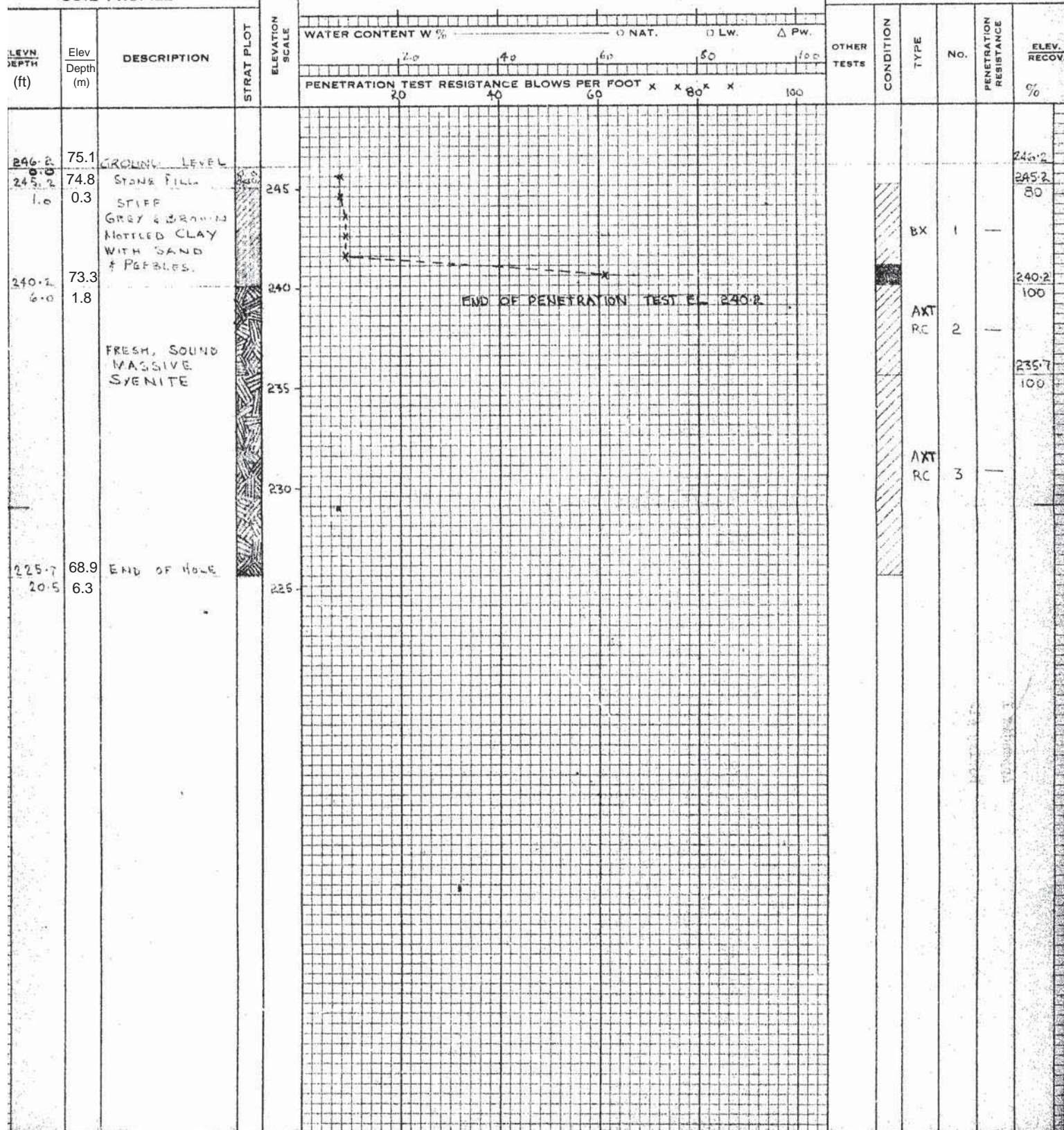
C.S. - CHUNK  
D.O. - DRIVE-OPEN  
D.F. - DRIVE-FOOT VALVE  
D.P. - DRIVE PISTON  
T.O. - THIN WALLED OPEN  
T.P. - THIN WALLED PISTON

F.S. - FOIL SAMPLE  
B.A. - BARREL AUGER  
S.A. - SPIRAL AUGER  
W.S. - WASHED SAMPLE  
R.C. - ROCK CORE

V. - IN-SITU VANE SHEAR TEST  
M. - MECHANICAL ANALYSIS  
U. - UNCONFINED COMPRESSION  
QC. - TRIAXIAL CONSOLIDATED QUICK  
Q. - TRIAXIAL QUICK  
S. - TRIAXIAL SLOW  
Y. - UNIT WEIGHT  
K. - PERMEABILITY  
C. - CONSOLIDATION  
CA. - CASING  
WL. - WATER LEVEL IN CASING  
WT. - WATER TABLE IN SOIL

## SOIL PROFILE

## SAMPLES





## OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG: 1020  
 CASING: 1020  
 STANDARD SAMPLERS TO FIT UNLESS NOTED  
 SAMPLER HAMMER WT: 400 LBS  
 DROP: 15 INCHES

JOB: 1020  
 DATUM: 1020  
 COMPILED BY: JA  
 CHECKED BY: JA  
 PENETRATION BORING: 1020  
 DATE REPORT: 1020  
 BORING DATE: 1020

## SAMPLE CONDITION



DISTURBED  
 FAIR  
 GOOD  
 LOST

## SAMPLE TYPES

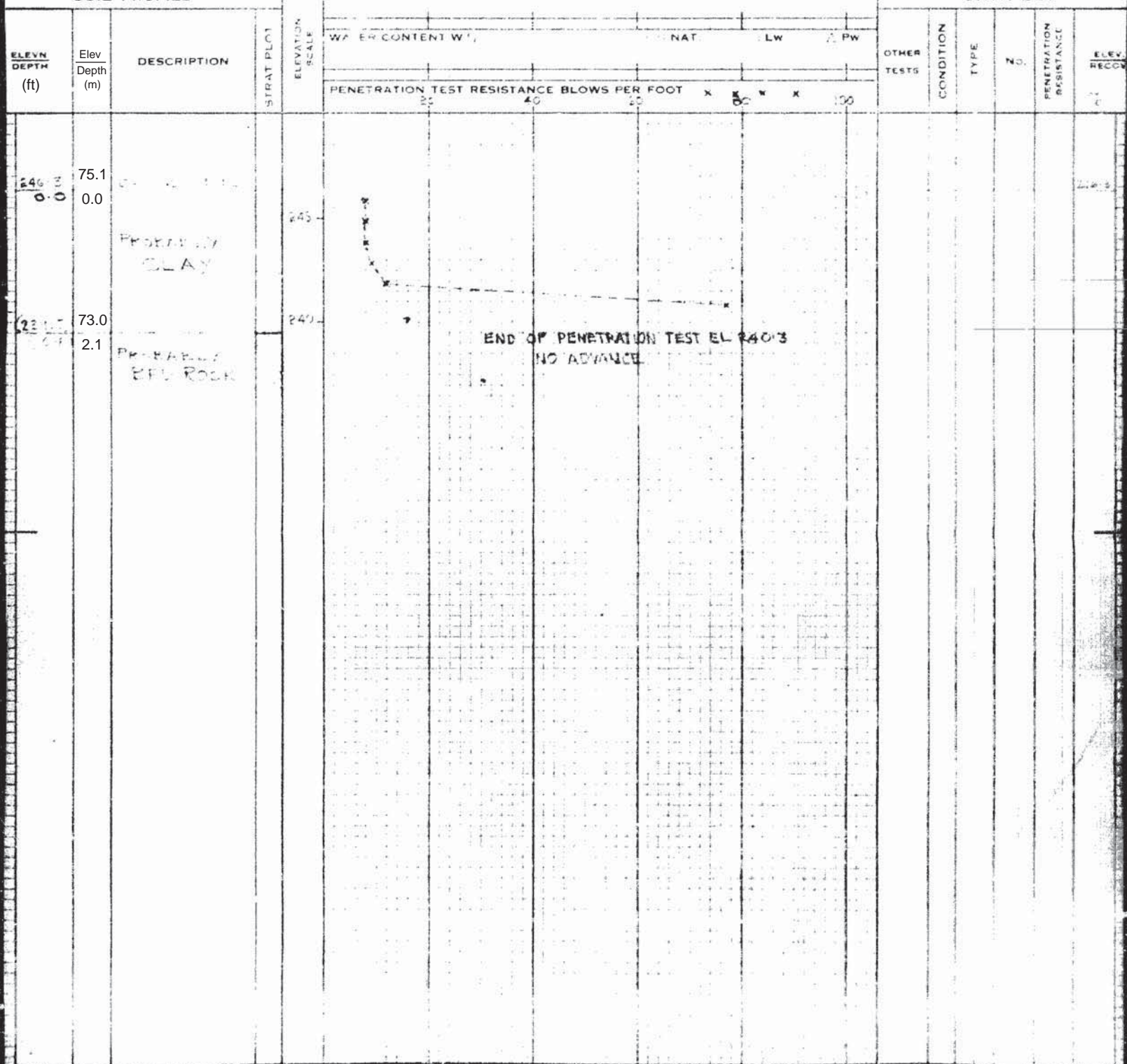
C.S. - CHUNK  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 D.P. - DRIVE PISTON  
 T.O. - THIN WALLED OPEN  
 T.P. - THIN WALLED PISTON  
 F.S. - FOIL SAMPLE  
 B.A. - BARREL AUGER  
 S.A. - SPIRAL AUGER  
 W.S. - WASHED SAMPLE  
 R.C. - ROCK CORE

## ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST  
 M. - MECHANICAL ANALYSIS  
 U. - UNCONFINED COMPRESSION  
 Q. - TRIAXIAL CONSOLIDATED QUICK  
 Q. - TRIAXIAL QUICK  
 S. - TRIAXIAL SLOW  
 γ. - UNIT WEIGHT  
 K. - PERMEABILITY  
 C. - CONSOLIDATION  
 CA. - CASING  
 WL. - WATER LEVEL IN CASING  
 WT. - WATER TABLE IN SOIL

## SOIL PROFILE

## SAMPLES





## OFFICE REPORT ON SOIL EXPLORATION

## APPENDIX I

DRILL RIG. M4-4158  
CASING No. 116 STANDARD SAMPLERS TO FIT UNLESS NOTED  
SAMPLER HAMMER WT. 430 DROP 15 INCHES

JOE 0:00 PERE WATSON  
BORING # 14  
DATE REPORT MAR 1 1954  
COMPILED BY JH CHECKED BY JH BORING DATE FEB 1 1954

### SAMPLE CONDITION

DISTURBED  
FAIR  
GOOD  
LOST

## SAMPLE TYPES

C.S. - CHUCK	F.S. - FOIL SAMPLE
D.O. - DRIVE OPEN	B.A. - BARREL AUGER
D.F. - DRIVE FOOT VALVE	S.A. - SPIRAL AUGER
D.P. - DRIVE PISTON	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE
T.P. - THIN WALLED PISTON	

## ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST	7. - UNIT WEIGHT
M. - MECHANICAL ANALYSIS	K. - PERMEABILITY
U. - UNCONFINED COMPRESSION	C. - CONSOLIDATION
Qc. - TRIAXIAL CONSOLIDATED QUICK	CA. - CASING
Q. - TRIAXIAL QUICK	WL. - WATER LEVEL IN CASING
S. - TRIAXIAL SLOW	WT. - WATER TABLE IN SOIL

### SOIL PROFILE

## SAMPLES





ELEV. DEPTH (ft)	Elev Depth (m)	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W <sub>L</sub>			NAT. LW Δ PW			OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECOVER
					PENETRATION TEST RESISTANCE BLOWS PER FOOT			X X X X								
247.9 0.0	75.6 0.0	PROBABLY CLAY		247												247.9
241.0 6.9	73.5 2.1	PROBABLY BED ROCK		241												
END OF PENETRATION TEST EL. 241.1																



## OFFICE REPORT ON SOIL EXPLORATION

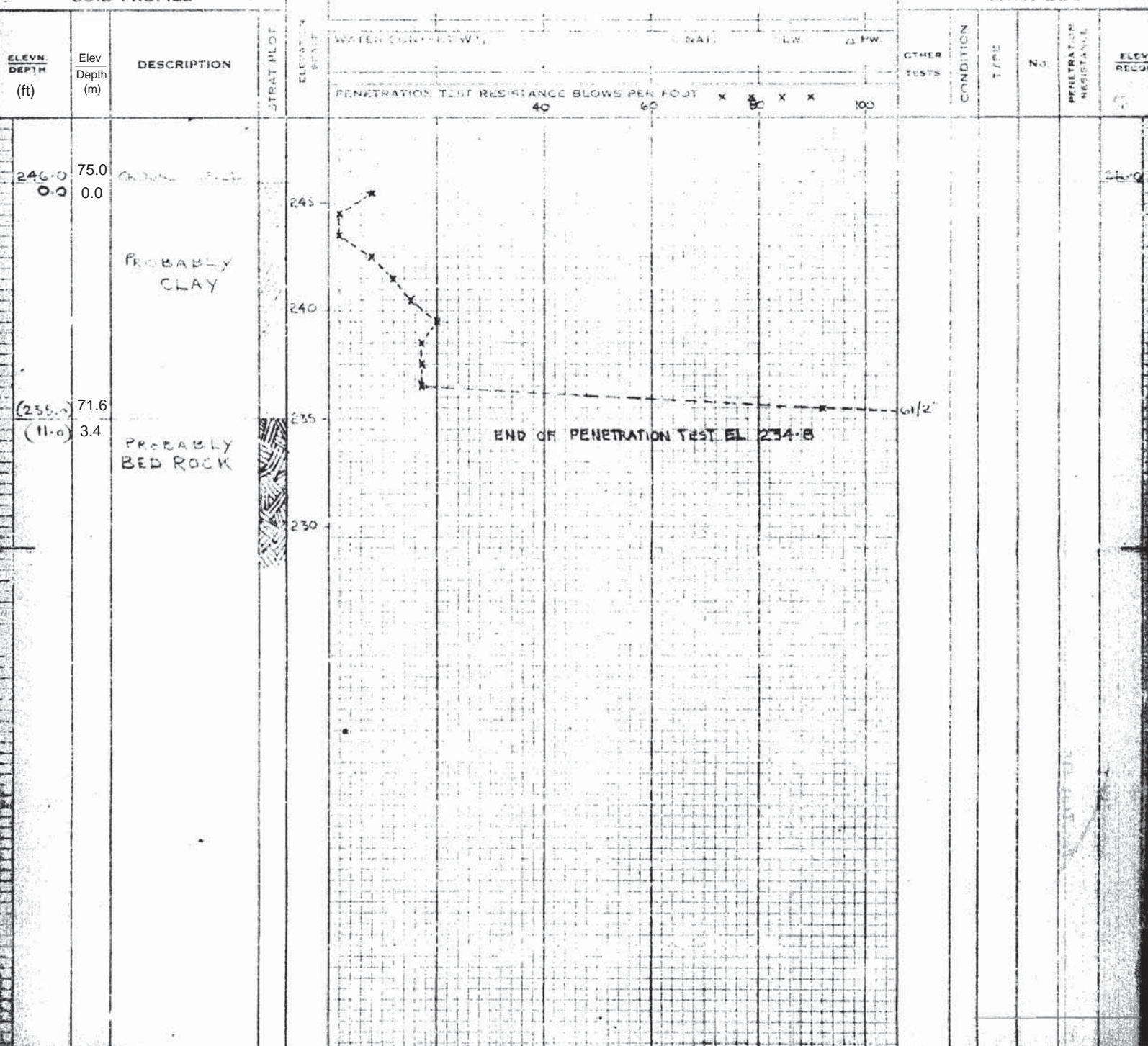
APPENDIX I

DRILL RIG: MACHINE JOB: 1020 PENETRATION: 15  
 CASING: 1 1/2" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM: GEODETIC DATE REPORT: MAR 4/54  
 SAMPLER HAMMER, WT.: 420 DROP: 15 INCHES COMPILED BY: JA CHECKED BY: MA BORING DATE: FEB 2/54

SAMPLE CONDITION		SAMPLE TYPES			ABBREVIATIONS	
	DISTURBED	C.S. - CHUNK	F.S. - FOUL SAMPLE	V. - IN-SITU VANE SHEAR TEST	U. - UNCONFINED COMPRESSION	7. - UNIT WEIGHT
	FAIR	C.O. - DRIVE OPEN	B.A. - BARREL AUGER	M. - MECHANICAL ANALYSIS	U.C. - TRIAXIAL CONSOLIDATED QUICK	K. - PERMEABILITY
	GOOD	D.F. - DRIVE FOOT VALVE	B.A. - BARREL AUGER	U. - UNCONFINED COMPRESSION	U.C. - TRIAXIAL CONSOLIDATED QUICK	C. - CONSOLIDATION
	LOST	D.F. - DRIVE FOOT VALVE	W.C. - WASHED SAMPLE	Q. - TRIAXIAL QUICK	WL. - WATER LEVEL IN CASING	L. - CASING
		D.C. - THIN WALLED OPEN	R.C. - ROCK CORE	S. - TRIAXIAL SLOW	WT. - WATER TABLE IN SOIL	
		T.P. - THIN WALLED OPEN				

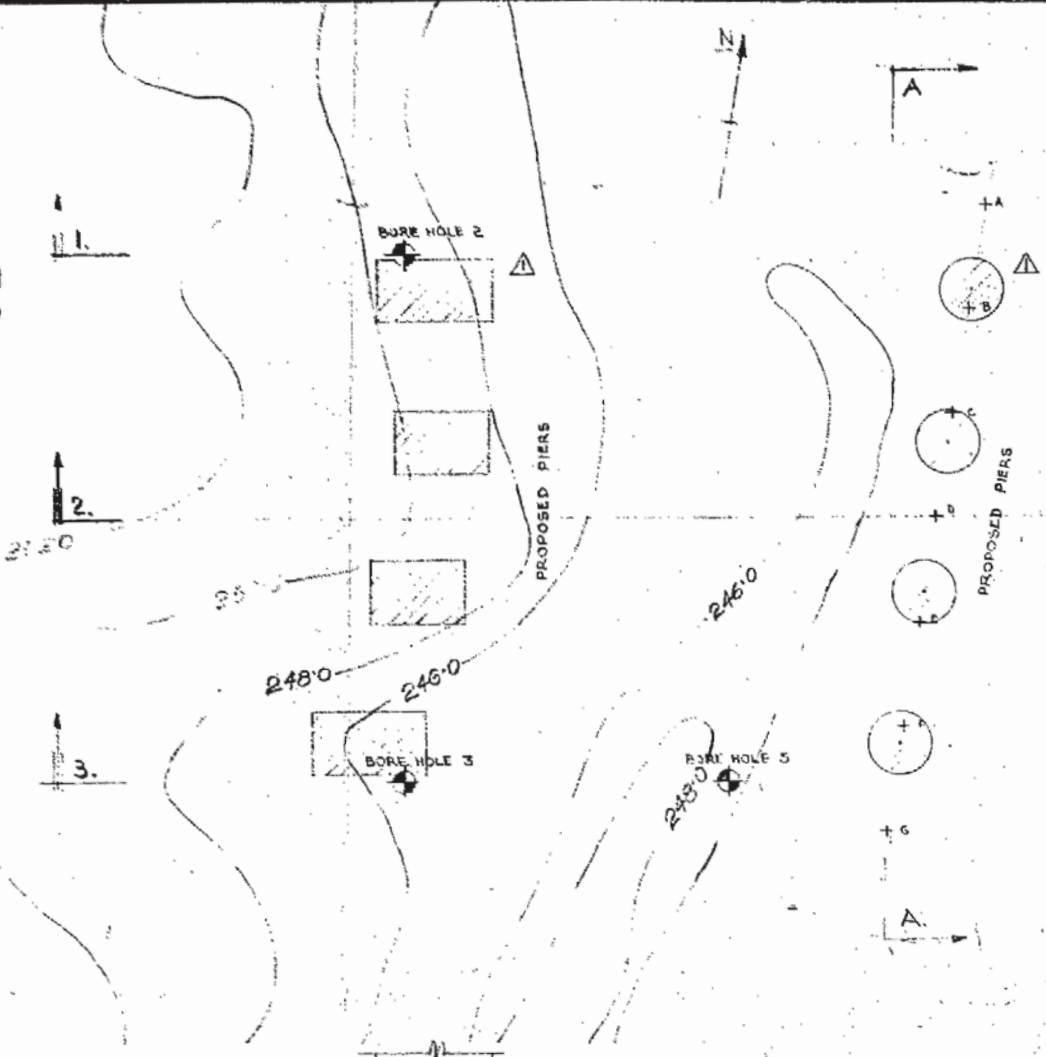
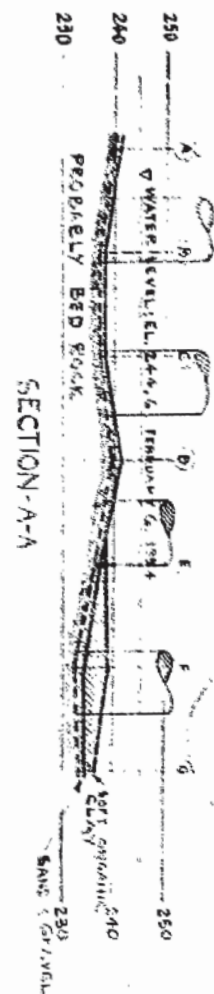
## SOIL PROFILE

## SAMPLES

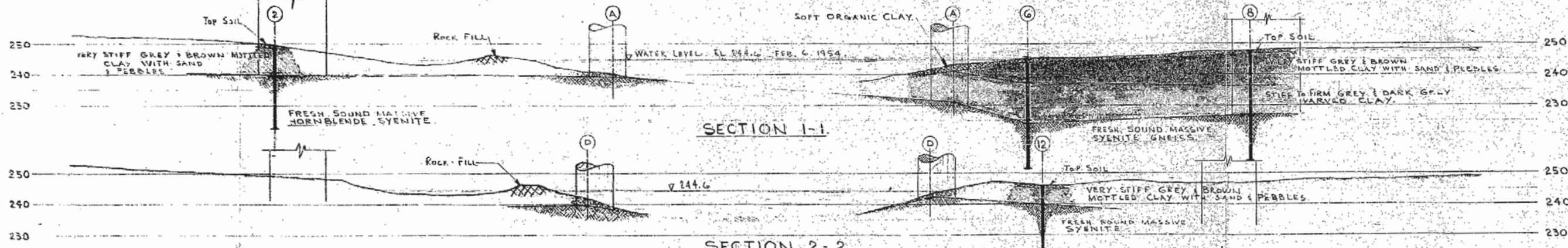




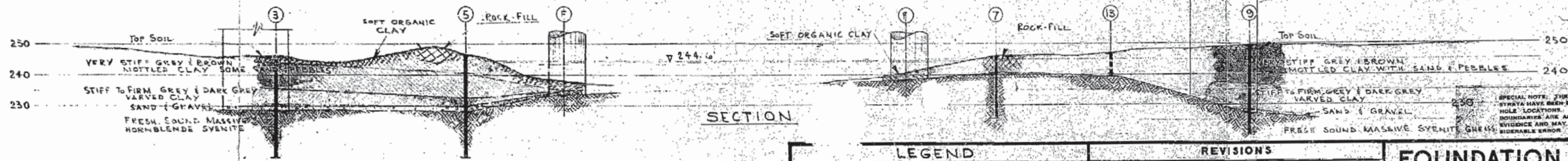
SECTION A-A












SECTION 1-1



SECTION 2-2



SECTION

LEGEND			REVISIONS			FOUNDATION COMPANIES CANADA	
	PLAN	SECTION	MARK	DATE	DESCRIPTION		
PENETRATION TEST				APRIL 19/54	ADDED LINES OF PROPOSED PIERS	DEPARTMENT OF HIGHWAYS ONTARIO	
BORING & PENETRATION TEST						CATARAQUI RIVER BRIDGE	
JET PROBING						BORING PLAN & SOIL STRATIGRAPHY DATE March 4, 1954 SCALE 1" = 20'-0"	
						MADE P.F.A.	CHKD. 
						APPD. 	No. 1020 - C - 2


SPECIAL NOTE: THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BOREHOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO QUOTE "CONSIDERABLE ERROR"



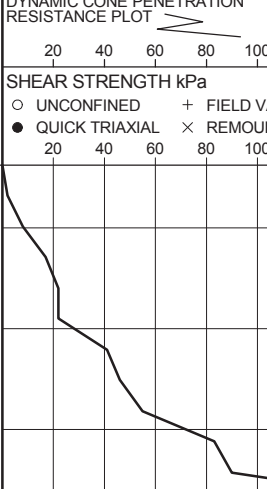


# **APPENDIX B**


## **Record of Borehole Logs and Laboratory Test Results (Golder Associates Ltd., 2009)**


PROJECT 08-1111-0044				RECORD OF BOREHOLE No E23				1 OF 1 METRIC										
G.W.P. 78-99-01				LOCATION N 4904814.1 ; E 308903.4				ORIGINATED BY DM										
DIST HWY 401				BOREHOLE TYPE Portable Equipment, Continuous Sampling				COMPILED BY AT										
DATUM Geodetic				DATE March 9, 2009				CHECKED BY KSL										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
76.6	GROUND SURFACE																	
0.0	Clayey silt, trace sand, containing rootlets (FILL)		1	SS	3													
76.0	Soft Brown Moist		2	SS	15													
0.6	SILTY CLAY, trace sand		3	SS	38													
	Very stiff to hard		4	SS	53													
	Brown to grey Moist		5	SS	35													
73.4	END OF BOREHOLE SPOON REFUSAL	6	SS	100/0.1														
3.2	NOTES:  1. Water level in open borehole at a depth of 2.1 m below ground surface (Elev. 74.5 m) upon completion of drilling.  2. A Dynamic Cone Penetration Test was advanced 0.8 m South of Borehole E23, refusal encountered at a depth of 3.2 m below ground surface (Elev. 73.4 m).  3. An additional borehole was advanced 2 m West of Borehole E23, refusal encountered at a depth of 3.1 m below ground surface (Elev. 73.5 m).																	

PROJECT <u>08-1111-0044</u>		<b>RECORD OF PENETRATION TEST No E23A</b>		1 OF 1 <b>METRIC</b>	
G.W.P. <u>78-99-01</u>		LOCATION <u>N 4904813.3 ; E 308903.6</u>		ORIGINATED BY <u>DM</u>	
DIST <u>          </u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Equipment, Dynamic Cone Penetration Test</u>		COMPILED BY <u>AT</u>	
DATUM <u>Geodetic</u>		DATE <u>March 9, 2009</u>		CHECKED BY <u>KSL</u>	


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		w <sub>p</sub>	w	w <sub>L</sub>		
								○ UNCONFINED   + FIELD VANE ● QUICK TRIAXIAL   × REMOULDED	WATER CONTENT (%)					
76.6 0.0	GROUND SURFACE Start of Dynamic Cone Penetration Test (DCPT)													
73.4 3.2	END OF DCPT Refusal to further Penetration (135 blows/0.3 m)													



PROJECT 08-1111-0044				<b>RECORD OF BOREHOLE No E23B</b>				1 OF 1 <b>METRIC</b>									
G.W.P. 78-99-01				LOCATION N 4904813.3 ; E 308902.0				ORIGINATED BY DM									
DIST _____ HWY 401				BOREHOLE TYPE Portable Equipment, Continuous Sampling				COMPILED BY AT									
DATUM Geodetic				DATE March 9, 2009				CHECKED BY KSL									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
76.7	GROUND SURFACE							20	40	60	80	100					
0.0	Clayey silt, some sand, containing rootlets (FILL)		1	SS	3												
76.1	Soft Brown Moist		2	SS	14												
0.6	SILTY CLAY/CLAYEY SILT, trace to some sand, trace gravel		3	SS	18												
	Firm to hard Brown Moist		4	SS	40												
			5	SS	39												
73.6	END OF BOREHOLE SPOON REFUSAL		6	SS	75												
3.1	NOTE:  1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 74.9 m) upon completion of drilling.																

PROJECT 08-1111-0044			RECORD OF BOREHOLE No E24			1 OF 1 METRIC							
G.W.P. 78-99-01			LOCATION N 4904820.5 ; E 308930.1			ORIGINATED BY DM							
DIST _____ HWY 401			BOREHOLE TYPE Portable Equipment, Continuous Sampling			COMPILED BY AT							
DATUM Geodetic			DATE March 10, 2009			CHECKED BY KSL							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
75.1	GROUND SURFACE							20 40 60 80 100	25 50 75				
0.0	Peat (Fibrous), trace sand, containing rootlets and decomposed wood fragments (FILL)		1	SS	39								
74.5	Hard Black to brown Wet		2	SS	29								
0.6	SILTY CLAY, trace to some sand, trace gravel Very stiff Brown to grey Wet		3	SS	27								
73.2	SILT, some sand, trace gravel Very dense Brown Wet		4	SS	100/0.15								
2.1	END OF BOREHOLE SPOON REFUSAL												
NOTE: 1. Water level in open borehole at ground surface (Elev.75.1 m) upon completion of drilling. 2. Two Dynamic Cone Penetration Tests were advanced 1.8 m East and 2.0 m South of Borehole E24, refusal encountered at a depth of 1.8 m and 2.0 m below ground surface (Elev.73.3 m and 73.1 m).													

PROJECT <u>08-1111-0044</u>		<b>RECORD OF PENETRATION TEST No E24A</b>		1 OF 1 <b>METRIC</b>	
G.W.P. <u>78-99-01</u>		LOCATION <u>N 4904819.9 ; E 308931.4</u>		ORIGINATED BY <u>DM</u>	
DIST <u>          </u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Equipment, Dynamic Cone Penetration Test</u>		COMPILED BY <u>AT</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10, 2009</u>		CHECKED BY <u>KSL</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT   NATURAL MOISTURE   LIQUID CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED   + FIELD VANE ● QUICK TRIAXIAL   × REMOULDED			WATER CONTENT (%) w <sub>p</sub> w   w <sub>L</sub>				GR	SA	SI	CL
75.1 0.0	GROUND SURFACE Start of Dynamic cone Penetration Test (DCPT)																	
73.2 1.8	END OF DCPT Refusal to further Penetration (Hammer Bouncing)																	

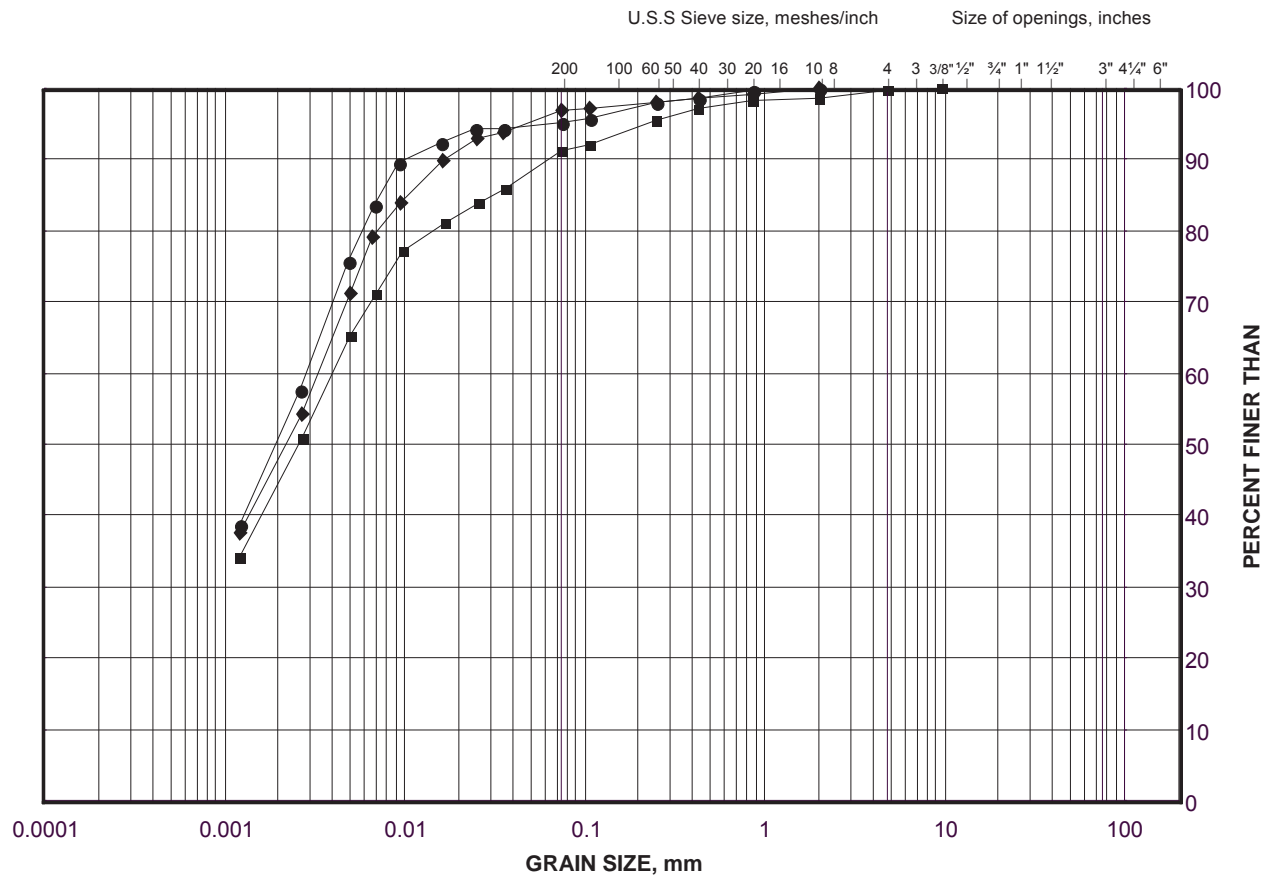
PROJECT <u>08-1111-0044</u>		<b>RECORD OF PENETRATION TEST No E24B</b>		1 OF 1 <b>METRIC</b>	
G.W.P. <u>78-99-01</u>		LOCATION <u>N 4904818.7 ; E 308929.8</u>		ORIGINATED BY <u>DM</u>	
DIST <u>          </u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Equipment, Dynamic Cone Penetration Test</u>		COMPILED BY <u>AT</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10, 2009</u>		CHECKED BY <u>KSL</u>	

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT   NATURAL MOISTURE   LIQUID CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa			WATER CONTENT (%)			GR	SA	SI		CL			
						○ UNCONFINED			+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED								
75.1	GROUND SURFACE																		
0.0	Start of Dynamic cone Penetration Test (DCPT)																		
73.1																			
2.0	END OF DCPT Refusal to Further Penetration (100 blows/0.15 m)																		

# GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE B1



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E22	10	71.3
■	E24	3	73.6
◆	E23	3	75.1

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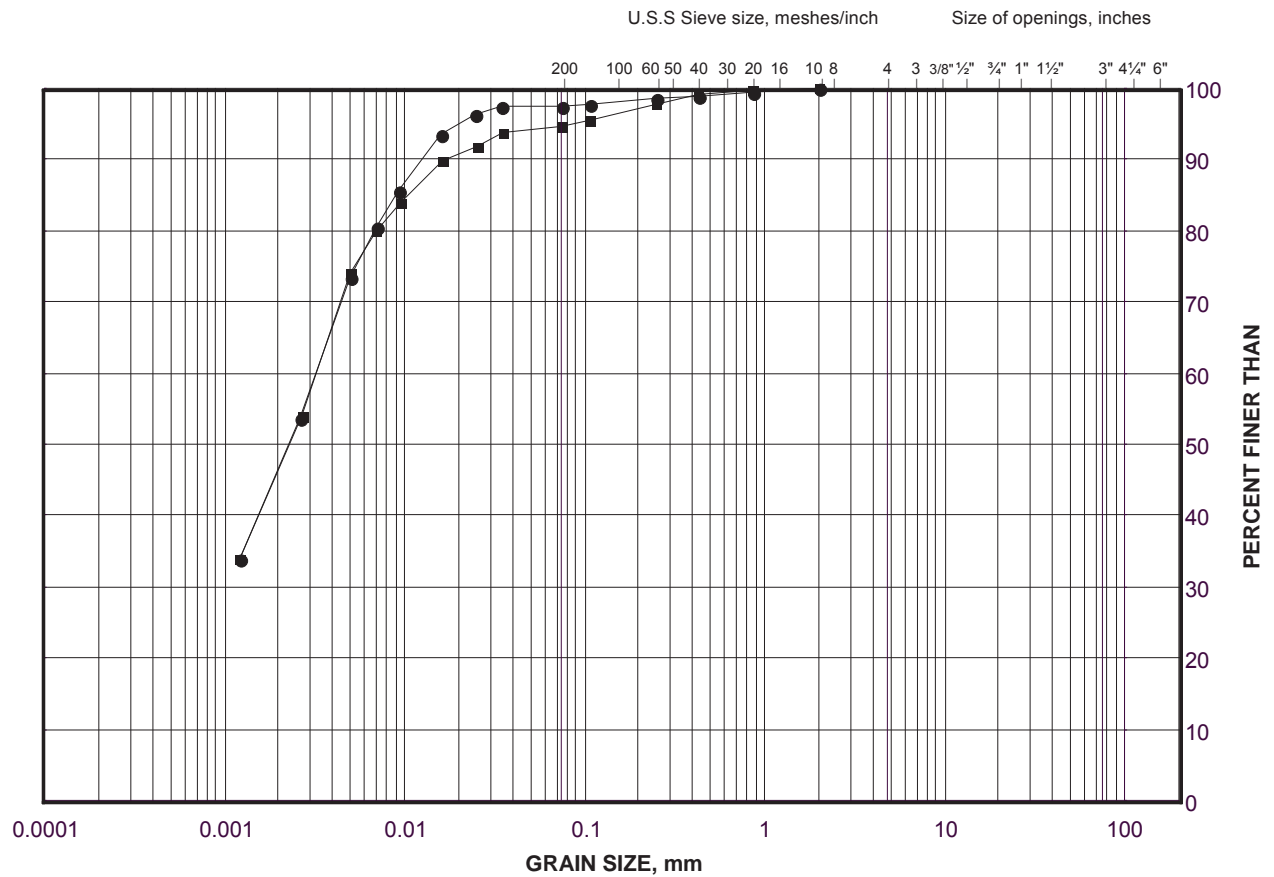
Golder Associates

Date: 30-Mar-10

# GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B2



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

## LEGEND

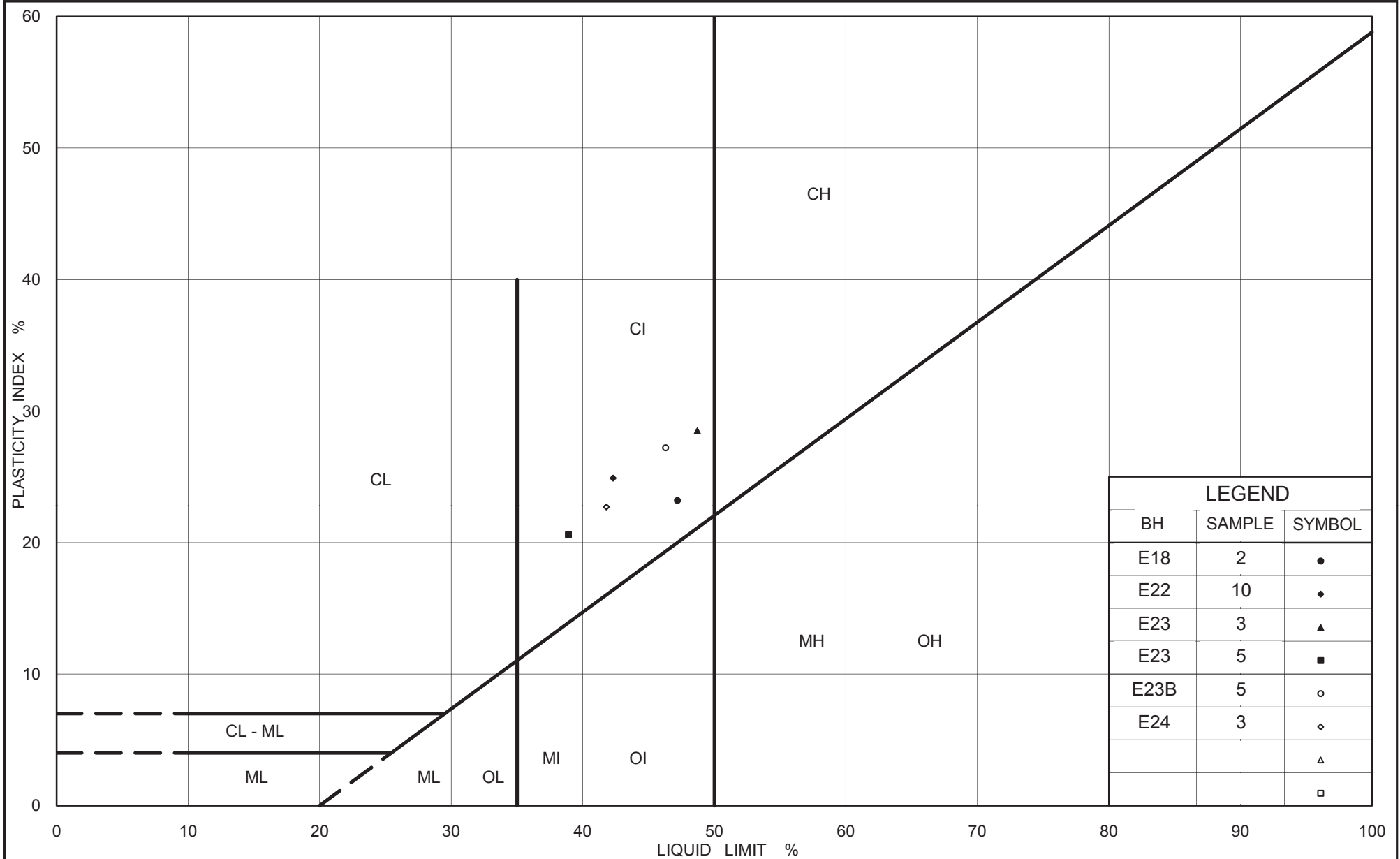
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E23B	3	75.2
■	E23B	5	74.0

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Date: 30-Mar-10



Ministry of Transportation

Ontario

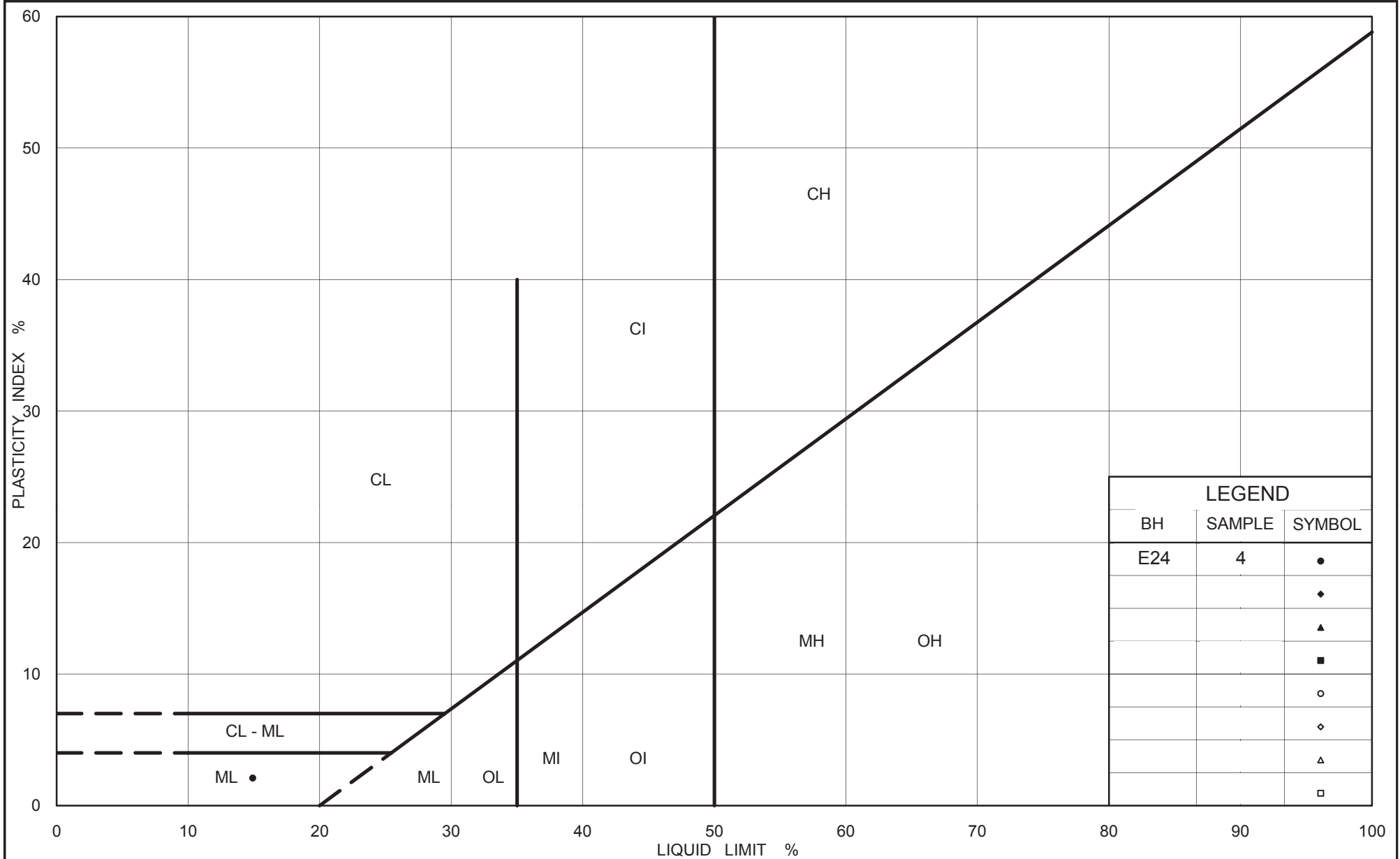
## PLASTICITY CHART Silty Clay

Figure No. B3

Project No. 08-1111-0044

Checked By: KSL





Ministry of Transportation

Ontario

## PLASTICITY CHART

### Silt

Figure No. B4

Project No. 08-1111-0044

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At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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