



December 2014

## REPORT ON

**Foundation Investigation and Design  
Brinston Road Underpass Replacement  
Site No. 31-202  
Highway 401, 19 km East of Highway 416  
Iroquois, Ontario  
W.P. 316-01-01**

**Submitted to:**  
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REPORT



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# **PART A**

**FOUNDATION INVESTIGATION REPORT  
BRINSTON ROAD UNDERPASS REPLACEMENT  
SITE 31-202  
HIGHWAY 401, 19 KM EAST OF HIGHWAY 416  
IROQUOIS, ONTARIO  
W.P. 316-01-01**



### 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. Two additional bridge replacements were added as part of Scope Change 4. This report presents the results of the detailed foundation investigation conducted for the replacement of the Brinston Road underpass, Site No. 31-202 (WP 316-01-01), located on Highway 401 about 19 km east of Highway 416 in Iroquois, 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 Golder Associates Ltd. (Golder) for the MTO (then the Ontario Department of Highways) titled “*Report to Department of Highways, Ontario, on Site Investigation, Proposed Highway 401 Underpass, Iroquois, Ontario*”, dated February 1961 (GEOCRES No. 31B-31).

The purpose of the current foundation investigation was to assess the subsurface conditions for the proposed bridge replacement by drilling three additional boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO’s Request for Proposal (RFP) dated April 2012 and the work was carried out in accordance with Golder’s change proposal to MMM, dated October 18, 2013.



## **2.0 SITE DESCRIPTION**

The Brinston Road underpass is located on Highway 401, about 19 km east of Highway 416 in Iroquois, Ontario. The existing bridge (Site No. 31-202) is located at about Station 17+770 on Highway 401.

The existing bridge consists of a four-span concrete deck with abutments founded on piles and piers supported on spread footings. The existing structure is aligned approximately north-south and is about 65 m long and 8.5 m wide. It is understood that the structure was originally built in 1965 and was last rehabilitated in 1984.

The natural ground surface is at about Elevation 81 m at the south of Highway 401, and increases to the north.

In the vicinity of the site, Highway 401 is a four-lane, divided highway and Brinston Road is a two-lane roadway. In the area of the underpass, Brinston Road has been constructed on embankments that are up to about 6 to 7 m in height above Highway 401 and the natural ground level, with the pavement surface ranging from about Elevation 86.8 to 87.0 m in the vicinity of the bridge. The Brinston Road embankment side slopes are oriented as steep as about 2 horizontal to 1 vertical (2H:1V). Based on visual observation at the time of the site investigation, the existing embankment side slopes appear to be performing satisfactorily. Concrete slope paving exists on the embankment front slopes.



### 3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed underpass bridge replacement was carried out between January 8 and 14, 2014 at which time three boreholes (numbered 13-421 to 13-423, inclusive) were advanced at the locations shown on Drawing 1.

The boreholes were advanced with 108 mm inside diameter continuous-flight hollow-stem augers and/or wash boring using NW casing with a truck-mounted drill rig, supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rout Quebec. The boreholes were advanced to depths of about 15.9 to 21.6 m below the existing pavement/ground surface in the overburden, to auger refusal. Soil samples in the boreholes were obtained at vertical intervals of about 0.60 to 1.52 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A standpipe piezometer was installed in Borehole 13-421 to monitor the groundwater level at the site. The standpipe consists of a 32 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock. The site conditions were restored following completion of work.

The field work was supervised by members of Golder's technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory in Ottawa for further examination. Index and classification tests consisting of grain size distribution, Atterberg limits, and water content testing were carried out on selected soil samples. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations were measured relative to existing site features by Golder personnel. The elevations and horizontal coordinates of the boreholes were established based on site survey data received from MMM (survey dated November 27, 2013). The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawings 1 and 2.

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)
13-421	North Abutment	4970729.2	399125.7	86.8
13-422	Central Pier (Within the median of Highway 401)	4970698.4	399150.3	80.9
13-423	South Abutment	4970656.9	399165.9	86.7

**Notes:** 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system.  
2) Ground surface elevations shown are relative to Geodetic Datum.



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Five boreholes and five penetration tests were advanced as part of the original investigation at this site in 1960. The elevations of the ground surface at the borehole and penetration test locations were surveyed relative to Geodetic datum at the time of the investigation. The borehole locations in plan were established relative to existing site features by comparing the site plans prepared at the time of original design with the current site survey data received from MMM. As such, the MTM NAD83 northing and easting coordinates summarized in the following table and shown on Drawings 1 and 2 should be considered approximate only.

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Former Ground Surface Elevation (m)
BH-1	South Abutment	4970659.4	399154.9	80.4
BH-3	South Pier	4970678.1	399157.1	80.5
BH-5	Central Pier (Within the median of Highway 401)	4970689.6	399138.4	80.7
BH-7	North Pier	4970714.1	399137.1	79.7
BH-9	North Abutment	4970721.7	399120.6	80.6

**Notes:** 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system and are approximate only.  
2) Ground surface elevations shown are relative to Geodetic Datum.



## **4.0 SITE GEOLOGY AND STRATIGRAPHY**

### **4.1 Regional Geological Conditions**

The site is located in the physiographic region known as the Glengarry Till Plain, just east of the Edwardsburg Sand Plain, as delineated in *The Physiography of Southern Ontario*.<sup>1</sup>

The Glengarry Till Plain is characterized by the undulating to rolling ground surface where the depth to bedrock is typically less than 30 m and glacial till is typically less than 7 m deep.<sup>1</sup>

### **4.2 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes put down as part of the current investigation and the results of related in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The borehole logs from the previous investigation, carried out in 1960 (prior to construction of the bridge), are included in Appendix B. The results of geotechnical laboratory testing carried out as part of the 2014 and 1960 investigations are also included in Appendices C and D, respectively.

The interpreted stratigraphic conditions along the centreline of the existing bridge and at the proposed abutment and central pier locations are shown on Drawings 1 and 2. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic sections included on Drawings 1 and 2 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.

In general, the subsurface conditions at the location of the proposed bridge replacement consist of the embankment fill and silty clay at the abutments and grade fill at the central pier location, overlying glacial till and dolomitic bedrock.

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

#### **4.2.1 Pavement Structure and Embankment Fill**

The Brinston Road pavement structure was penetrated in the northbound lane at Boreholes 13-421 and 13-423. At the borehole locations, the pavement structure consists of about 100 to 200 mm of asphalt/concrete overlying about 200 mm of gravel and sand base course. The granular base is underlain by about 7.1 to 7.2 m of subbase/embankment fill. The subbase/embankment fill generally consists of sand, with varying amounts of gravel and silt to sandy silt with varying amounts of clay, and trace to some gravel. The embankment fill was fully penetrated to depths of about 7.6 m (Elevations 79.1 to 79.2 m) at Boreholes 13-421 and 13-423, respectively.

The grade fill within the median of Highway 401 was penetrated at Borehole 13-422. At the borehole location, the pavement structure consisted of about 0.5 m of crushed rock over 1.8 m of sandy clayey silt fill.

Standard Penetration Test (SPT) "N" values measured in the embankment fill ranged from 5 to 67 blows per 0.3 m of penetration indicating a loose to very dense or stiff to very stiff relative density.

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<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.



The results of grain size distribution testing carried out on six samples of the embankment fill are provided on Figures C1 and C2 in Appendix C. The results of grain size distribution testing carried out on one sample of the grade fill within the Highway 401 median is shown on Figure C3.

The results of Atterberg limit testing carried out on two samples of the clayey embankment fill indicate plasticity index values of about 7 and 8 percent and liquid limits of about 22 and 23 percent, as shown on Figure C6, indicating that the deposit is a clayey silt of low plasticity. The measured water contents of the samples varied from approximately 5 to 23 percent.

### 4.2.2 Silty Clay

A layer of silty clay about 1.1 m thick was encountered below the south embankment fill at Borehole 13-423. This layer was fully penetrated to a depth of about 8.7 m (Elevation 78.0 m).

An SPT “N” value of 8 blows per 0.3 m of penetration was recorded within the silty clay deposit. An in situ vane test in this material measured an undrained shear strength of greater than 96 kPa, indicating a very stiff consistency.

The results of grain size distribution testing carried out on one sample of the silty clay are provided in Figure C4 in Appendix C. An Atterberg limits test carried out on one sample of the silty clay measured a plasticity index value of about 26 percent and a liquid limit of about 52 percent, as shown on Figure C7, indicating that the deposit has a medium to high plasticity. The measured natural water content of the sample of this material was about 34 percent.

### 4.2.3 Glacial Till

The fill and silty clay deposit, where present, are underlain by a deposit of glacial till. In general, the glacial till is a heterogeneous mixture of sandy clayey silt to silty sand with varying amounts gravel. The glacial till was fully penetrated in three of the boreholes put down as part of the 1960 investigation (BH-1, BH5, and BH-9) at elevations ranging from about Elevation 64.8 to 65.3 m. All boreholes put down as part of the 2014 investigation were terminated following effective refusal of the auger drill rig at the inferred interface between the glacial till and the underlying bedrock. The inferred base of the glacial till is between about Elevation 65.1 and 65.4 m.

The results of grain size distribution testing carried out on seven samples of the glacial till obtained during the 2014 investigation are provided in Figure C5 in Appendix C. Figures D1 and D2 in Appendix D provide the grain size distribution test results of samples obtained during the 1960 investigation from the upper portion of the deposit (Elevation 72.2 to 76.5) and the lower portion of the deposit (Elevation 67.4 to 70.4), respectively. The grain size distribution test results may not reflect the full gravel contents of the material, because the samples were retrieved using a 50 mm outside diameter split-spoon or small-diameter tube sampler.

The results of Atterberg limit testing carried out on samples of the glacial till indicate plasticity indices of about 5 to 10 percent and liquid limits of about 15 to 23 percent. The results of Atterberg limit testing carried out on samples obtained from the 2014 investigation are shown on Figure C8 and indicate that the deposit behaves as a low plasticity clayey silt. The measured natural water contents of samples of the till obtained from each of the investigations ranged from about 5 to 11 percent, near the plastic limit for the till deposit.



Six triaxial compression tests were carried out on samples of the glacial till obtained during the 1960 investigation and the results are shown on Figure D3 in Appendix D. The results of three oedometer consolidation tests carried out on samples of the glacial till obtained during the 1960 investigation are shown on Figure D4 in Appendix D, and give an average compression index ( $C_c$ ) of about 0.05.

Within the upper 3 to 4 m of the deposit (above Elevation 75.0 m), the SPT “N” values measured in the glacial till range from about 7 to 69 blows per 0.3 m of penetration. In situ vane testing within this portion of the deposit measured undrained shear strengths of greater than about 100 kPa. These values correspond to a very stiff to hard consistency.

Below about Elevation 75 m, SPT “N” values ranging from about 2 to 11 blows per 0.3 m of penetration were measured in the glacial till deposit and in situ shear vane test results ranged from about 30 to 150 kPa, indicating a firm to very stiff consistency.

#### 4.2.4 Refusal and Bedrock

During the current investigation, sampler refusal was encountered at a depth of about 21.4 m (Elevation 65.4 m) in Borehole 13-421. Practical refusal to advancement of the augers was encountered at depths of about 15.9 and 21.6 m (Elevation 65.1 m) in Boreholes 13-422 and 14-423, respectively. During the 1960 investigation, the bedrock encountered beneath the glacial till was cored for 4.9 to 5.4 m in Boreholes BH-1, BH-5 and BH-9. The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
13-421	86.8	21.4	65.4
13-422	80.9	15.9	65.1
13-423	86.7	21.6	65.1
BH-1	80.3*	15.5*	64.8
BH-5	80.7*	15.8*	64.9
BH-9	80.5*	15.2*	65.3

**Note:** \* Ground surface and depth to bedrock at the time of the 1960 investigation.

The bedrock encountered in Boreholes BH-1, BH-5, and BH-9 was indicated to be hard, sound grey dolomite with occasion softer light bands.





#### **4.2.5 Groundwater Conditions**

The groundwater levels measured in the monitoring well in Borehole 13-421 are summarized in the table below:

<b>Borehole</b>	<b>Ground Surface Elevation (m)</b>	<b>Water Level Depth (m)</b>	<b>Water Level Elevation (m)</b>	<b>Date</b>
13-421	86.8	7.1*	79.7*	January 10, 2014
		6.2	80.6	February 19, 2014
		7.0	79.8	August 21, 2014

**Note:** \* Groundwater level may not have stabilized following drilling at the time of measurement.

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

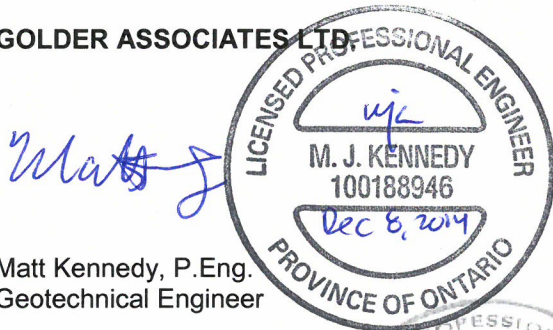


## FOUNDATION REPORT BRINSTON ROAD UNDERPASS REPLACEMENT - HIGHWAY 401

### 5.0 CLOSURE

This 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., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

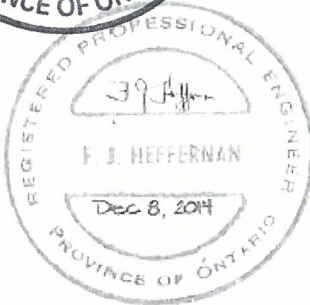
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# **PART B**

**FOUNDATION DESIGN REPORT**  
**BRINSTON ROAD UNDERPASS REPLACEMENT**  
**SITE 31-202**  
**HIGHWAY 401, 19 KM EAST OF HIGHWAY 416**  
**IROQUOIS, ONTARIO**  
**W.P. 316-01-01**



## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides foundation design recommendations for the proposed replacement of the existing Brinston Road underpass on Highway 401. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation as well as that obtained during the previous subsurface investigation carried out in 1960 as part of the original design. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the replacement structure.

Where comments are made on construction, they are provided to highlight those aspects that could affect the 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, scheduling and the like.

The existing bridge is shown in plan on Drawing 1 and consists of a two-lane, four-span, cast-in-place concrete structure. The two middle spans are about 20.2 m long, and the two outer spans are about 11.6 m long. It is understood that the existing bridge, constructed in 1965, is to be replaced with a two-span structure on the same alignment as the existing bridge with no significant change in width. The new underpass will be founded on abutments located within or near the existing abutment foundation footprints. The proposed pavement grades at the new structure will be up to about 1.0 m higher than the existing pavement grades.

### **6.2 Existing Foundations**

The existing Brinston Road underpass is a four-span structure with a reinforced concrete deck and non-integral abutments. The existing underpass bridge is understood to be in fair condition. Based on the 1961 design drawings (Drawings TWP #28-202-1A to #28-202-12A, inclusive), the existing foundations beneath the abutments are understood to consist of 300 mm diameter piles driven to about Elevation 76.5 m through the compact gravelly sand and stiff to very stiff sandy clayey silt embankment fill, to terminate in the stiff to very stiff sandy clayey silt and silty sand glacial till. The correspondence included in the GEOCRE database included discussion of the design and use of steel pipe piles; however, the General Arrangement drawing (Drawing TWP #28-202-1A), dated June 1961, specifies the use of timber piles at the abutment foundations.

Each abutment foundation consists of nine to ten piles on each of three rows: the inner row battered at 1H:3V, the middle row battered at 1H:5V, and the outer row vertical. The design load on each pile was about 150 kN (15 tons). Both abutment pile caps are perched within the existing embankments with the top of each pile cap at about Elevation 83.8 m. The existing abutments are supported on piles deriving resistance from within the upper portion of the clayey glacial till only. The limited capacity of the pipe piles at the abutments of the existing four-span bridge are not considered to be sufficient for support of the abutments of the proposed two-span structure.

No cobbles or boulders were encountered during augering of the boreholes through the embankment fill in the present investigation or in the underlying glacial till in the present or previous investigations. Therefore, it is expected that most of the existing piles would have been driven on line. However, because of the glacial origin of the till deposit, cobbles or boulders should be expected. The position of the pile heads of the existing piles can be verified when the pile caps are removed.



The existing pier foundations are understood to consist of spread footings that measure about 2.6 m by 11.0 m, which are founded on glacial till between about Elevation 77.0 m and 76.7 m, which is on the order of 4 m below the existing Highway 401 pavement grade (at about Elevation 80.8 m).

### 6.3 Foundation Options

Based on the subsurface conditions, only deep foundation options have been considered for the replacement of the existing Brinston Road underpass, as shallow foundation would not provide sufficient bearing resistances or acceptable settlement performance for the structure. Due to the loose relative density of the embankment fills, spread footings founded within the existing embankments are not considered to be feasible. Similarly, the upper soils encountered at the central pier are loose to compact or firm to stiff and are not considered to be suitable for support of spread footings for the central pier. A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the dolomite bedrock are feasible for support of the replacement bridge structure. This option would provide high geotechnical resistances and minimal post-construction settlements; in addition, this option would permit the use of integral abutments. Steel H-pile foundations would allow for the construction of integral abutments. The use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which may contain cobbles and boulders) and seating onto the dolomite bedrock.
- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and central pier. This foundation option would have similar advantages to steel H-piles in terms of high geotechnical resistances and minimal settlements. Though cobble or boulder-sized particles were not encountered at the borehole locations, some should be expected in the glacial till deposit. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation if cobbles and/or boulders are encountered within the till deposit during driving.
- **Drilled concrete caissons:** Caissons deriving their support from bearing within the dolomite bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from potential water-bearing cohesionless layers within the till soils during construction. In addition, the caissons would have to be socketted at least nominally into the bedrock to permit cleaning of the caisson bases, and such sockets would have to be advanced by rock coring and/or chisel drilling into the strong dolomite bedrock. For this deep foundation option, consideration must also be given to removal of the existing abutment piles, as the proposed new abutment is to be located at approximately the same location as the existing abutment; while new steel H-piles or pipe piles may be able to be located so as to avoid conflict with the existing piles, larger diameter caissons would likely necessitate removal of all existing piles.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments and centre pier for the bridge replacement on steel H-piles driven to found on the bedrock.



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

### **6.4.1 Founding Elevations**

The abutments for the replacement bridge may be supported on steel H-piles or closed-ended pipe (tube) piles driven to found on the dolomite bedrock. Based on the geotechnical investigations carried out at the site, the bedrock surface is considered to be relatively flat and was encountered between about Elevation 64.8 m and 65.4 m at the borehole locations. Based on the borehole results, 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 pile tip elevations are recommended for design of steel H-piles or pipe piles:

<b>Foundation Element</b>	<b>Borehole Numbers</b>	<b>Bedrock Surface Elevation (m)</b>	<b>Design Pile Tip Elevation (m)</b>
North Abutment	13-421, BH-9	65.3 to 65.4	65.2
Central Pier	13-422, BH-5	64.9 to 65.1	64.8
South Abutment	13-423, BH-1	64.8 to 65.1	64.7

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

At the abutment locations, the bedrock surface was encountered at elevations ranging from about 64.8 to 65.4 m. Based on the results of the investigations, steel H-piles driven to the bedrock surface would be greater than 5 m in length and are therefore considered to be feasible for use in an integral abutment configuration. However, the upper portion of the piles should be cased in a loose sand filled, corrugated steel pipe (or similar) to provide suitable flexibility of steel H-piles. The corrugated steel pipe will be within the existing compact, gravelly sand fill and stiff clayey silt fill which will provide suitable flexibility for the integral piles.

Depending on the preferred location of the abutment foundations, the piles may be driven behind or in front of the existing pile caps and piled foundations. Consideration may also be given to driving the new abutment piles adjacent to (or in between) the existing piles following removal of the existing pile cap and exposure of the existing piles. It is understood that to minimize the length of the proposed bridge and provide an integral abutment configuration, one row of H-piles is to be installed on an east-west line between the two inner rows of battered piles. The new piles will be offset transversely from the existing piles.

There is a potential for interference between the new piles and the upper 4 m portion of the middle row of battered piles. Augering for the corrugated steel pipe casing will uncover some 3 m of the area of potential conflict. If, on driving the new H piles, interference is encountered with the existing piles within 4 or 5 m, the piles can be withdrawn and moved along the east-west line to avoid the existing pile. The new piles could then be readily driven through the firm to stiff glacial till to the surface of the bedrock. If any of the new piles do not reach the bedrock because of boulders at depth, another pile could be added to provide the required total number of piles bearing on the bedrock.



Cobbles or boulders were not encountered at the borehole locations, but some should be expected in this glacial deposit. For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacial till. In this regard, steel H-piles are preferred over closed-ended steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. Each pile should be reinforced at the tip with a driving shoe to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through soils that may contain boulders, 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 design of HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at Ultimate Limit States (ULS) may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the dolomite bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. Similar axial geotechnical 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 driving shoes and should be driven to bedrock. For piles driven to refusal on bedrock, and as described in OPSS 903, 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.

#### 6.4.3 Downdrag Load (Negative Skin Friction)

The placement of additional embankment fill at the new abutments and immediate approach embankments will raise the effective stress level in the sandy clayey silt that underlies the site, producing settlement that could generate downdrag loads on the new abutment piles. These downdrag loads (i.e., negative skin friction) should be considered in design. No significant raise in the grade of Highway 401 is expected and, therefore, downdrag forces are not anticipated on pile foundations at the new central pier.

The downdrag loads could vary depending on the magnitude of the grade raise and on the sequence of construction. However, assuming an underside of the abutment stem of about Elevation 82.0 m and that the upper portion of the piles are cased in a sand-filled, corrugated steel pipe (CSP) to about Elevation 79.0 m, the unfactored downdrag load acting on a single HP 310x110 pile over the maximum length of pile is estimated to be about 675 kN. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *Canadian Highway Bridge Design Code (CHBDC)*.

#### 6.4.4 Resistance to Lateral Loads

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





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

For cohesionless soils:

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

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

For cohesive soils:

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

Where:  $s_u$  is the undrained shear strength of the soil (kPa); and,  
 $B$  is the pile diameter/width (m).

The following ranges for the values of  $n_h$  and  $s_u$  may be used in the structural analysis. The ranges in values reflect:

- The variability in the subsurface conditions and the soil properties;
- The approximate nature of the analysis;
- The non-linear nature of the soil behaviour (such that  $n_h$  is a function of deflection); and,
- The two extremes of the design; the requirement for flexibility in the case of integral abutments and the requirement for lateral resistance of horizontal loads.

Location	Elevation (m)	Soil Type	$n_h$ (MN/m <sup>3</sup> )	$s_u$ (kPa)
North Abutment (PCL ~ 82.0 m)	79.2 – PCL <sup>1</sup>	Loose to Compact Clayey Silt and Sand (Fill)	2 to 4	-
	75.0 – 79.2	Very Stiff Sandy Clayey Silt (Glacial Till)	-	100 to 125
	65.5 – 75.0	Stiff Sandy Clayey Silt (Glacial Till)	-	40 to 100
	65.5	Bedrock	-	-
Central Pier (PCL ~ 79.2 m)	78.6 – PCL <sup>1</sup>	Loose Clayey Silt and Sand (Fill)	1 to 3	-
	75.0 – 78.6	Very Stiff Sandy Clayey Silt (Glacial Till)	-	100 to 125
	65.5 – 75.0	Firm to Stiff Sandy Clayey Silt (Glacial Till)	-	30 to 75
	65.5	Bedrock	-	-
South Abutment (PCL ~ 82.0 m)	79.3 – PCL <sup>1</sup>	Loose to Compact Clayey Silt and Sand (Fill)	2 to 4	-
	75.0 – 79.3	Very Stiff Sandy Clayey Silt (Glacial Till)	-	100 to 125
	65.5 – 75.0	Firm to Stiff Sandy Clayey Silt (Glacial Till)	-	30 to 75
	65.5	Bedrock	-	-

**Note:** <sup>1</sup> PCL = Pile Cap Level.





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

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

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

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

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

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

## 6.5 Caisson Foundations

### 6.5.1 Founding Elevations

Alternatively, support of the abutments or central pier may be provided by caisson foundations. Due to the relatively high water table and the difficulty in socketting a liner into the strong bedrock, it may not be feasible to dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. The axial geotechnical resistance for rock-socketed caissons should be based on the side-wall (shaft) resistance of the rock socket rather than end-bearing. For design purposes, it is recommended that the caissons be founded at Elevation 63.5 m (i.e., a rock socket of approximately 2 m).

The use of a temporary or permanent liner or casing will be required to advance the caissons through the potential water-bearing cohesionless layers within the glacial till deposit while minimizing loss of ground. The casing should be extended so that it is “seated” a minimum of 300 mm into the bedrock.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).



#### 6.5.2 Axial Geotechnical Resistance

The *unfactored* geotechnical side wall (shaft) resistance at ULS can be taken as 1,500 kPa provided that the caisson socket is formed within competent bedrock (i.e., Rock Quality Designation values greater than 75 percent); however, it is noted that the bedrock coring completed as part of the 1960 investigation did not include reporting of the RQD values, and there is some potential that the rock socket would have to be extended during construction if the rock quality is found to be poor. This value assumes that the side wall of the socket will be cleaned of any smeared material. End-bearing resistance may also be considered in design provided that the base of each caisson is thoroughly cleaned of any cuttings or other material. The *unfactored* geotechnical end-bearing resistance at ULS can be taken as 5,000 kPa. To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

For a 0.9 m diameter caisson socketted 2 m in to the competent bedrock, this would equate to a factored axial geotechnical resistance at ULS of about 4,600 kN. SLS resistances do not apply to caissons founded within the dolostone bedrock, because the SLS resistance for 25 mm of settlement will be greater than the factored axial geotechnical resistance at ULS.

#### 6.5.3 Downdrag Load (Negative Skin Friction)

As discussed for driven steel piles, the placement of additional embankment fill at the new abutments and immediate approach embankments will raise the effective stress level in the sandy clayey silt that underlies the site, producing settlement that could generate downdrag loads on caissons supporting the new abutments. No significant raise in the grade of Highway 401 is expected and, therefore, downdrag forces are not anticipated if caisson foundations are adopted at the new central pier.

The downdrag loads could vary depending on the magnitude of the grade raise and on the sequence of construction. However, for initial assessment purposes, assuming an underside of the abutment stem of about Elevation 82.0 m, the unfactored downdrag load acting on a 0.9 m diameter caisson is estimated to be about 1650 kN. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *Canadian Highway Bridge Design Code (CHBDC)*.

#### 6.5.4 Resistance to Lateral Loads

The resistance to lateral loading developed by the soil in front of the caissons, and the reductions due to group effects, may be determined as outlined in Section 6.4.3.

### 6.6 Feasibility of Integral Abutments

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



The flexible pile-supported abutment foundations discussed in Section 6.4 meet MTO's foundation criteria for integral abutments.

### 6.7 Seismic Considerations

The site is located between Kemptville and Cornwall, Ontario and according to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio,  $A$ , applicable to this site is 0.2. The corresponding acceleration related seismic zone,  $Z_a$ , is 4.

Below the groundwater table, the soil at the site generally consists of an extensive deposit of glacial till consisting of sandy, clayey silt overlying dolomite bedrock. The SPT "N" values obtained in the glacial till ranged between 7 to 69 blows per 0.3 m of penetration in the upper 3 to 4 m of the deposit (above Elevation 75 m), and between 2 to 11 blows per 0.3 m of penetration in the lower portion of the deposit (below about Elevation 75 m). The results of Atterberg limit testing carried out on samples of the glacial till gave plasticity indices ranging from about 5 to 9 percent. The results of grain size distribution testing carried out on seven samples of the glacial till obtained during the 2014 investigation gave fines contents between 44 and 61 percent.

Golder carried out a liquefaction assessment using the subsurface information collected at the site. The methodology used for the assessment is consistent with those outlined in Section C4.6.2 of the *Commentary* to the CHBDC and state-of-practice techniques. The assessment involved comparing the cyclic shear stresses applied to the soil by the design ground motions outlined in the CHBDC, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

Based on the average fines content (about 50 percent) and the relatively low average plasticity index of the samples tested (about 6 percent), the sandy, clayey silt till was first assessed as sand-like material. The CRR was calculated using the parameter,  $(N_1)_{60cs}$ , that is based on the SPT "N" blow counts obtained in the field at the boreholes put down as part of the 2014 investigation and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the assessment considering the till as a sand-like material indicated a generally low susceptibility to liquefaction, with the exception of some thin, discrete, and discontinuous layers within the deposit. Samples of the deposit at these locations were re-examined and, based on the material behaviour and plasticity indices that were typically higher than the average value of the overall deposit, these discrete layers were considered to behave as clay-like soil and also have a low susceptibility to liquefaction. The soil may be considered non-liquefiable for design purposes.

### 6.8 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls:

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS 501 (*Compacting*).



- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150, 3190.100, and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.7 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary* to the CHBDC).

#### 6.8.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50
Passive, $K_p$	3.0

- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43
Passive, $K_p$	3.7	3.7



- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
  - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- Where movements are not sufficient to mobilize the full passive resistance,  $K_p$  may be determined in accordance with Figure C6.16 of the *Commentary* to the CHBDC based on the amount of displacement.

#### 6.8.2 Seismic Lateral Earth Pressures for Design

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

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.8.1, above, plus the earthquake-induced dynamic earth pressure. The site-specific zonal acceleration ratio ( $A$ ) for the site is 0.2. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of  $A = 0.2$ .
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient ( $k_h$ ) used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e.,  $k_h = 0.3$ ). For structures which allow lateral yielding, ( $k_h$ ) is taken as 0.5 times the zonal acceleration ratio (i.e.,  $k_h = 0.1$ ).
- The following seismic active pressure coefficients ( $K_{AE}$ ) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Seismic Active Pressure Coefficients,  $K_{AE}$

Material	Case (a)	Case (b)	
	SSM	Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50



- The above  $K_{AE}$  values for yielding walls are applicable provided that the wall can move up to 250A mm, where A is the design zonal acceleration ratio of 0.2. This corresponds to displacements of up to approximately 50 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

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

K is the static active earth pressure coefficient,  $K_a$  (**to be used for yielding walls**);

K is the static at-rest earth pressure coefficient,  $K_o$  (**to be used for non-yielding walls**);

$K_{AE}$  is the seismic active earth pressure coefficient;

$\gamma$  is the unit weight of the backfill soil ( $\text{kN/m}^3$ ), as given previously;

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

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

## 6.9 Approach Embankments

It is understood that the overall grade of Brinston Road will be raised up to about 1.0 m to accommodate an increase in the bridge soffit elevation required for clearance above Highway 401. In general, the existing width and alignment of Brinston Road are to be maintained and, therefore, the existing embankments will require widening by approximately 3 m to accommodate the proposed grade raise.

Based on the results from the boreholes drilled through the existing Brinston Road embankments, the road structure is generally underlain by embankment fill consisting of gravelly sand, overlying clayey silt and sand which is underlain by clayey sandy silt till and dolomite bedrock. A layer of silty clay about 1.1 m thick was encountered beneath the southern embankment fill (Borehole 13-423), overlying the glacial till deposit.

The silty clay encountered beneath the southern embankment and the glacial till deposit are compressible soils that are expected to experience some consolidation settlements under increased load. Within the footprint of the existing approach embankments, the increased load imposed by additional fill placed to accommodate the proposed 1.0 m high grade raise would result in additional settlement of these compressible soils of less than about 25 mm beneath the centreline of the embankment. Though relatively small in magnitude, these settlements would occur over time following construction of the replacement bridge and could generate downdrag loads on deep foundations (as described in Section 6.4.3 and 6.5.3).

### 6.9.1 General Embankment Construction

It is recommended that all topsoil/organic material or existing loose surficial fill present within the footprint of the portions of the embankments that are widened to accommodate the grade raise be stripped prior to placement of embankment fill. The topsoil/organic material should be stripped to the underlying very stiff silty clay or sandy clayey silt till.





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The new embankment fill associated with the grade raise and widening for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (*Earth Excavation and Grading*) and OPSS 501 (*Compacting*). Benching of the existing Brinston Road embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil (*OPSS 802 – Topsoil*) and seeding (*OPSS 804 – Seed and Cover*) or pegged sod (*OPSS 803 – Sodding*) is recommended as soon as practicable after construction of the embankments.

### 6.9.2 Global Stability

Static and seismic slope stability analyses of the embankments with the proposed grade raise were carried out with the commercially available slope stability analysis software, SlopeW (part of the software package, Geo-Studio 2007 Version 7, produced by Geo-Slope International Ltd.). A Morgenstern-Price method was used to determine the factor of safety. The analyses were based on the existing topographic information provided by the design team and the available subsurface information.

The soil and bedrock stratigraphy between the borehole locations is based on our interpretation of the geological conditions of the area and consequently the actual conditions may vary from that used in our model. The soil parameters used in the analyses were based on in situ and laboratory testing data as well as published correlations and are given in the following table:

Soil Stratum	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle	Undrained Shear Strength (kPa)
Grade Fill	21	35°	-
Loose to Compact Clayey Silt and Sand (Existing Embankment Fill)	20	31°	-
Stiff Silty Clay	18.5	29°	50
Very Stiff Sandy Clayey Silt (Glacial Till)	22.7	33°	150
Stiff Sandy Clayey Silt (Glacial Till)	22.7	33°	50

The analyses were carried out for both static undrained (i.e., short-term) and static drained (i.e., long-term) conditions. Static undrained conditions represent the critical condition experienced during and immediately following construction of the embankments. With time, the excess pore water pressures generated in the silty clay and glacial till deposits as a result of the additional loading would dissipate and ‘drained’ conditions would occur, resulting in a higher factor of safety against instability. A minimum factor of safety of 1.3 is considered acceptable against static undrained and static drained deep-seated embankment instability.

The stability of the embankments was also evaluated under seismic loading conditions. The minimum factor of safety value that is typically required against instability during a seismic event is 1.1. A horizontal seismic coefficient of 0.1 was used for the analyses. This value is based on the peak horizontal ground acceleration for the area of the site specified in the CHBDC (with half that value being used, per standard practice).



The results of the stability analyses indicate that, provided the final embankment slopes are maintained at no steeper than 2H:1V, the embankments will have an acceptable factor of safety of at least 1.3 and 1.1 against deep-seated rotational instability under static and seismic conditions, respectively.

### 6.9.3 Settlement

Settlement of the existing embankments has likely occurred over time since the original bridge construction. The additional loading imposed by the proposed 1.0 m grade raise would result in further consolidation settlement of the silty clay underlying the southern embankment (where encountered) and the sandy clayey silt till that underlies the site.

Settlement analyses under the raised 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, based on laboratory testing data from the original (1960) and current (2014) investigations, correlations with the SPT “N” values, and engineering judgement from experience with similar soils in this region of Ontario.

Based on the assessment, the settlement of the foundation soils under the embankments with a grade increase of 1.0 m is estimated to be a maximum of about 25 mm beneath the centreline of the embankment. Because of the nature of the sandy, clayey silt glacial till, the majority of the consolidation settlement will be completed over several years following embankment construction.

Additional settlement of the embankments will occur as a result of compression of the new grade fill and the existing embankment fill. The magnitude of compression of the new fill may range from 0.5 to 1 percent of its thickness, 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 to raise the grade, settlement of the fill itself is expected to occur essentially during embankment construction. Similarly, settlement of the existing embankment fill will be elastic in nature and should also occur during construction.

## 6.10 Construction Considerations

The following sections identify construction issues that should be considered at this stage as they may impact the detail design of the project and provisions in the Contract Documents.

### 6.10.1 Excavation and Temporary Protection Systems

The excavations for pile caps would extend a minimum of 1.7 m deep (for frost protection purposes) into the existing grade fill at the central pier location. At the abutments, the excavations for pile caps could be maintained at a higher elevation, within the approach embankments.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill would be classified as Type 3 soil, while the very stiff sandy clayey silt till would be classified as Type 2 soil, based on the OHSA. According to OHSA, excavations are classified on the basis of the weakest soil present in the excavation wall; therefore, excavations that extend through, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). It is noted that the silty clay and till below the water table would be classified as Type 4 soil, based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V; however, with appropriate groundwater control, it is anticipated that temporary excavation slopes through these materials can also be formed at 1H:1V.





If the above open-cut excavation side slopes cannot be accommodated, then a temporary protection system (i.e., temporary excavation shoring) will be required. Where shoring is required, the protection system should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

The selection and design of the protection system will be the responsibility of the Contractor. However, the following comments are provided to aid in the costing and assessment of temporary protection system options for this site:

- It is considered that either a soldier pile and lagging system or an interlocking sheetpile system would be feasible at this site. The use of an interlocking sheetpile system has an advantage over soldier pile and lagging in that it would aid in groundwater control.
- The soldier piles or sheetpiles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to a few metres. The soldier pile and lagging or sheetpiling would be supported against lateral movement using walers, tie backs and/or internal struts/braces.

#### 6.10.2 Groundwater Control

Based on the groundwater level encountered in the monitoring well installed in the northern embankment and groundwater conditions observed in the boreholes immediately following drilling, the groundwater level is expected to be about 6 m below the existing Brinston Road grade at the abutment locations and within about 1 m of the existing Highway 401 grade at the central pier location.

The excavation required for construction of the pile cap at the central pier is anticipated to extend up to about 1 m below the groundwater level for frost protection requirements. Some groundwater or surface water inflow into the excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps in the bottom of the excavations provided suitably sized pumps are used. Additional groundwater flow is expected during periods of sustained wet weather and the groundwater control methods used may need to be adjusted accordingly during and immediately following such events. Dewatering will be required to lower the groundwater level to approximately 0.5 m below the pile cap founding level during pile driving and pile cap formation. The water-bearing till at this site is relatively fine-grained (silty) and therefore will have a lower permeability. This relatively small drawdown is not expected to have an adverse impact on the existing or new structure foundations at this site.

It is estimated that less than 50,000 litres per day of water will require handling during excavation for the central pier, and that a Permit-To-Take-Water (PTTW) will not be required for construction. However, if excavations are to extend deeper than 1.7 m at the central pier location, the dewatering rate may exceed 50,000 Litres per day, and a Permit to Take Water (PTTW) would be required for this site.

Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade.

#### 6.10.3 Vibration Monitoring During Pile Driving

If the existing structure is not completely removed prior to commencement of pile driving, vibration monitoring is recommended during pile installation to assist in maintaining vibration levels within tolerable ranges for the existing portions of the bridge in close proximity to Highway 401. A Non-Standard Special Provision (NSSP) has been provided in Appendix C to address this requirement.



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## **FOUNDATION REPORT BRINSTON ROAD UNDERPASS REPLACEMENT - HIGHWAY 401**

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A maximum peak particle velocity of 100 mm/sec is recommended at the existing structure foundations. The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the installation procedures for the remaining piles.

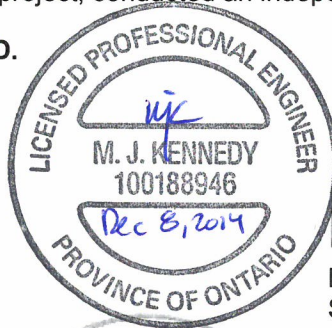


## 7.0 CLOSURE

This 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., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

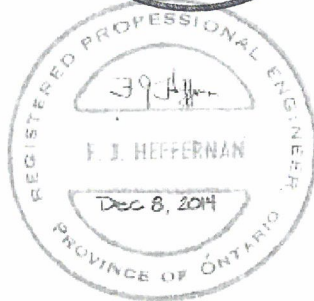
### GOLDER ASSOCIATES LTD.

Matt Kennedy, P.Eng.  
Geotechnical Engineer



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

Fintan Heffernan, P.Eng.  
Designated MTO Contact



MJK/LCC/FJH/bg

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## FOUNDATION REPORT BRINSTON ROAD UNDERPASS REPLACEMENT - HIGHWAY 401

**Table 1 – Comparison of Foundation Alternatives**

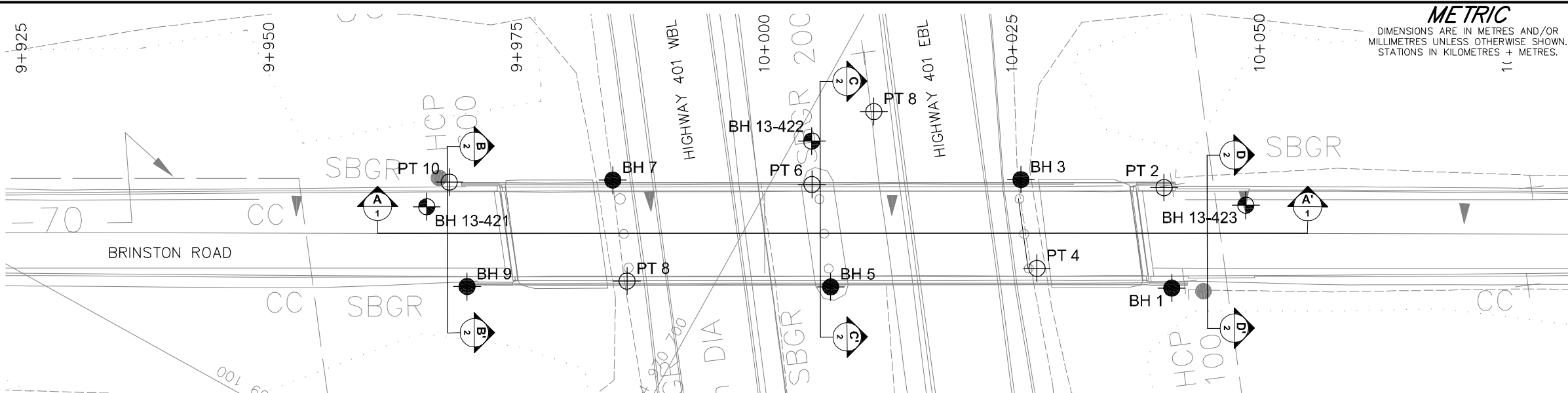
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to bedrock	<ul style="list-style-type: none"> <li>Feasible for support of bridge replacement</li> <li>Preferred option from a foundations perspective</li> </ul>	<ul style="list-style-type: none"> <li>Abutment pile caps could be within embankments, reducing depth of excavation and temporary excavation support requirements</li> <li>Higher geotechnical resistances and negligible settlement</li> <li>Less potential for interference with existing piles (vs. pipe piles)</li> <li>Preferred foundation option for integral abutment construction</li> </ul>	<ul style="list-style-type: none"> <li>Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles “hanging up” in the glacial till deposit and lower geotechnical resistances; however, this is expected to be a relatively small risk as no cobbles/boulders were encountered in the boreholes</li> <li>Temporary protection systems may be required at the central pier</li> <li>Some groundwater control would be required at the central pier</li> </ul>	<ul style="list-style-type: none"> <li>Moderate cost</li> </ul>	<ul style="list-style-type: none"> <li>Low risk of driven H-piles “hanging up” in glacial till</li> </ul>
Closed-ended steel pipe (tube) piles, driven to found in bedrock	<ul style="list-style-type: none"> <li>Feasible for support of bridge replacement</li> </ul>	<ul style="list-style-type: none"> <li>Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system</li> <li>Higher geotechnical resistances and negligible settlement</li> <li>Allows for semi-integral abutment configuration</li> </ul>	<ul style="list-style-type: none"> <li>Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving resulting in more piles “hanging up”, lower geotechnical resistances, and greater potential for interference with existing piles</li> <li>Temporary protection systems may be required at the central pier</li> <li>Some groundwater control would be required at the central pier</li> </ul>	<ul style="list-style-type: none"> <li>Moderate cost</li> </ul>	<ul style="list-style-type: none"> <li>Moderate risk of pipe piles “hanging up” in glacial till</li> </ul>



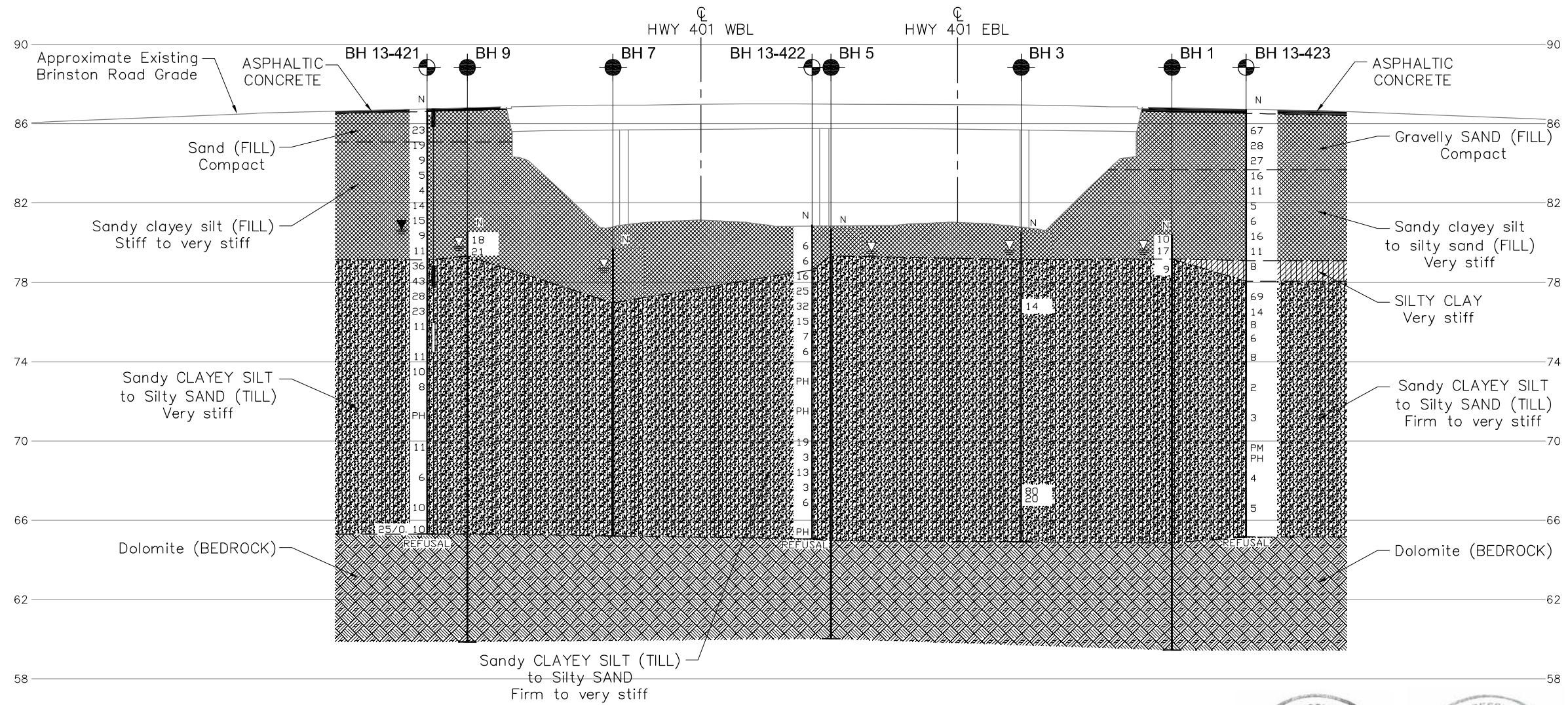
## FOUNDATION REPORT BRINSTON ROAD UNDERPASS REPLACEMENT - HIGHWAY 401

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons founded on bedrock	<ul style="list-style-type: none"><li>Feasible</li></ul>	<ul style="list-style-type: none"><li>Could eliminate the need for pile caps at the central pier, and allow for structural continuity between caissons and piers</li><li>Construction from existing grade would reduce excavation and groundwater control requirements at center pier (reduced impact on Highway 401)</li></ul>	<ul style="list-style-type: none"><li>Temporary or permanent liners required to control ground and groundwater in till deposit</li><li>Rock coring, churn drilling or chisel drilling required to form rock sockets in strong to very strong bedrock</li><li>Conflict with existing abutment piles likely, requiring removal of existing piles</li></ul>	<ul style="list-style-type: none"><li>Construction of deep caissons more expensive than alternative foundation options</li></ul>	<ul style="list-style-type: none"><li>Some risk of difficulty in removing existing abutment piles to avoid conflict with new caissons</li></ul>

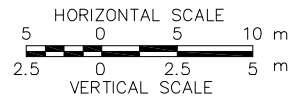




PLAN



PROFILE ALONG BRINSTON ROAD



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

CONT No. 2014-4037  
WP No. 316-01-01



HWY 401 - BRIDGE REPLACEMENT  
BRINSTON ROAD UNDERPASS  
SITE No. 31-202  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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



KEY PLAN



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (Geocres No. 31B-31)
- Probehole - Previous Investigation (Geocres No. 31B-31)
- 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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-421	86.8	4970729.2	399125.7
13-422	80.9	4970698.4	399150.3
13-423	86.7	4970656.9	399165.9
BH 1	80.4	4970659.4	399154.9
BH 3	80.5	4970678.1	399157.1
BH 5	80.7	4970689.6	399138.4
BH 7	79.7	4970714.1	399137.1
BH 9	80.6	4970721.7	399120.6

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 design configuration as shown elsewhere in the Contract 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.

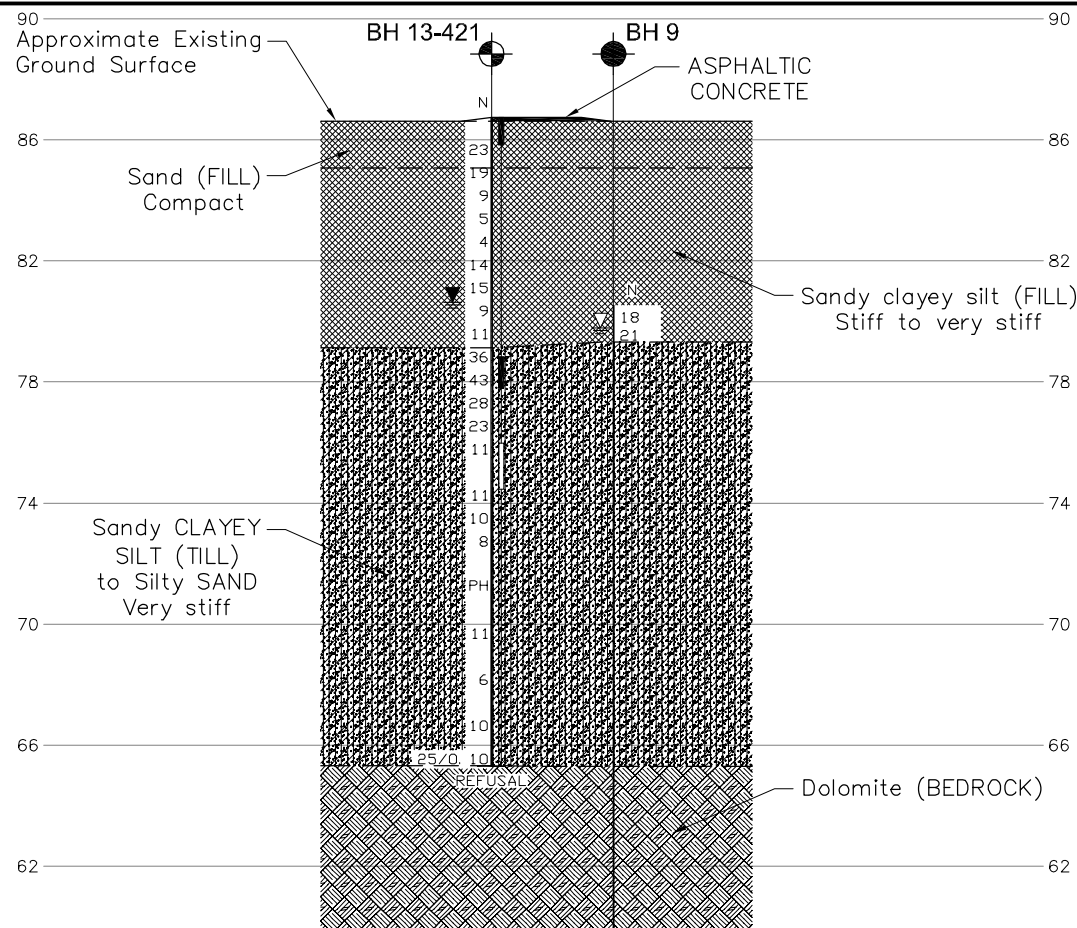
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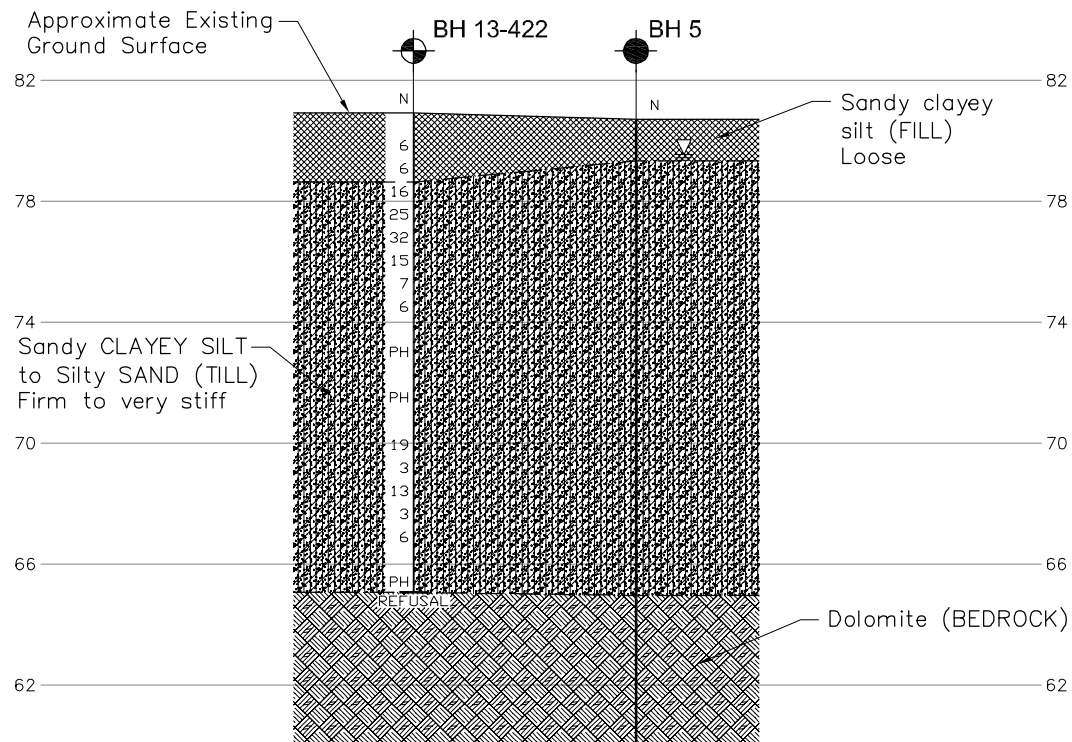
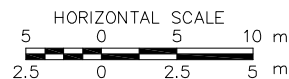
NO.	DATE	BY	REVISION
Geocres No. 31B-85			
HWY. 401			PROJECT NO. 12-1121-0099
SUBM'D. MJK	CHKD. FJH	DATE: Feb. 2014	SITE: 31-202
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG.1



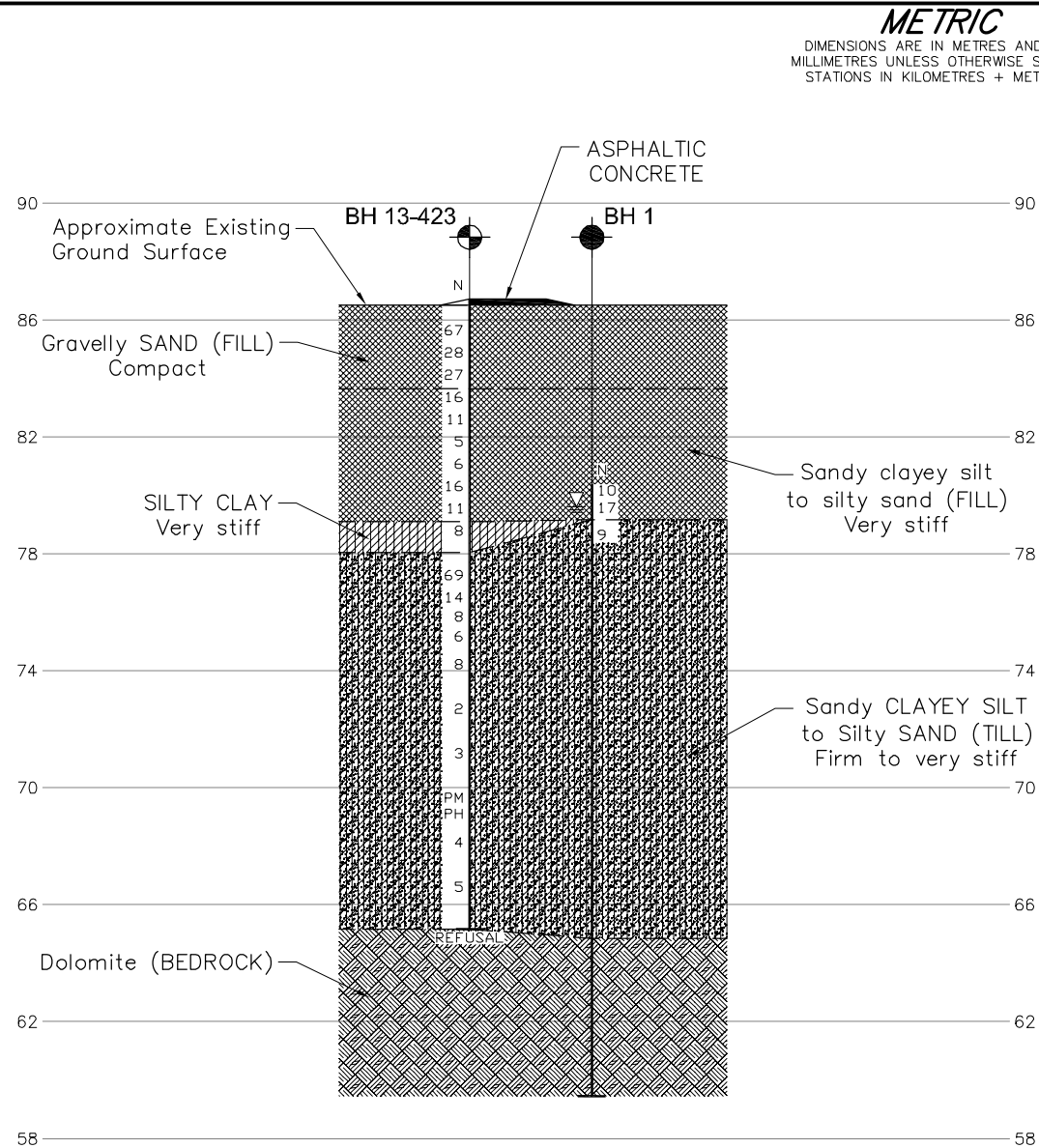
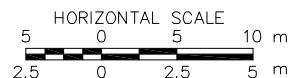




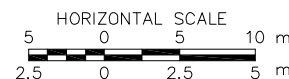
**CROSS-SECTION B-B' – NORTH ABUTMENT**



**CROSS-SECTION C-C' – CENTRE PIER**



**CROSS-SECTION D-D' – SOUTH ABUTMENT**



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

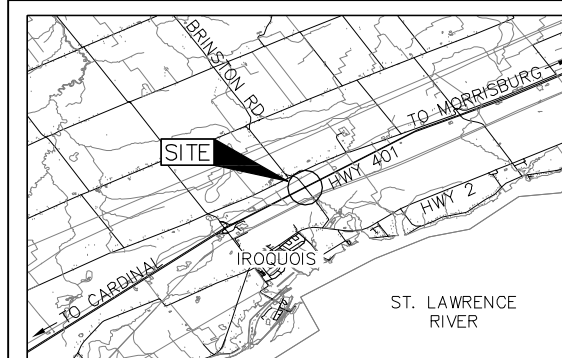
CONT No. 2014-4037  
WP No. 316-01-01

HWY 401 – BRIDGE REPLACEMENT  
BRINSTON ROAD UNDERPASS  
SITE No. 31-202  
BOREHOLE LOCATIONS AND SOIL STRATA

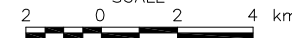
SHEET



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



KEY PLAN  
SCALE



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NO.	DATE	BY	REVISION
Geocres No. 31B-85			
HWY. 401			PROJECT NO. 12-1121-0099
SUBM'D: MJK	CHKD: FJH	DATE: Feb. 2014	SITE: 31-202
DRAWN: JM	CHKD: MJK	APPD: FJH	DWG: 2





# **APPENDIX A**

## **Borehole Records, Current Investigation (2014)**



## LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE		III. SOIL DESCRIPTION		
AS	Auger sample	(a) Cohesionless Soils		
BS	Block sample	<b>Density Index (Relative Density)</b>		<b>N</b>
CS	Chunk sample			<u>Blows/300 mm</u>
DO or DP	Seamless open-ended, driven or pushed tube samplers			<u>Or Blows/ft.</u>
DS	Denison type sample		Very loose	0 to 4
FS	Foil sample		Loose	4 to 10
RC	Rock core		Compact	10 to 30
SC	Soil core		Dense	30 to 50
SS	Split spoon sampler		Very dense	over 50
ST	Slotted tube	(b) Cohesive Soils		
TO	Thin-walled, open	<b>C<sub>u</sub> or S<sub>u</sub></b>		
TP	Thin-walled, piston	<b>Consistency</b>		
WS	Wash sample		<u>kPa</u>	<u>Psf</u>
DT	Dual tube sample		Very soft	0 to 12
DD	Diamond drilling		Soft	12 to 25
			Firm	25 to 50
			Stiff	50 to 100
			Very stiff	100 to 200
			Hard	Over 200
				Over 4,000
II. PENETRATION RESISTANCE		IV. SOIL TESTS		
<b>Standard Penetration Resistance (SPT), N:</b>		w	Water content	
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.).		w <sub>p</sub> or PL	Plastic limited	
		w <sub>l</sub> or LL	Liquid limit	
		C	Consolidation (oedometer) test	
		CHEM	Chemical analysis (refer to text)	
		CID	Consolidated isotropically drained triaxial test <sup>1</sup>	
		CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>	
<b>Dynamic Cone Penetration Resistance (DCPT); N<sub>d</sub>:</b>		D <sub>R</sub>	Relative density	
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).		DS	Direct shear test	
		G <sub>s</sub>	Specific gravity	
		M	Sieve analysis for particle size	
		MH	Combined sieve and hydrometer (H) analysis	
		MPC	Modified Proctor compaction test	
		SPC	Standard Proctor compaction test	
		OC	Organic content test	
		SO <sub>4</sub>	Concentration of water-soluble sulphates	
		UC	Unconfined compression test	
		UU	Unconsolidated undrained triaxial test	
		V	Field vane test (LV-laboratory vane test)	
		γ	Unit weight	
<b>Cone Penetration Test (CPT):</b>		Note:	<sup>1</sup> Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.	
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm <sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q <sub>t</sub> ), porewater pressure (u) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.				

## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	3.1416
$\ln x$	natural logarithm of x
$\log_{10} x$ or $\log x$	logarithm of x to base 10
$g$	acceleration due to gravity
$t$	time
FOS	factor of safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma'$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial vertical effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3) / 3$
$\tau$	shear stress
$u$	porewater pressure
$E$	modulus of deformation
$G$	shear modulus of deformation
$K$	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) formerly ( $G_s$ )
$e$	void ratio
$n$	porosity
$S$	degree of saturation
*	Density symbol is $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

#### (a) Index Properties (continued)

$w$	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity Index $= (w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index $= (w - w_p) / I_p$
$I_c$	consistency index $= (w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (overconsolidated range)
$C_s$	swelling index
$C_\alpha$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation (vertical direction)
$T_v$	time factor (vertical direction)
$U$	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p$ or $\tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u$ or $s_u$	undrained shear strength ( $\phi = 0$ analysis)
$p$	mean total stress $(\sigma_1 + \sigma_3) / 2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3) / 2$
$q$	$(\sigma_1 - \sigma_3) / 2$ or $(\sigma'_1 - \sigma'_3) / 2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

Notes:

$$^1 \tau = c' + \sigma' \tan \phi'$$

$$^2 \text{ shear strength} = (\text{compressive strength}) / 2$$

# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING STATE

**Fresh:** no visible sign of rock material weathering

**Faintly Weathered:** weathering limited to the surface of major discontinuities.

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

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

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

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

## BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very Thickly Bedded	> 2 m
Thickly Bedded	0.6 m to 2m
Medium Bedded	0.2 m to 0.6 m
Thinly Bedded	60 mm to 0.2 m
Very Thinly Bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly Laminated	< 6 mm

## JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very Wide	> 3 m
Wide	1 – 3 m
Moderately Close	0.3 – 1 m
Close	50 – 300 mm
Very Close	< 50 mm

## GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

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

## CORE CONDITION

### Total Core Recovery

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

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

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




### Description and Notes

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

### Abbreviations

BD -	Bedding	PY -	Pyrite
FO -	Foliation/Schistosity	Ca -	Calcite
CL -	Clean	PO -	Polished
SH -	Shear Plane/Zone	K -	Slickensided
VN -	Vein	SM -	Smooth
FLT -	Fault	RO -	Ridged/Rough
CO -	Contact	ST -	Stepped
JN -	Joint	PL -	Planar
FR -	Fracture	IR -	Irregular
MB -	Mechanical Break	UN -	Undulating
BR -	Broken Rock	CU -	Curved
BL -	Blast Induced	TCA -	To Core Axis
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

PROJECT <u>12-1121-0099-1420</u>		<b>RECORD OF BOREHOLE No 13-421</b>		SHEET 1 OF 3		<b>METRIC</b>	
G.W.P. <u>316-01-01</u>		LOCATION <u>N 4970729.2 ; E 399125.7</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>January 8-9, 2014</u>		CHECKED BY <u>MJK</u>			



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 (%)									
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>							
86.8	GROUND SURFACE																					
0.0	ASPHALTIC CONCRETE																					
0.3	Gravel (FILL)																					
	Grey																					
	Sand, some silt (FILL)																					
	Compact																					
85.1 1.7	Brown																					
	Moist																					
			1	SS	23													0	82	16	2	
			2	SS	19																	
			3	SS	9														5	31	42	22
			4	SS	5																	
79.2 7.6																						
			5	SS	4																	
	6	SS	14																			

Continued Next Page

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/26/14 JM



PROJECT		12-1121-0099-1420		RECORD OF BOREHOLE No 13-421		SHEET 3 OF 3		METRIC								
G.W.P.		316-01-01		LOCATION		N 4970729.2 ; E 399125.7		ORIGINATED BY								
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY								
DATUM		Geodetic		DATE		January 8-9, 2014		CHECKED BY								
JMK																
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
							20	40	60	80	100	25	50	75	kN/m <sup>3</sup>	GR SA SI CL
65.4	Sandy CLAYEY SILT to Silty SAND, trace to some gravel (TILL) Very stiff Grey Wet		21	SS	10											
21.4	END OF BOREHOLE SAMPLER REFUSAL  NOTES:  1. Water level in well at a depth of 6.2 m below ground surface (Elev. 80.6 m), measured on February 19, 2014.  1. Water level in well at a depth of 7.0 m below ground surface (Elev. 79.8 m), measured on August 21, 2014.		22	SS	25/0.10											



PROJECT <u>12-1121-0099-1420</u>		<b>RECORD OF BOREHOLE No 13-422</b>		SHEET 2 OF 2		<b>METRIC</b>	
G.W.P. <u>316-01-01</u>		LOCATION <u>N 4970698.4 ; E 399150.3</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>January 13, 2014</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT	UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					GR	SA	SI	CL	
								20	40	60	80	100	25	50	75							
	--- CONTINUED FROM PREVIOUS PAGE ---																					
	Sandy CLAYEY SILT to Silty SAND, trace to some gravel (TILL) Firm to very stiff Grey Wet																					
			11	SS	19																	
			12	SS	3																	
			13	SS	13																	
			14	SS	3																	
			15	SS	6																	
			16	TP	PH																	
65.1																						
15.9	END OF BOREHOLE AUGER REFUSAL																					

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



PROJECT <u>12-1121-0099-1420</u>		<b>RECORD OF BOREHOLE No 13-423</b>		SHEET 1 OF 3		<b>METRIC</b>	
G.W.P. <u>316-01-01</u>		LOCATION <u>N 4970656.9 ; E 399165.9</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>January 9-10, 2014</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   LIQUID CONTENT   LIMIT			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	
								20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>						
86.7	GROUND SURFACE																				
0.0 86.5	ASPHALTIC CONCRETE																				
86.3	Crushed rock (BASE)																				
0.4	Gravel and sand, some silt (FILL) Very dense Brown Moist		1	SS	67							○					37	45	15	3	
85.2																					
1.5	Gravelly sand, some silt, trace clayey silt layers (FILL) Compact Brown Moist		2	SS	28																
			3	SS	27							○						23	59	16	2
83.7																					
3.1	Sandy clayey silt to silty sand, trace to some gravel (FILL) Very stiff Brown to grey Moist		4	SS	16																
			5	SS	11																
			6	SS	5																
			7	SS	6							○						21	30	36	13
			8	SS	16																
			9	SS	11																
79.1																					
7.6	SILTY CLAY, trace sand Very stiff Brown to grey Moist		10	SS	8								○					0	7	39	54
78.0																					
8.7	Sandy SILT, some clay, trace gravel (TILL) Very stiff Brown to grey Moist to wet		11	SS	69																
76.8																					

Continued Next Page

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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/26/14 JM

Continued Next Page

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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/26/14 JM

PROJECT <u>12-1121-0099-1420</u>		<b>RECORD OF BOREHOLE No 13-423</b>		SHEET 3 OF 3		<b>METRIC</b>	
G.W.P. <u>316-01-01</u>		LOCATION <u>N 4970656.9 ; E 399165.9</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>January 9-10, 2014</u>		CHECKED BY <u>MJK</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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>					
	--- CONTINUED FROM PREVIOUS PAGE ---																			
65.1	Sandy CLAYEY SILT to Silty SAND, some clay, some sand to sandy, trace to some gravel (TILL) Firm Grey Wet		21	SS	5															
21.6	END OF BOREHOLE AUGER REFUSAL																			

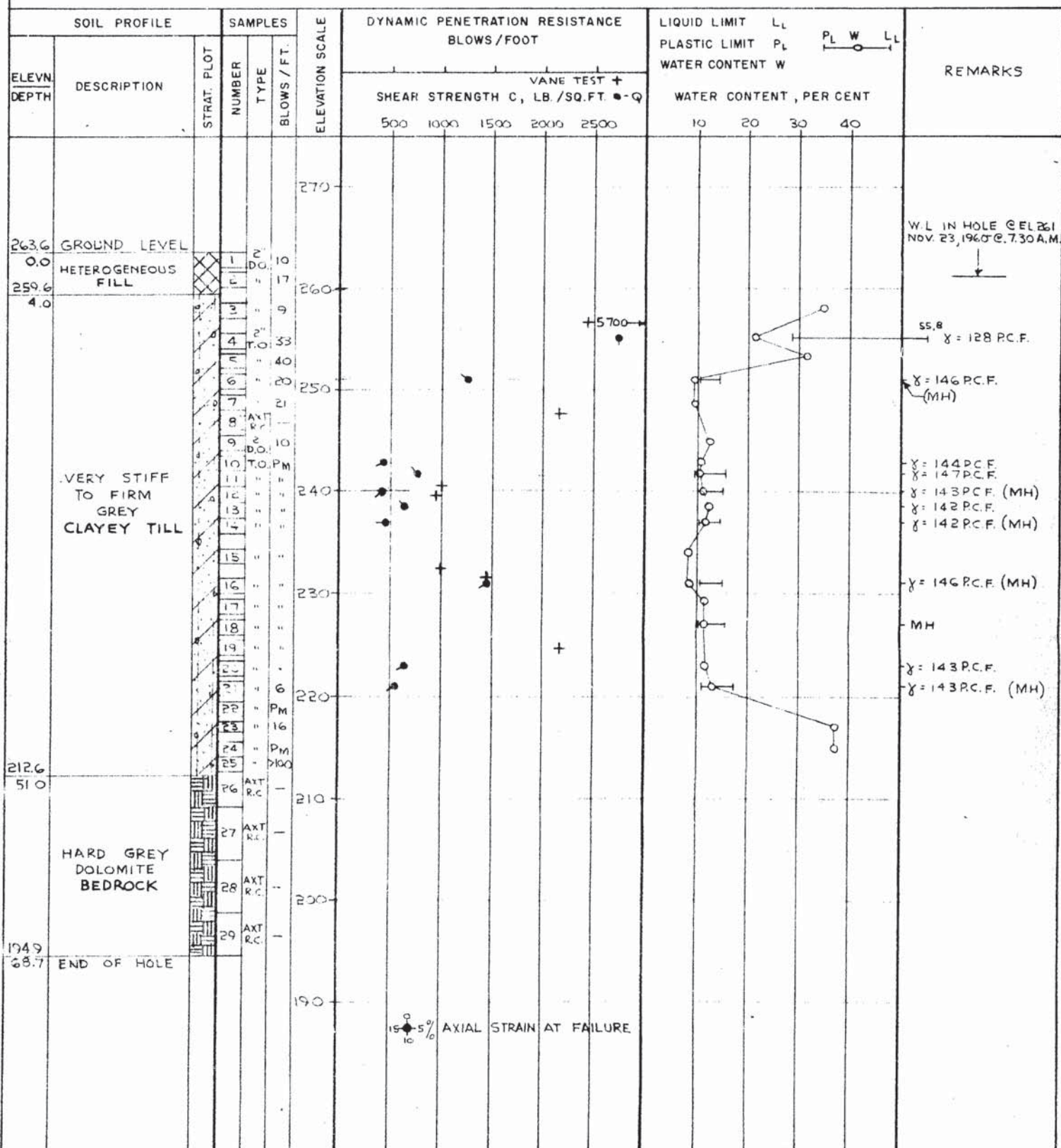


# **APPENDIX B**

## **Borehole Records, Previous Investigation (1960)**

## RECORD OF BOREHOLE 1

LOCATION SEE FIGURE 5 BORING DATE NOV. 17-22, 1960 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER Bx CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



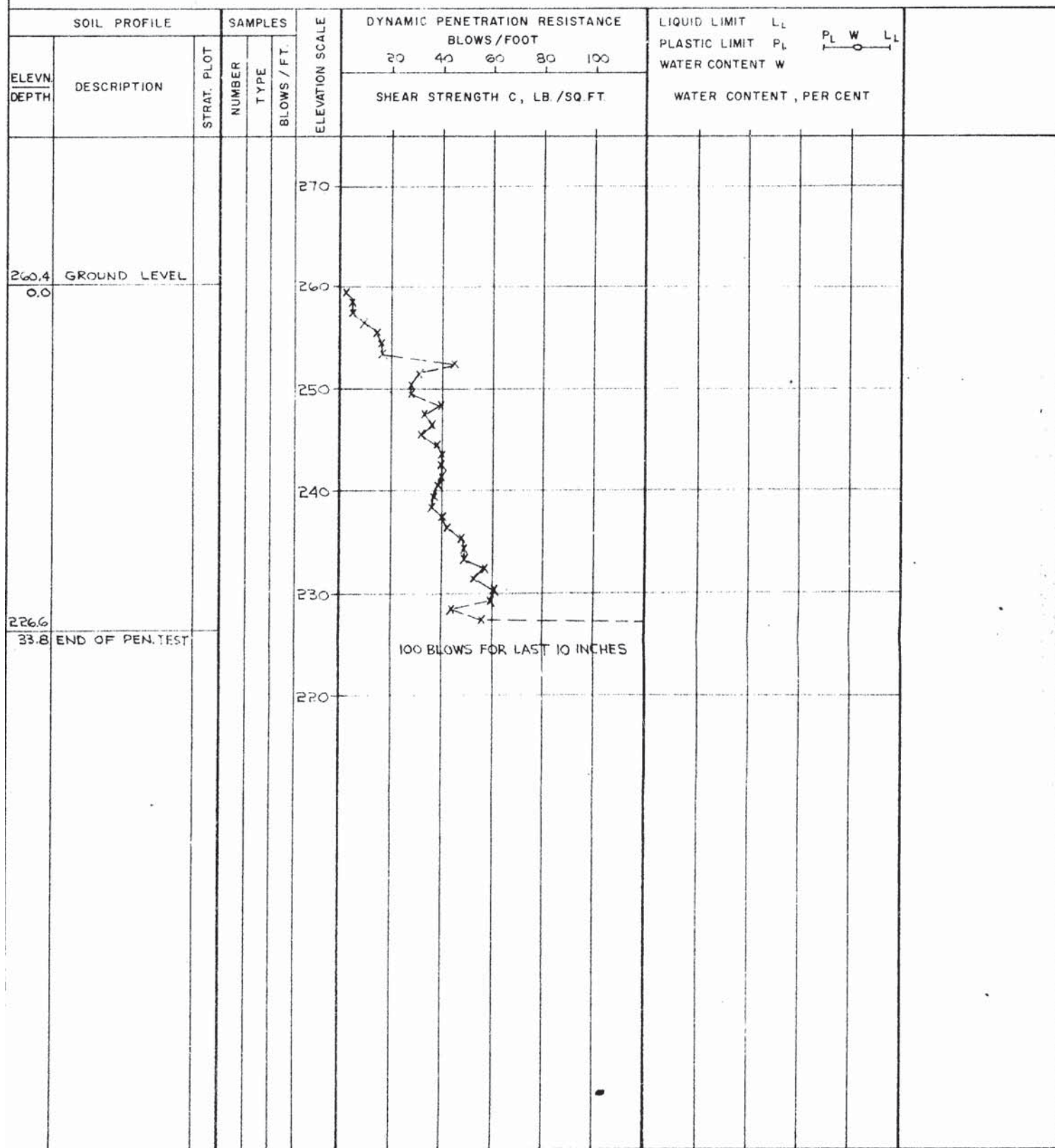
VERTICAL SCALE  
1 INCH TO 10 FEET

GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED S.G.S

# RECORD OF BOREHOLE 2

LOCATION SEE FIGURE 5 BORING DATE NOV. 23, 1960 DATUM GEODETIC  
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —  
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.  
 CHECKED S.G.S.



## RECORD OF BOREHOLE 3

LOCATION SEE FIGURE 5

BORING DATE

Nov. 24-25, 1960

DATUM

GEODETIC

BOREHOLE TYPE



















WASH BORING

BOREHOLE DIAMETER

BX CASING

SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES

PEN TEST HAMMER WEIGHT — LB. DROP — INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT				LIQUID LIMIT $L_L$ PLASTIC LIMIT $P_L$ WATER CONTENT $W$			REMARKS
ELEVATION DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE		SHEAR STRENGTH $C$ , LB./SQ.FT.				WATER CONTENT, PER CENT			
264.0	GROUND LEVEL											W.L. IN CASING @ EL 261 NOV. 25, 1960 @ 7.30 A.M. <div>↓</div>
0.0	HETEROGENEOUS FILL											
259.5			1 2" T.O.	18								
4.5			2 " "									
			3 4 D.O.	22								
			4 " "	14								
			5 T.O. PM									
			6 " "									
			7 " "									
	VERY STIFF TO FIRM GREY CLAYEY TILL		8 " "									
			9 " "									
			10 " "									
			11 " "									
			12 " "	11								
			13 " "									
			14 D.O.	8								
			15 T.O.	20								
			16 " "	16								
212.8			17 " "	7								
51.2	END OF HOLE											

 VERTICAL SCALE  
 1 INCH TO 10 FEET

GOLDER &amp; ASSOCIATES

 DRAWN J.A.  
 CHECKED S.G.S.





# RECORD OF BOREHOLE 5

LOCATION SEE FIGURE 5 BORING DATE NOV. 28-30, 1960 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES PEN. TEST HAMMER WEIGHT LB DROP INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT $L_L$ PLASTIC LIMIT $P_L$ $W$ $L_L$ WATER CONTENT $W$			REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER		TYPE	VANE TEST + SHEAR STRENGTH $C$ , LB /SQ.FT					WATER CONTENT, PER CENT			
						500	1000	1500	2000	2500				
264.6	GROUND LEVEL													
0.0	HETEROGENEOUS FILL													W.L. IN HOLE @ E.L. 260.8 DEC 3, 1960 @ 7.30 A.M. <div>↓</div>
260.1			1	T.O.	20									
4.5			2	"										
			3	"	16									
			4	"	7									
	VERY STIFF TO FIRM GREY CLAYEY TILL		5	"	5									
			6	"	3									
			7	"	29									
			8	"	13									
			9	"	11									
			10	"	9									
213.0			11	AXT R.C.										
51.6			12	"										
	HARD GREY DOLOMITE BEDROCK		13	"										
196.8														
67.8	END OF HOLE													

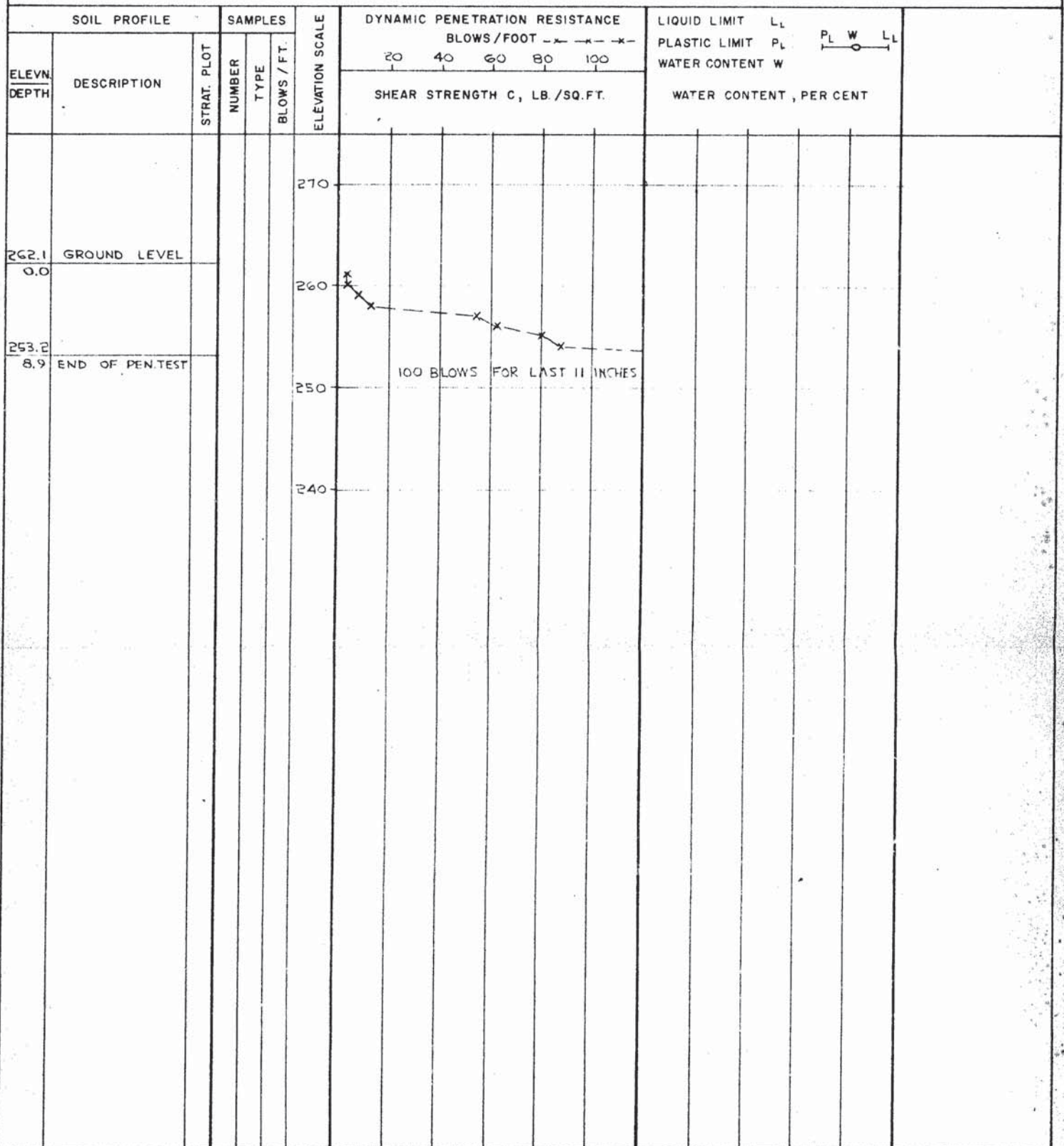
VERTICAL SCALE  
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.  
CHECKED S.S.S.

## RECORD OF BOREHOLE 6

LOCATION SEE FIGURE 5 BORING DATE NOV. 25, 1960 DATUM GEODETIC  
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —  
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
1 INCH TO

GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED S.G.S.



## RECORD OF BOREHOLE 7

LOCATION SEE FIGURE 5

BORING DATE

NOV. 26-28, 1960

DATUM

GEODETIC

BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER

8X CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT				LIQUID LIMIT $L_L$ PLASTIC LIMIT $P_L$ WATER CONTENT $W$			REMARKS
ELEV N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		SHEAR STRENGTH $C$ , LB./SQ.FT.				WATER CONTENT, PER CENT			
261.3 0.0	GROUND LEVEL		1	2" T.O.	53								W.L. IN HOLE @ EL. 260.3 NOV. 29, 1960 @ 7.30 A.M. 
252.3 9.0	HETEROGENEOUS FILL		2	"	24								
			3	"	8								
			4	"	8								
			5	"	7								
			6	"	6								
			7	"	32								
			8	"	14								
			9	"	8								
213.8 47.5	END OF HOLE		10	"									

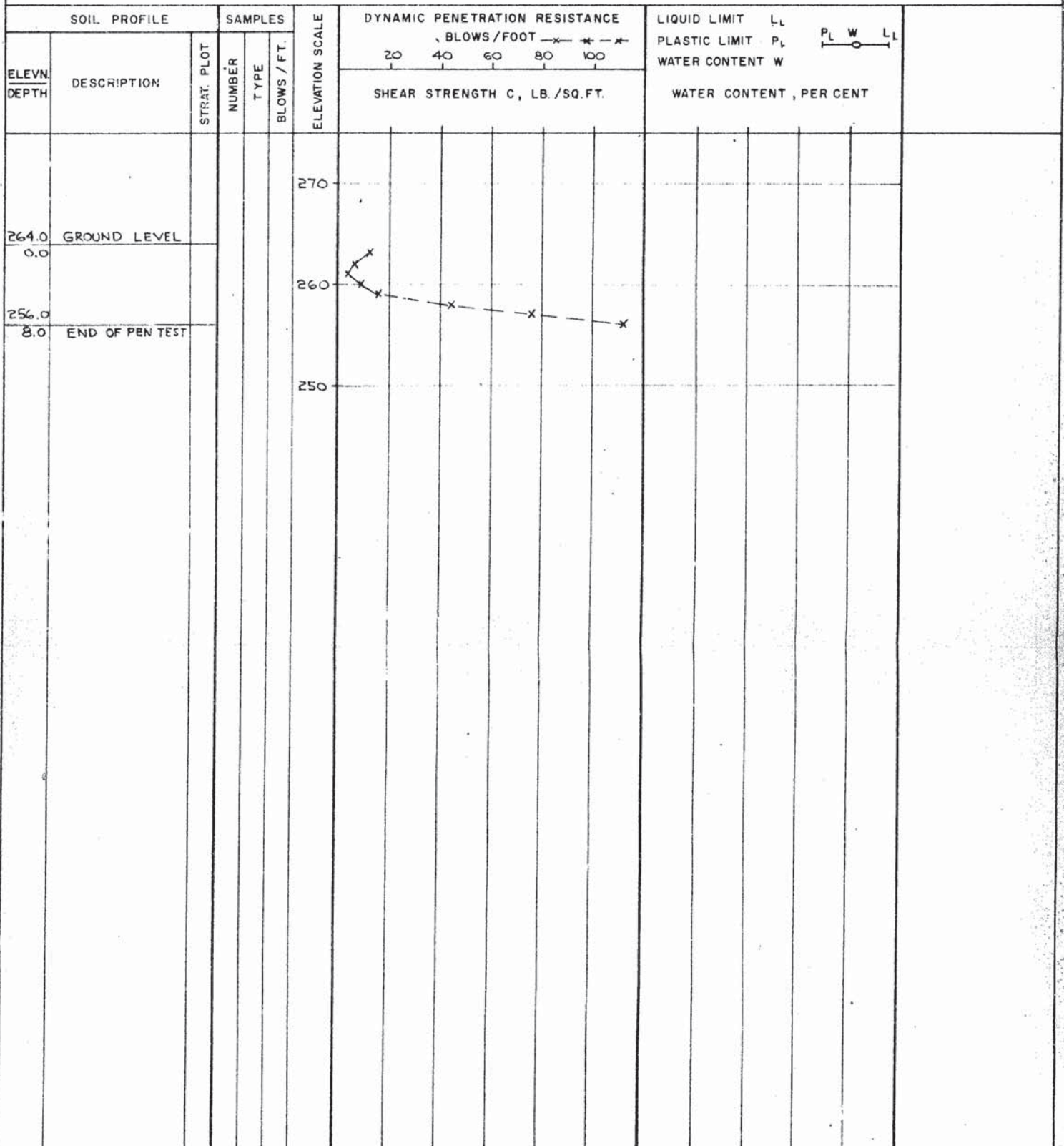
 VERTICAL SCALE  
 1 INCH TO 10 FEET

GOLDER &amp; ASSOCIATES

 DRAWN J.A.  
 CHECKED S.G.S.

## RECORD OF BOREHOLE 8

LOCATION SEE FIGURE 5 BORING DATE DEC. 8, 1960 DATUM GEODETIC  
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —  
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
1 INCH TO 10 FEET

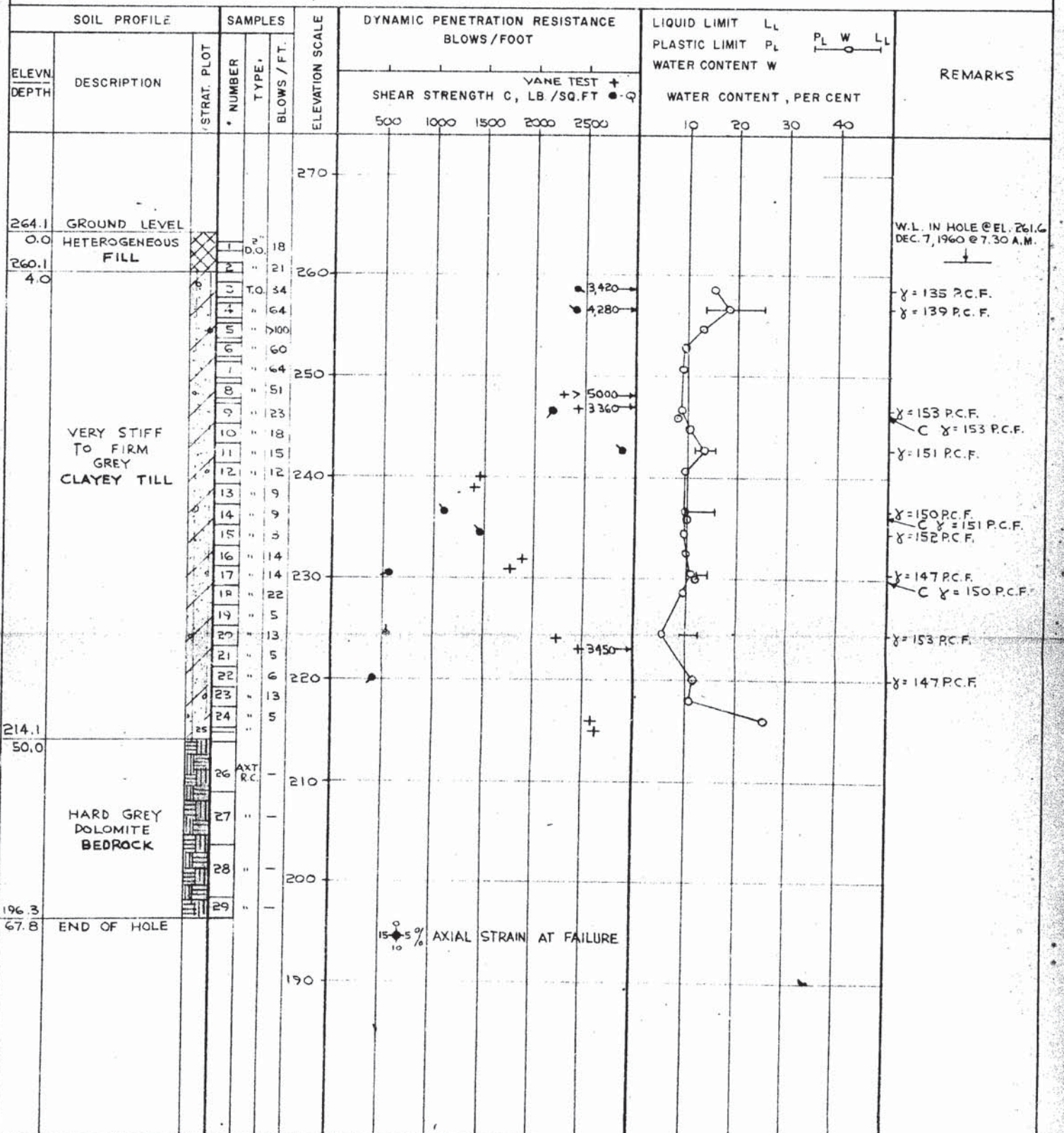
GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED S.G.S.



# RECORD OF BOREHOLE 9

LOCATION SEE FIGURE 5 BORING DATE DEC. 2-6, 1960 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



VERTICAL SCALE  
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J A  
 CHECKED S. G. S.





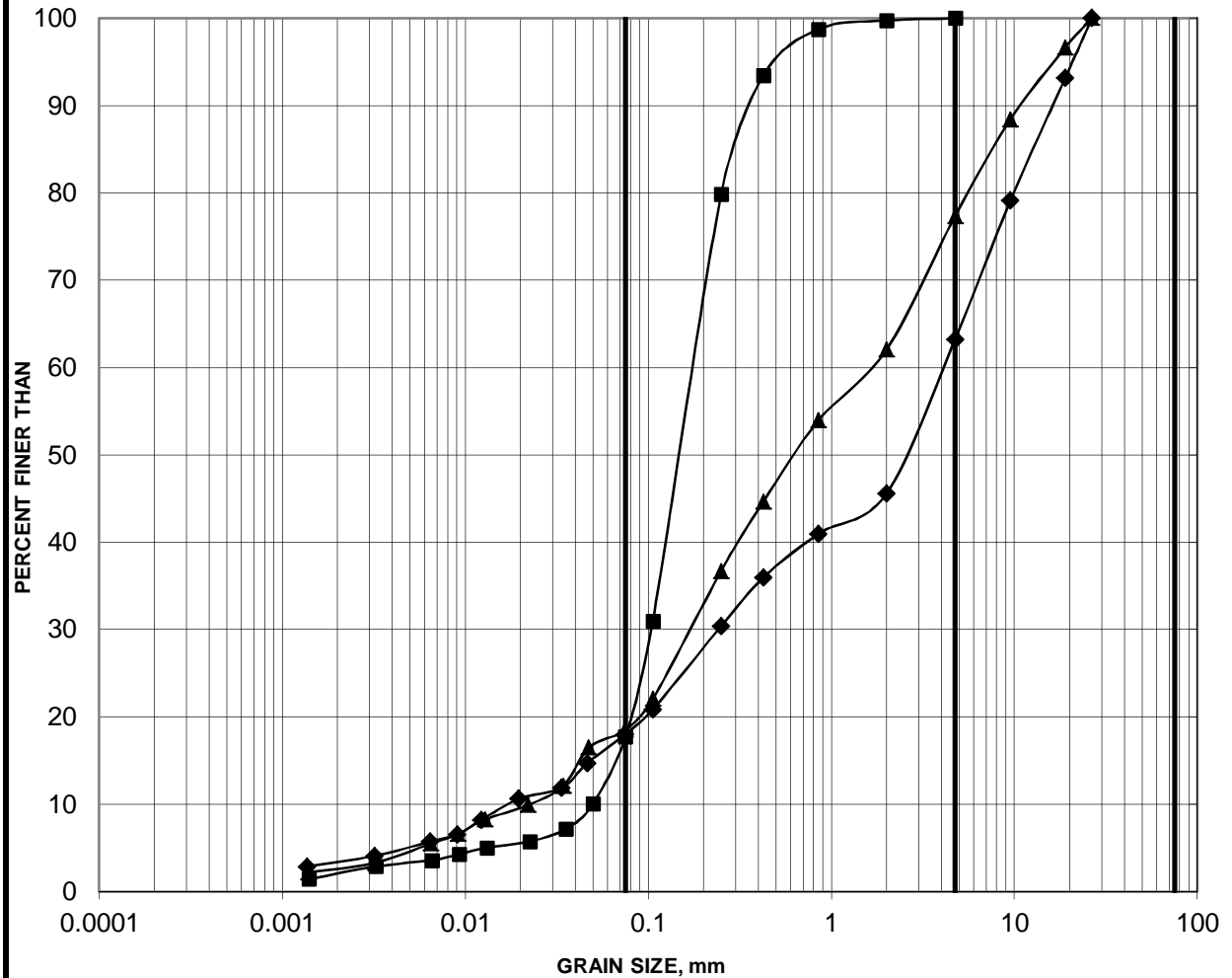
# **APPENDIX C**

## **Laboratory Test Results, Current Investigation (2014)**

# GRAIN SIZE DISTRIBUTION

FIGURE C1

## SAND and GRAVEL (EMBANKMENT FILL)



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

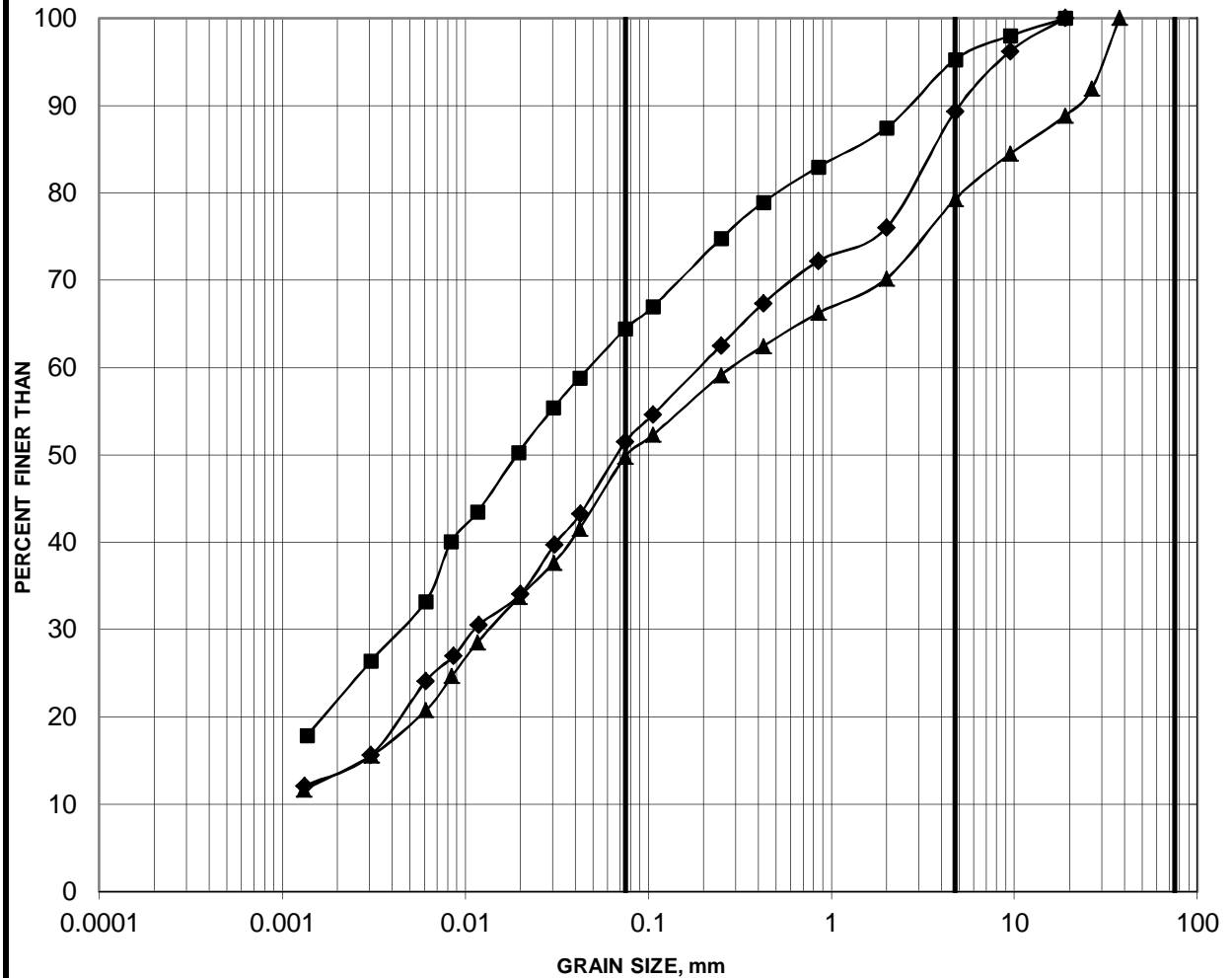
Borehole	Sample	Depth (m)
■ 13-421	1	0.76-1.37
◆ 13-423	1	0.76-1.37
▲ 13-423	3	2.29-2.90



# GRAIN SIZE DISTRIBUTION

FIGURE C2

## Sandy CLAYEY SILT (EMBANKMENT FILL)



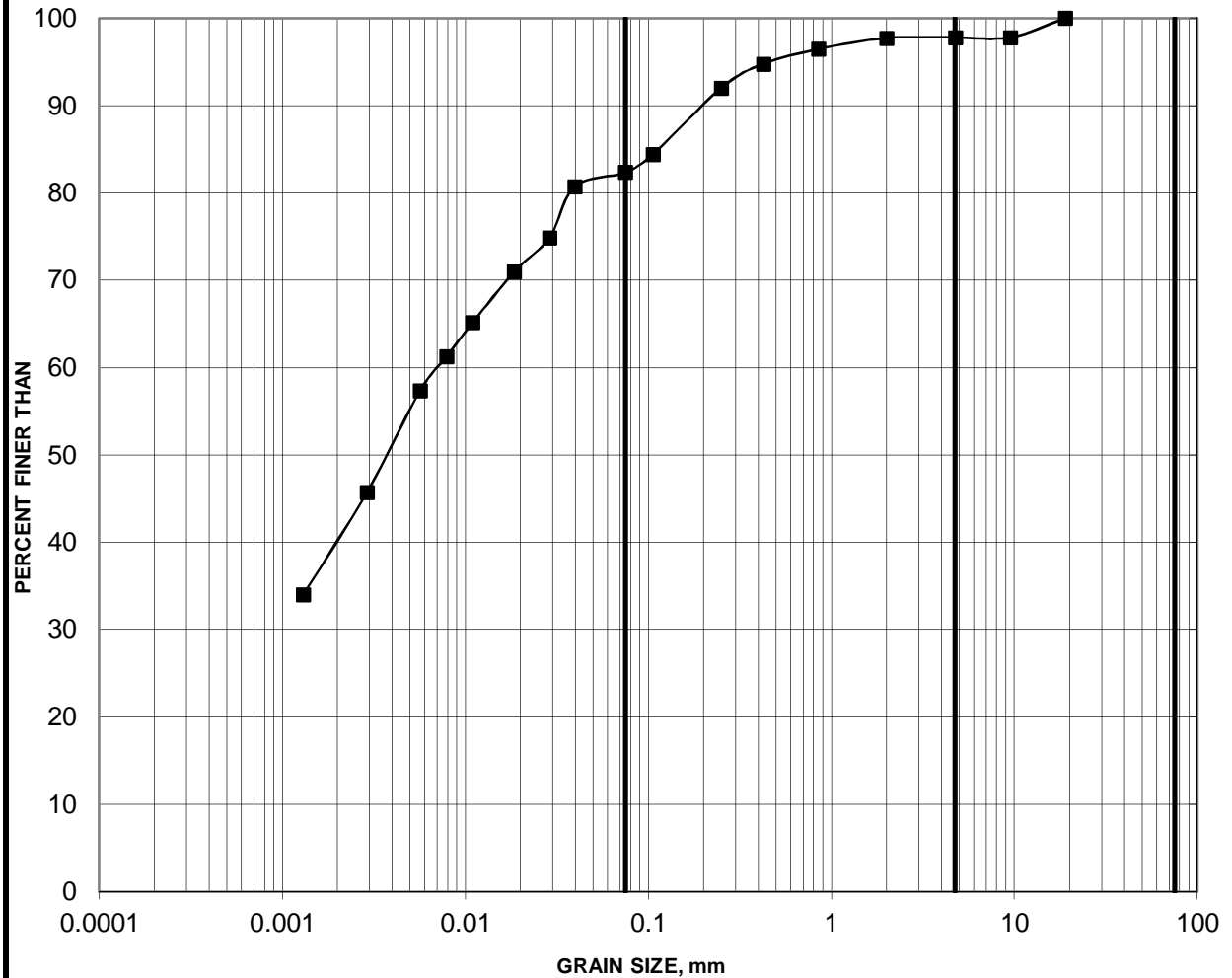
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
13-421	3	2.29-2.90
13-421	7	5.33-5.94
13-423	7	5.33-5.94

# GRAIN SIZE DISTRIBUTION

FIGURE C3

## CLAYEY SILT, some sand (MEDIAN FILL)

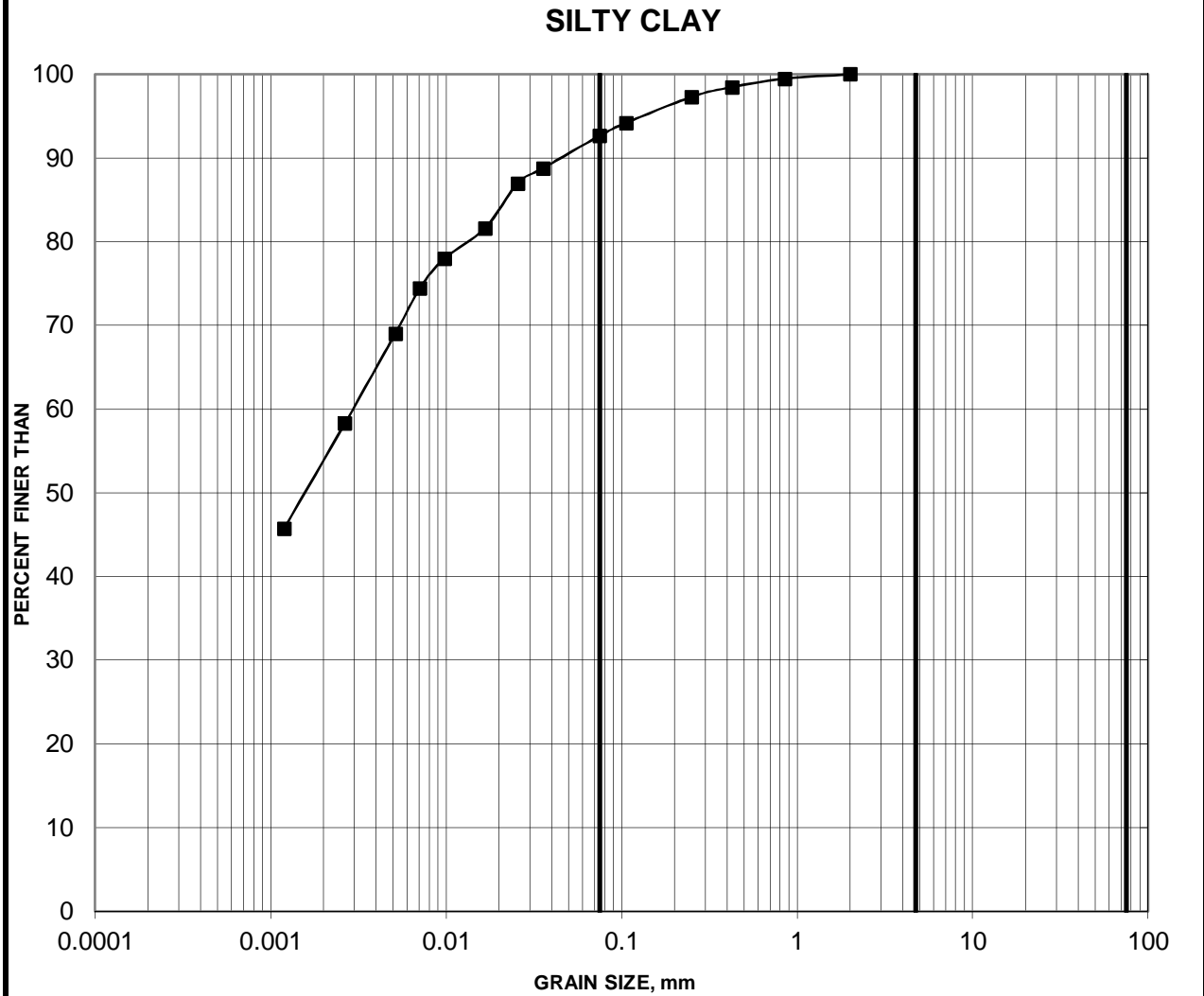


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■ 13-422	1	0.76-1.37

# GRAIN SIZE DISTRIBUTION

FIGURE C4



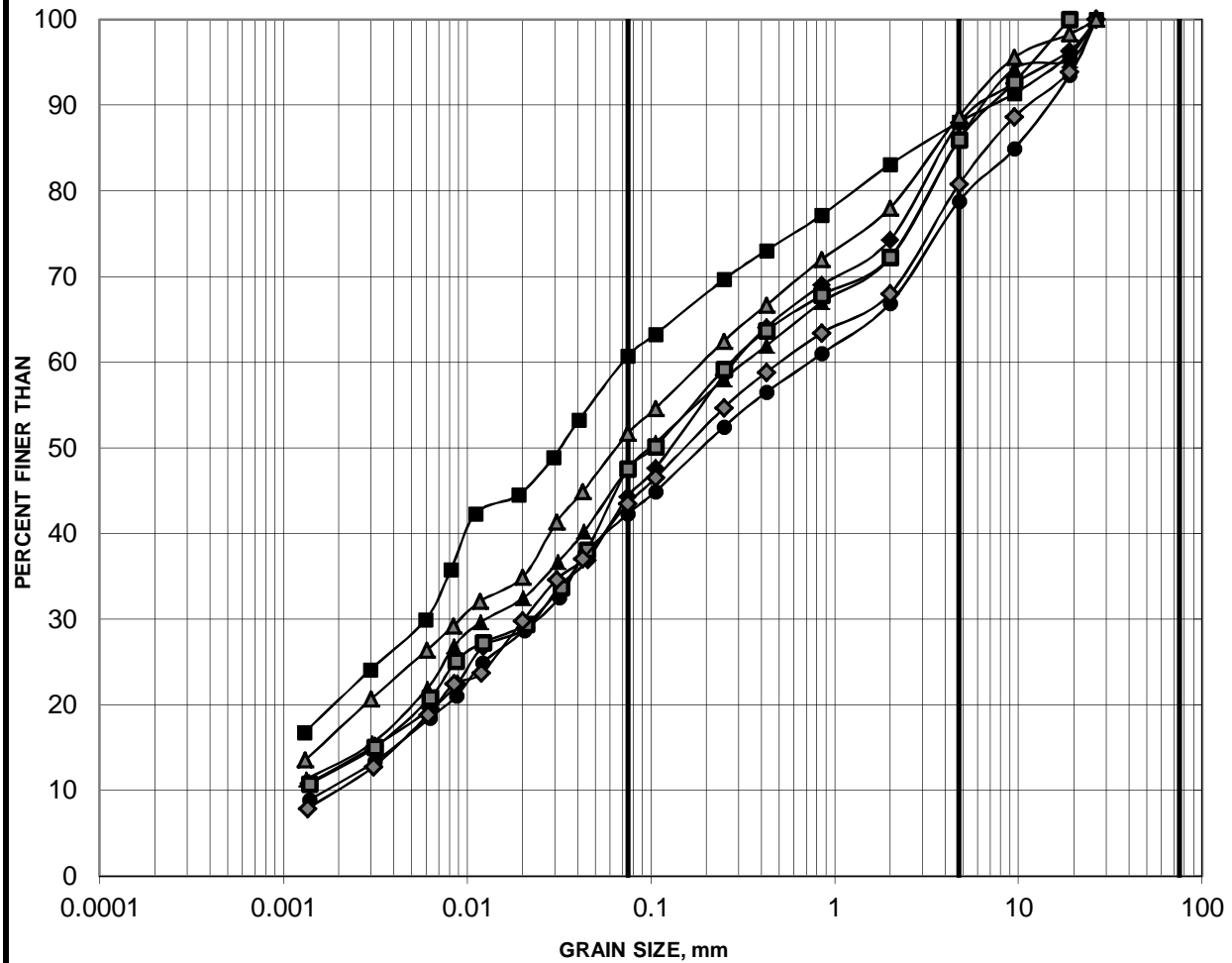
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■ 13-423	10	7.62-8.23

# GRAIN SIZE DISTRIBUTION

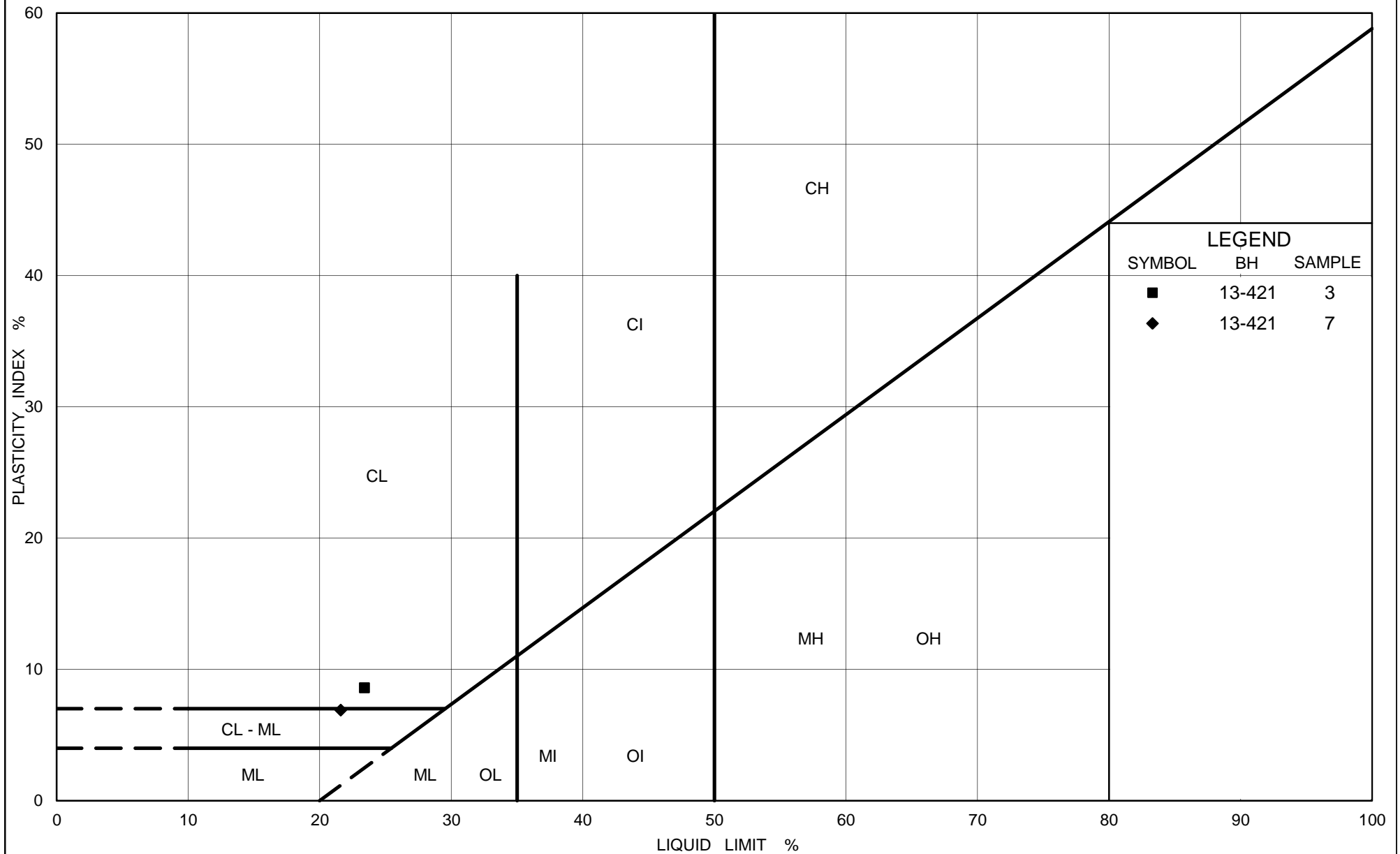
FIGURE C5

## Sandy CLAYEY SILT to silty SAND (TILL)



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

Borehole	Sample	Depth (m)
13-421	11	8.38-8.99
13-421	20	18.29-18.90
13-422	5	3.81-4.42
13-422	9	7.62-8.13
13-422	14	12.95-13.56
13-423	12	9.91-10.52
13-423	16	13.72-14.33



Ontario

Ministry of Transportation

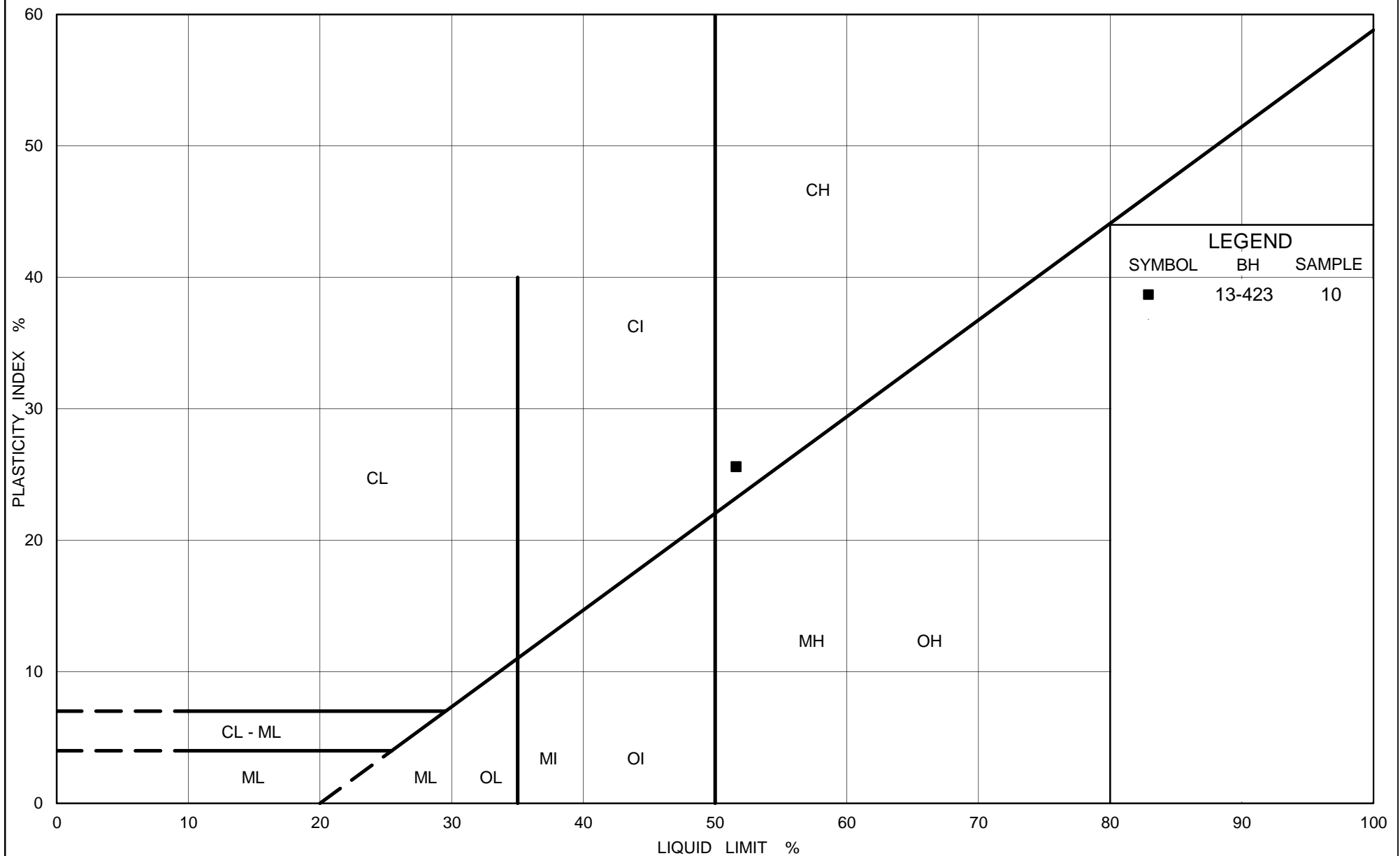
# PLASTICITY CHART

## Sandy CLAYEY SILT (EMBANKMENT FILL)

FIG No. C6

Project No. 12-1121-0099/1420

Compiled By : CW Checked By : CNM



Ministry of Transportation

Ontario

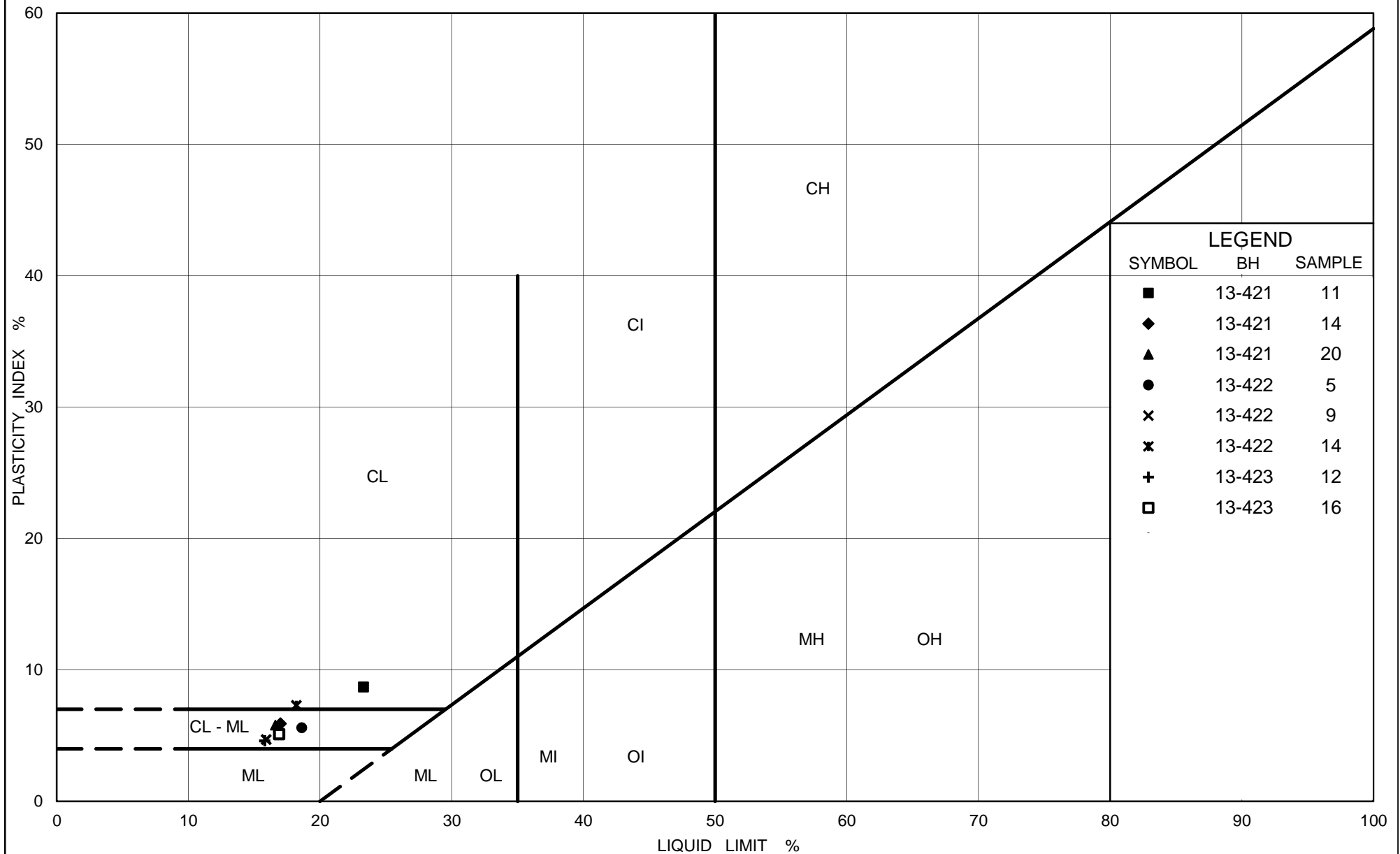
## PLASTICITY CHART

### Silty Clay

FIG No. C7

Project No. 12-1121-0099/1420

Compiled By : CW    Checked By : CNM



Ontario

Ministry of Transportation

# PLASTICITY CHART

## Sandy CLAYEY SILT to silty SAND (TILL)

FIG No. C8

Project No. 12-1121-0099/1420

Compiled By : CW Checked By : CNM



# **APPENDIX D**

## **Laboratory Test Results, Previous Investigation (1960)**

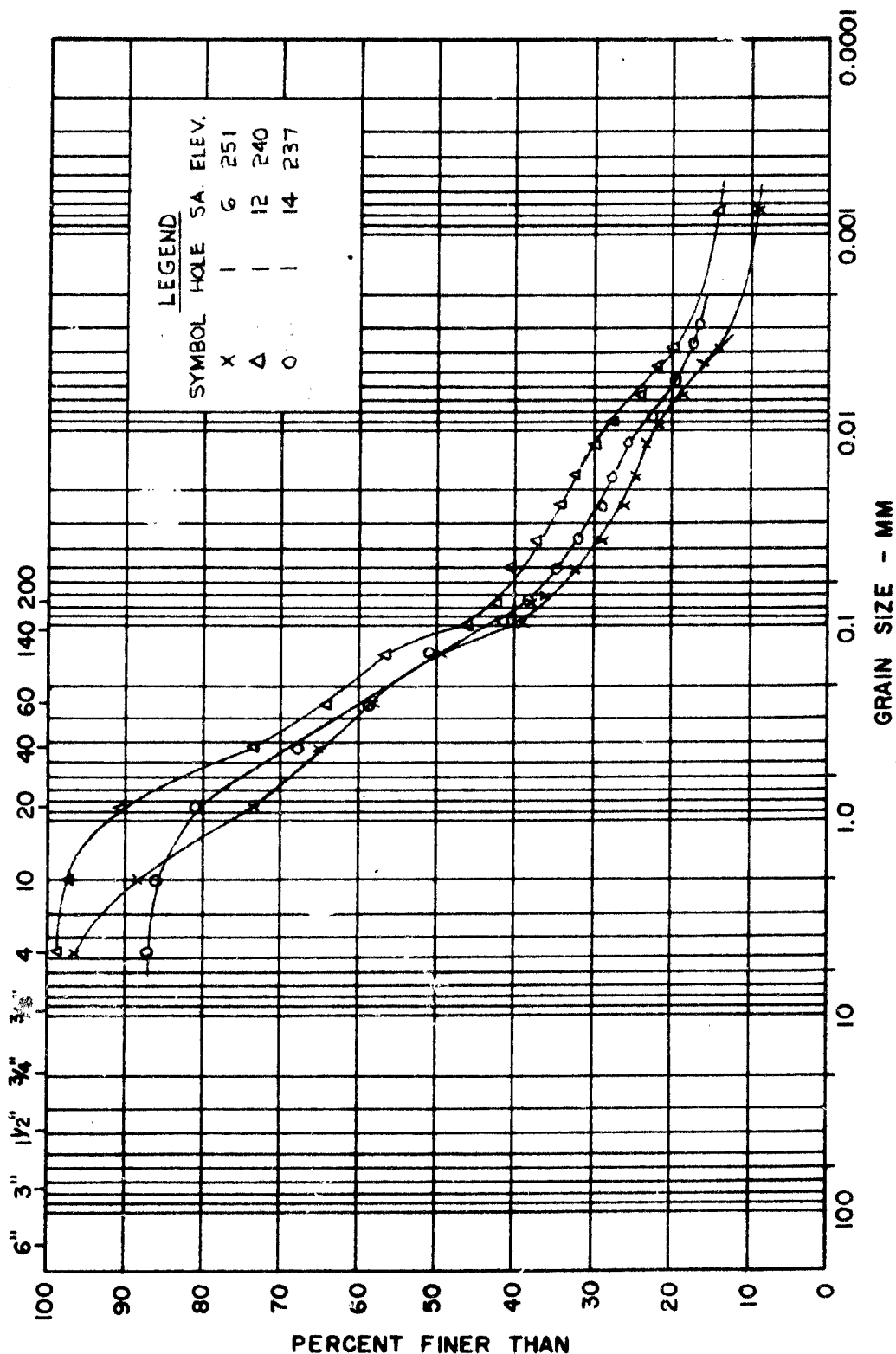


# GRAIN SIZE DISTRIBUTION CLAYEY TILL STRATUM

FIGURE DI

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES / IN.



GOLDER & ASSOCIATES

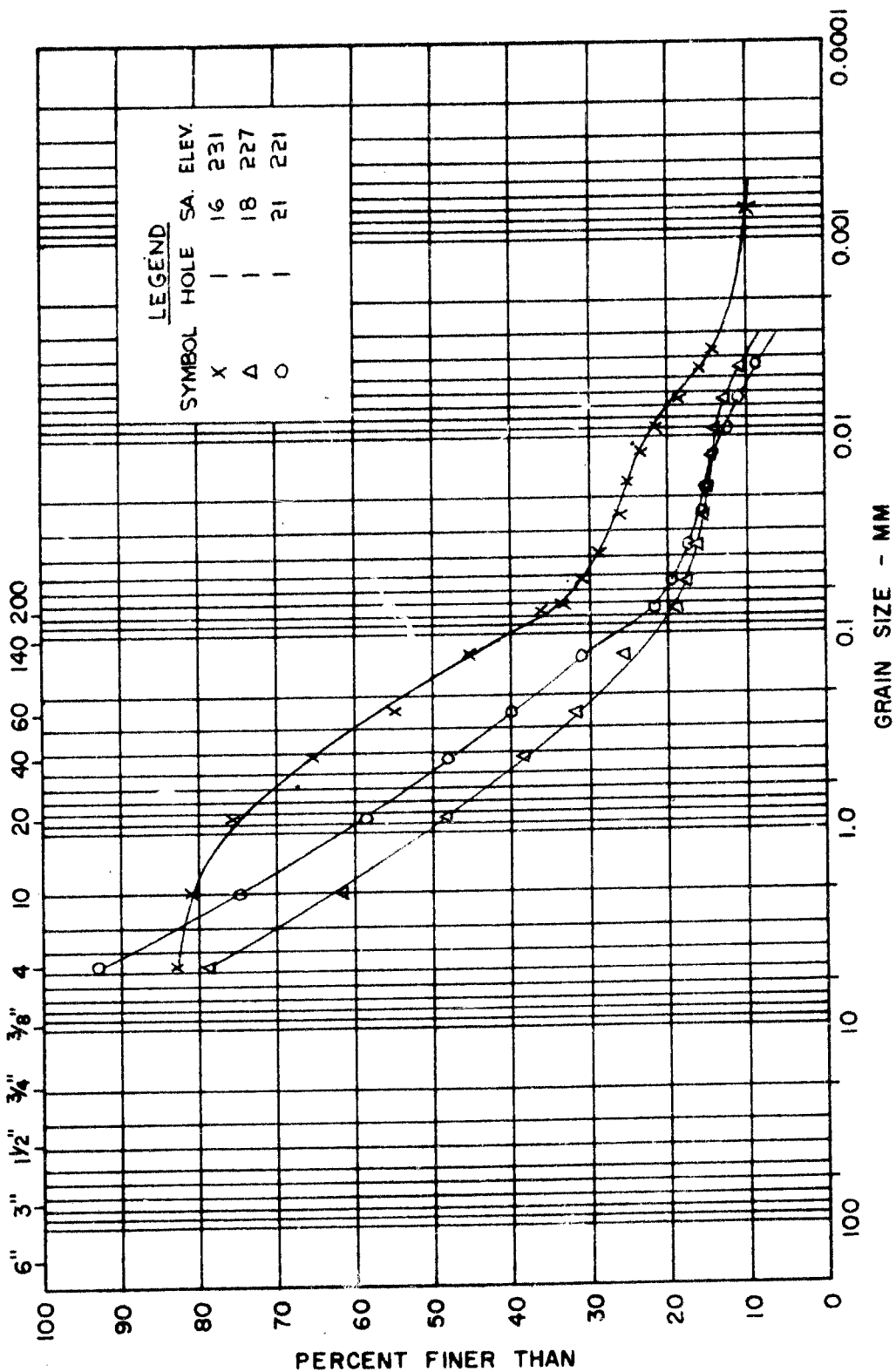
# GRAIN SIZE DISTRIBUTION

CLAYEY TILL STRATUM

FIGURE D2

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.

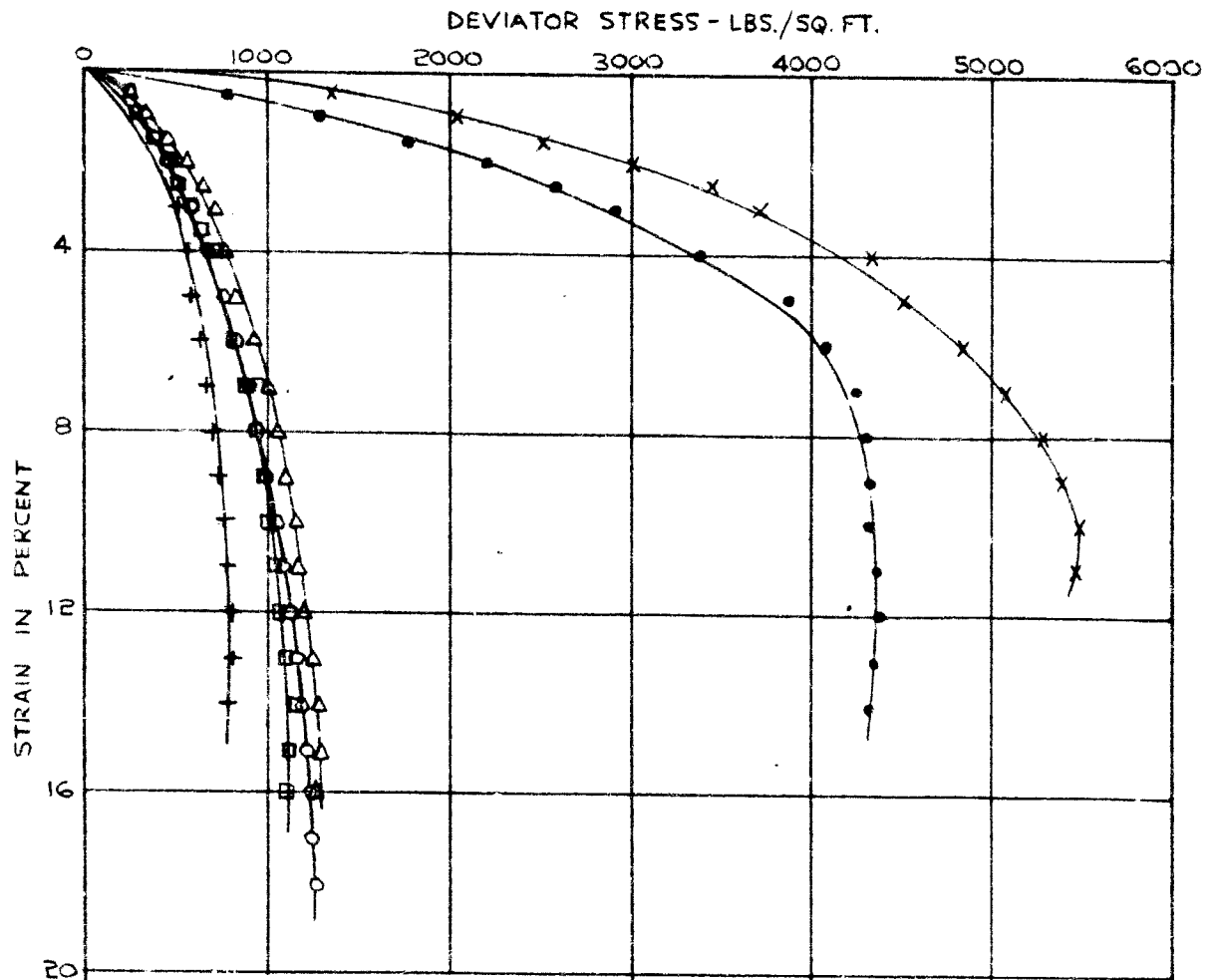


COBBLE SIZE	GRAVEL SIZE		SAND SIZE			SILT SIZE		CLAY SIZE	
	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE

# UNDRAINED TRIAXIAL COMPRESSION TESTS

TYPICAL STRESS-STRAIN CURVES

FIGURE D3

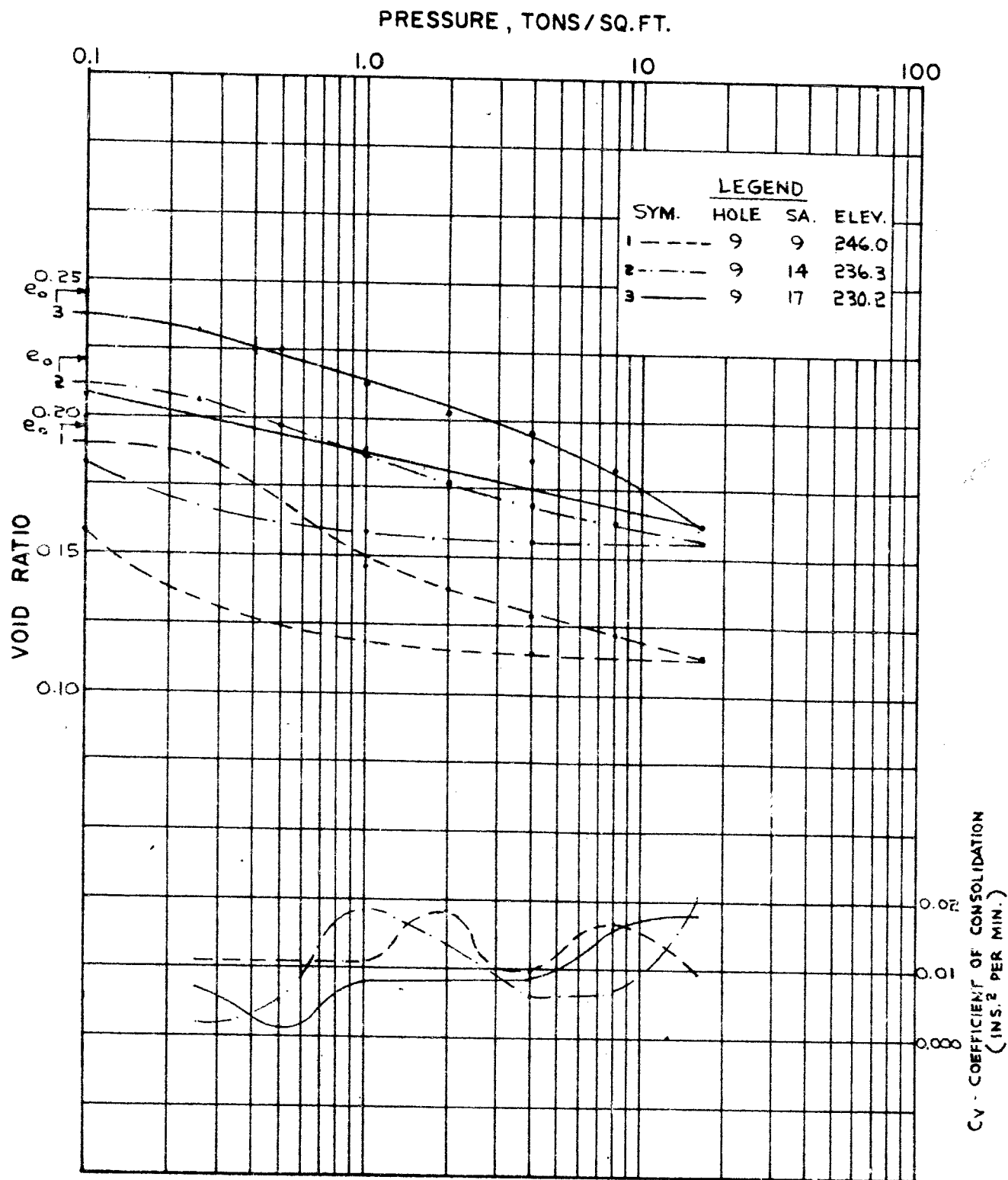


## LEGEND

SYMBOL	HOLE	SAMPLE	ELEV.
X	1	4	255.1
O	1	13	238.6
Δ	1	20	222.6
●	9	9	246.1
□	9	17	230.6
+	9	22	216.6

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE D4





# **APPENDIX E**

## **Non-Standard Special Provisions**



## FOUNDATION REPORT BRINSTON ROAD UNDERPASS REPLACEMENT - HIGHWAY 401

### VIBRATION MONITORING – Item No.

Special Provision

#### 1.0 SCOPE

This special provision describes requirements for vibration monitoring during pile installation for the replacement of the Brinston Road underpass.

#### 2.0 DEFINITIONS

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

#### 3.0 SUBMISSION REQUIREMENTS

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments on the existing Aultsville Road underpass.
- Proposed frequency of readings.
- Action plan to be taken to adjust deep foundation installation methods if readings show vibrations exceeding tolerable levels.

#### 4.0 MONITORING

The vibration monitoring equipment shall be placed on the existing Brinston Road underpass. The Contractor shall take readings on the existing structure throughout pile driving operations, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on the existing bridge structures shall not exceed 100 mm/s (peak particle velocity). If the readings are not within these limits, the Contractor must alter the deep foundation installation procedures until the vibrations at the existing structure are within acceptable levels.

#### 5.0 BASIS OF PAYMENT

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

#### END OF SECTION



## **FOUNDATION REPORT BRINSTON ROAD UNDERPASS REPLACEMENT - HIGHWAY 401**

### **DEEP FOUNDATIONS – Item No.**

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Special Provision

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#### **1.0 SCOPE**

The predominant soil deposit at this site is a clayey silt till. Because of the glacial origin of the till deposit, cobbles or boulders should be expected. The Contractor is advised that appropriate equipment and construction procedures may be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundation elements and shoring elements. The Contractor is also advised that the glacial till may be susceptible to disturbance under conditions of unbalanced hydrostatic head, and that appropriate equipment and construction procedures may be required for caisson construction through the till deposit.

Where caisson foundations are adopted, these will extend into the dolostone bedrock, which is strong to very strong. Appropriate construction procedures and equipment will be required to penetrate the bedrock.

#### **2.0 BASIS OF PAYMENT**

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

**END OF SECTION**



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