November 10, 2014

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Seventh Street Underpass Replacement
Queen Elizabeth Way
Regional Municipality of Niagara
GWP 2177-08-00

Submitted to:
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REPORT

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Table of Contents

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION 1

2.0 SITE DESCRIPTION 1

3.0 INVESTIGATION PROCEDURES 1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS 2

4.1 Regional Geology 2

4.2 Subsurface Conditions 3

4.2.1 Asphalt 3

4.2.2 Fill 3

4.2.3 Clayey Silt 4

4.2.4 Clayey Silt Till 4

4.2.5 Silt and Sandy Gravel 5

4.2.6 Shale Slabs/Cobbles and Boulders 5

4.2.7 Shale Bedrock 6

4.3 Groundwater Conditions 6

5.0 CLOSURE 7

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS 8

6.1 General 8

6.2 Foundation Options 8

6.3 Shallow Foundations 10

6.3.1 Founding Elevations 10

6.3.2 Geotechnical Resistance/Reaction 10

6.4 Driven Steel H-Pile or Steel Pipe (Tube) Foundations 11

6.4.1 Founding Elevations 11

6.4.2 Axial Geotechnical Resistance/Reaction 12

6.5 Approach Embankments 12

6.5.1 Subgrade Preparation and Embankment Construction 12

6.5.2 Approach Embankment Stability ... 13
6.5.3 Approach Embankment Settlement ... 14
6.6 Construction Considerations ... 15
6.6.1 Excavation and Temporary Protection Systems ... 15
6.6.2 Groundwater Control ... 15
6.6.3 Subgrade Protection ... 15
6.6.4 Driving Piles into Shale Slab/Cobble and Boulder Layer ... 15
6.6.5 Conflicts with Existing Foundation Elements ... 16
6.6.6 Vibration Monitoring During Pile Installation ... 16
6.7 Recommendations for Further Work During Detail Design ... 16
7.0 CLOSURE ... 17

REFERENCES

TABLES

Table 1 Comparison of Foundation Options

DRAWINGS

Drawing 1 Seventh Street Underpass – Borehole Locations and Soil Strata

FIGURES

Figure 1 Static Global Stability – Seventh Street – South Approach Embankment
Figure 2 Static Global Stability – Seventh Street – North Approach Embankment

APPENDIX A Borehole Records

Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Records of Boreholes 13-01 and 13-02

APPENDIX B Laboratory Test Results

Figure B1 Grain Size Distribution Test Result – Sand and Gravel Fill
Figure B2 Grain Size Distribution Test Result – Clayey Silt Fill
Figure B3 Plasticity Chart – Clayey Silt Fill
Figure B4 Grain Size Distribution Test Results – Sand and Silt Fill
Figure B5 Grain Size Distribution Test Results – Clayey Silt
Figure B6 Plasticity Chart – Clayey Silt
Figure B7 Grain Size Distribution Test Results – Clayey Silt Till
Figure B8 Plasticity Chart – Clayey Silt Till

APPENDIX C Records of Boreholes from Previous Investigation (GEOCRES No. 30M03-016)

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
SEVENTH STREET UNDERPASS REPLACEMENT
QUEEN ELIZABETH WAY
REGIONAL MUNICIPALITY OF NIAGARA
GWP 2177-08-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement/rehabilitation of seven structures (Seventh Street, Lyon's Creek, Tee Creek and Black Creek) on the Queen Elizabeth Way (QEW) highway in the Regional Municipality of Niagara, Ontario.

This report addresses the replacement/rehabilitation of the existing Seventh Street underpass structure, based on the results from a 2013/2014 foundation investigation as well as the results from an earlier investigation, as follows:

- MTO GEOCRES No. 30M03-016: Report titled "Foundation Investigation Report for Proposed Seventh Street Underpass and Q.E.W., Twp. Of Louth, County of Lincoln, District #4 (Hamilton), W.J. 66-F-65, W.P. 212-63", prepared by Department of Highways Ontario, dated August 1966.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2011-E-0045 dated June 2011, and in Section 5.8 of the *Technical Proposal* for this assignment. Additional boreholes with bedrock coring were subsequently undertaken, per the change request dated February 2014.

2.0 SITE DESCRIPTION

The Seventh Street underpass structure is located at the intersection of the QEW and Seventh Street in the City of St. Catharines, within the Regional Municipality of Niagara, Ontario. The existing underpass consists of a 103.0 m long (measured along centerline of Seventh Street and between the centerlines of the abutment bearings) by 22.2 m to 22.6 m wide (measured between outside edges of the deck) four-span structure, with the existing abutment and piers supported on vertical and battered piles.

In general, the terrain in this area is relatively flat, with the natural ground surface in the immediate vicinity of the structure at about Elevation 87.5 m on the south side of the highway and about Elevation 85.5 m on the north side. The QEW has been constructed on a low embankment relative to the natural ground surface, with the existing pavement grade at approximately Elevation 89.3 m.

Seventh Street has been constructed on embankment fill, with embankment heights of about 7.5 m and 9.5 m at the south and north approaches to the underpass structure, respectively. The pavement grade on Seventh Street is at approximately Elevation 95.2 m at the abutments, and about Elevation 95.5 m at the structure crown. The abutment foreslopes and embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

The field work for the current subsurface investigation was carried out from April 22 to 25, 2013 and from July 28 to August 14, 2014, at which time four boreholes (13-01, 13-02, 14-01 and 14-02) were advanced using a truck-mounted CME-75 drill rig supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole locations are shown on Drawing 1. Boreholes 13-01 and 13-02 were advanced in the southeast and northwest quadrants of the proposed underpass through the Seventh Street embankment. Boreholes 14-01 and 14-02 were advanced on the existing ramps in the southwest and northeast quadrants of the proposed underpass. .

Boreholes 13-01 and 13-02 were drilled using 108 mm inner diameter hollow stem augers through the overburden to depths of 24.5 m and 29.0 m, respectively, where they were terminated after encountering shale in the bottom sample. Samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. Boreholes 14-01 and 14-02 were drilled using 108 mm inner diameter hollow stem augers without sampling through the overburden, to auger or split spoon refusal at depths of 19.4 m and 18.3 m, respectively. NQ-size coring was completed below this depth and encountered residual soils, cobbles and boulders. Shale bedrock was cored below the cobbles and boulders in Borehole 14-02. Both Boreholes 14-01 and 14-02 encountered difficulties in attempting to penetrate (core) through the cobble and boulder layer, and the coring operations in these boreholes were terminated after the second or third night shift, respectively.

The groundwater conditions were observed in the open boreholes during and immediately following the overburden drilling operations. Upon completion of drilling, the boreholes were backfilled with bentonite grout, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised full-time by a member of Golder's staff who located the boreholes in the field, contacted public utility companies to locate the existing underground services and cleared the borehole locations, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Cambridge for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. The geotechnical laboratory testing was completed according to applicable MTO LS standards.

The locations and ground surface elevations of the boreholes advanced as part of the 2013/2014 investigation were measured in the field by Callon Dietz, Ontario Land Surveyors. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
13-01	4,782,141.5	320,340.5	94.7	24.5
13-02	4,782,263.6	320,321.3	94.8	29.0
14-01	4,782,172.5	320,314.8	89.5	22.5
14-02	4,782,250.3	320,351.4	89.5	23.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of the QEW is located in the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984).

The Iroquois Plain physiographic region covers the area adjacent to Lake Ontario from the Niagara River to the Trent River. Due to the large area covered by the Iroquois Plain, it is divided into a number of sub-regions, one of which is the Niagara Fruit Belt that covers the area of the Seventh Street interchange. The Niagara Fruit Belt lies between Lake Ontario and the Niagara Escarpment and extends eastward from Hamilton to the Niagara River. The general topography of this region consists of relatively flat-laying ground sloping gradually towards Lake Ontario. The overburden within the majority of the Niagara Fruit Belt area is underlain by shale bedrock of the Queenston Formation, which contains stronger limestone interlayers.

4.2 Subsurface Conditions

As part of Golder's 2013/2014 subsurface investigation, four boreholes were advanced in the vicinity of the existing Seventh Street underpass structure. The borehole locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawing 1. This drawing also shows the approximate location of seven "deep" boreholes, which penetrated through the overburden soils to refusal, advanced as part of the 1966 investigation at this site by the Department of Highways Ontario. The locations of the 1966 boreholes are considered approximate, as they have been plotted based on station and offset measurements given in the 1966 report.

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing from the current investigation are also presented on the borehole records and on Figures B1 to B8 contained in Appendix B. The borehole records from the 1966 investigation are contained in Appendix C. The stratigraphic boundaries shown on the borehole records and interpreted stratigraphic section on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological changes. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions encountered at the site consist of embankment fill overlying a deposit of firm to very stiff clayey silt, which is underlain by very stiff to hard clayey silt till. The clayey silt till was not encountered in one borehole (Borehole 13-01) near the south abutment; at this location, the clayey silt is underlain by a layer of dense sandy gravel. Similar layers of dense silt and dense sandy gravel were encountered below the till in Borehole 13-02 near the north abutment. Based on the coring results from the 2014 boreholes, these soils are underlain by a layer of very dense gravel, cobbles and boulders, and shale fragments, which is in turn underlain by shale bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in Sections 4.2.1 to 4.2.7.

4.2.1 Asphalt

Approximately 300 mm of asphalt was encountered at the surface in Boreholes 13-01 and 13-02, which were advanced through the existing Seventh Street pavement in the southeast and northwest quadrants of the structure site, respectively.

4.2.2 Fill

Approximately 6.9 m and 9.1 m of fill associated with the Seventh Street embankment was encountered below the asphalt in Boreholes 13-01 and 13-02, respectively. The fill extends to about 7.2 m and 9.4 m (Elevations 87.5 and 85.4 m) behind the south and north abutments. The upper portion of the fill consists of sand and gravel road base, and the lower portion of the fill varies in composition from clayey silt to silt and sand. The measured

Standard Penetration Test (SPT) 'N' values within the sand and gravel fill range from 9 blows to 24 blows per 0.3 m of penetration, indicating that the sand and gravel has a loose to compact relative density. The measured SPT 'N' values within the clayey silt fill range from 5 blows to 14 bows per 0.3 m of penetration, suggesting a firm to stiff consistency. The measured SPT 'N' values within the silt and sand fill range from 6 blows to 13 blows per 0.3 m of penetration, suggesting that the material has a loose to compact relative density.

The results of grain size distribution tests completed on one sample of sand and gravel fill, one sample of the clayey silt fill and one sample of the silt and sand fill are shown on Figures B1, B2 and B4 in Appendix B. Atterberg Limits testing was completed on a sample of the clayey silt fill, and measured a plastic limit of about 16 per cent, a liquid limit of about 21 per cent, and a plasticity index of 5 per cent; these results are plotted on a plasticity chart on Figure B3 in Appendix B; this test result confirms that the cohesive portion of the fill consists of clayey silt of low plasticity. Laboratory testing of selected samples of sand and gravel fill materials measured natural water contents of approximately 4 per cent to 9 per cent. The natural water content measured on selected samples of clayey silt fill ranged between 15 per cent and 18 per cent and the natural content of two selected samples of silt and sand fill were 14 per cent and 22 per cent.

4.2.3 Clayey Silt

A 3.9 m to 15.2 m thick deposit of clayey silt was encountered below the fill materials in Boreholes 13-01 and 13-02, and in Boreholes 1 to 7 from the 1966 investigation, extending to a base elevation of approximately 71.0 m to 81.5 m. This deposit consists of clayey silt containing trace quantities of sand and gravel.

The measured SPT 'N' values within the clayey silt range from 6 blows to 85 blows per 0.3 m of penetration, but are typically between 6 and about 30 blows per 0.3 m of penetration. In situ vane testing was conducted in the clayey silt deposit in the current and 1966 investigations and measured undrained shear strengths that are generally greater than 100 kPa; undrained shear strength values of approximately 80 kPa to 85 kPa were measured in the upper portion of the deposit in two of the 1966 boreholes. These test results suggest that the clayey silt deposit generally has a stiff to very stiff consistency.

The results of grain size distribution tests completed on two selected samples of the clayey silt from the current investigation are shown on Figure B5 in Appendix B. Atterberg limits testing was carried out on four selected samples of the clayey silt from the current investigation, and measured plastic limits of 13 per cent to 16 per cent, liquid limits of 20 per cent to 27 per cent, and plasticity indices of 6 per cent to 11 per cent. These test results, which are plotted on a plasticity chart on Figure B6 in Appendix B and which are consistent with the 1966 results, confirm that this deposit consists of low plasticity clayey silt. The natural water contents measured on selected samples of the clayey silt samples range from about 16 per cent to 26 per cent.

4.2.4 Clayey Silt Till

A 1.7 m to 9.9 m thick deposit of clayey silt till was encountered below the clayey silt in Boreholes 13-02,14-02, and Boreholes 1 to 7 from the 1966 investigation, with the deposit surface between about Elevation 71.0 m and 81.5 m, and its base between about Elevation 66.1 m and 71.8 m. This till deposit consists of clayey silt with sand, containing trace quantities of gravel.

The measured SPT 'N' values within the clayey silt till range from 19 blows to greater than 100 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The results of grain size distribution tests completed on two selected samples of the clayey silt till from the current investigation are shown on Figure B7 in Appendix B. Atterberg limits testing was carried out on two selected samples of the till from the current investigation and measured plastic limits of 13 per cent and 15 per cent, liquid limits of 20 per cent and 22 per cent, and plasticity indices of 7 per cent. These test results, which are plotted on a plasticity chart on Figure B8 in Appendix B, confirm that the till consists of low plasticity clayey silt. The natural water contents measured on selected samples of the clayey silt till samples range from 10 per cent to 12 per cent, below the plastic limit for the material.

4.2.5 Silt and Sandy Gravel

A 3.0 m thick layer of silt containing some sand was encountered below the clayey silt till deposit in Borehole 13-02 near the north abutment, extending to a depth of about 26.2 m (Elevation 68.6 m). A 1.8 m to 3.1 m thick layer of sandy gravel was encountered below the clayey silt in Borehole 13-01 near the south abutment, and below the silt layer in Borehole 13-02 near the north abutment. The surface of these layers was encountered at about Elevation 74.3 m and 71.6 m at the south and north abutments, respectively, and the base of these layers was encountered at about Elevation 71.2 m and 66.8 m, again declining from the south to the north.

The measured SPT 'N' values within the silt and sandy gravel layers range from 33 blows to 49 blows per 0.3 m of penetration, suggesting that these layers have a dense relative density.

4.2.6 Shale Slabs/Cobbles and Boulders

Boreholes 13-01 and 13-02 from the current investigation, as well as Boreholes 1 to 7 from the 1966 investigation, encountered refusal within what was described at the time of investigation as shale bedrock, based on observation of the recovered material in the split-spoon sampler. However, based on the results of coring from Boreholes 14-01 and 14-02 advanced in the second phase of the current investigation, a layer of shale slabs, cobbles and boulders is present below the clayey silt and the clayey silt till deposits. This cobble and boulder/shale slab layer was proven only in Boreholes 14-01 and 14-02, where its surface was encountered at Elevation 70.2 m and 68.3 m, respectively. Borehole 14-01 penetrated the shale slabs/cobbles and boulders for 3.2 m (to Elevation 67.0 m) and was terminated within the layer; in Borehole 14-02, this layer is about 1.1 m thick, with its base at about Elevation 67.2 m,

Based on the results from Boreholes 14-01 and 14-02, observations of shale in the split-spoon sampler and/or refusal have been re-interpreted to represent a layer of shale slabs, cobbles and boulders. The surface elevation of this layer is summarized in the following table:

Borehole No.	Surface Elevation of Shale Slab/Cobble and Boulder Layer (m)
13-01	71.2
13-02	66.8
14-01	70.2
14-02	68.3
1 (1966)	70.8
2 (1966)	71.8

Borehole No.	Surface Elevation of Shale Slab/Cobble and Boulder Layer (m)
3 (1966)	68.8
4 (1966)	66.3
5 (1966)	66.9
6 (1966)	66.1
7 (1966)	66.8

The SPT 'N' values measured in this layer are all greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

4.2.7 Shale Bedrock

Shale bedrock was encountered below the cobbles and boulders at Elevation 67.2 m in Borehole 14-02 near the north abutment, where it was cored for 1.0 m. Based on the cored samples, the bedrock generally consists of red shale of the Queenston Formation.

The recovered bedrock core is described as slightly weathered, and weak to medium strong. The Rock Quality Designation (RQD) measured on the core samples was about 0 per cent, indicating a rock mass of very poor quality. The Total Core Recovery (TCR) of the core samples is about 36 per cent.

4.3 Groundwater Conditions

Details of the water conditions observed in the open boreholes at the time of drilling are summarized on the borehole records following the text of this report.

The water levels measured in the open boreholes during drilling operations are summarized in the following table:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date
13-01	94.7	12.6	82.1	April 25, 2013
1 (1966)	88.7	2.4	86.3	June 1966
2 (1966)	89.3	2.1	87.2	June 1966
3 (1966)	89.9	2.1	87.8	June 1966
4 (1966)	87.8	1.7	86.1	June 1966
5 (1966)	87.6	1.7	86.0	June 1966
6 (1966)	87.3	1.8	85.5	June 1966
7 (1966)	88.1	5.0	83.1	July 1966

The groundwater level should be expected to fluctuate seasonally and should be expected to rise during wet periods of the year.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Mehdi Mostakhdemi, M.Sc., P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
SEVENTH STREET UNDERPASS REPLACEMENT
QUEEN ELIZABETH WAY
REGIONAL MUNICIPALITY OF NIAGARA
GWP 2177-08-00

November 10, 2014
Report No. 12-1111-0088-1

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement/realignment of the existing QEW-Seventh Street underpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation, supplemented with data from a 1966 investigation at this site by Department of Highways Ontario. 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 preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the construction contract. Those requiring information on construction aspects 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 structure consists of a four-span underpass, with the existing abutments and piers supported on battered piles. Based on the *General Plan and Footing Layout* drawings for the existing structure, dated April 1968, the existing foundation details are summarized as follows:

Foundation Element	Pile Cap Width	Founding Elevation
South abutment	2.9 m	Pile cap: 90.8 m (298.0 ft.) Pile tip: 70.5 m (231.2 ft.)
South Pier	4.6 m	Pile cap: 88.8 m (291.2 ft.) Pile tip: 69.7 m (228.8 ft.) at east to 68.2 m (223.8 ft.) at west
Centre Pier	4.6 m	Pile cap: 88.4 m (290.0 ft.) Pile tip: 66.2 m (217.2 ft.)
North Pier	4.6 m	Pile cap: 88.8 m (291.5 ft.) Pile tip: 66.7 m (218.8 ft.)
North abutment	2.9 m	Pile cap: 90.8 m (298.0 ft.) Pile tip: 65.9 m (216.2 ft.)

6.2 Foundation Options

Based on the planning study completed to date, it is understood that the future widening could consist of three additional lanes on the QEW, and that the existing four-span underpass structure will require replacement. The location of the new south abutment is proposed to be west of the existing abutment and the new north abutment is proposed to be immediately east of the existing abutment.

The pavement grade at QEW is proposed to be at about Elevation 89.4 m at the structure site, similar to the existing grade. The finished grade for the realigned Seventh Street will be approximately Elevation 96.0 m and 95.8 m at north and south abutments, respectively. Based on the current natural ground surface in the vicinity of the approach embankments, the south approach embankment will be up to approximately 8.5 m in height, and the north approach embankment will be up to approximately 10.3 m in height.

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and piers for the new Seventh Street underpass. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the stiff to very stiff clayey silt deposit:** Strip or spread footings are constructable for support of the new abutments and piers at this site, and would permit semi-integral abutment design; however, the preliminary geotechnical resistances associated with the stiff portion of the deposit are likely not sufficiently high to permit design of the replacement structure on strip or spread footings. Further, significant excavation depths (up to 4 m at the piers, and more than 8 m at the abutments) would be required, and temporary protection systems would be required to permit excavation through the existing Seventh Street embankment at the abutment locations, as well as parallel to the QEW lanes for the pier excavations.
- **Footings "perched" on a compacted granular pad in the approach embankment:** Up to about 70 mm to 90 mm of settlement is predicted under the new 8.5 m to 10.3 m high approach embankments; while the majority of this settlement is expected to be completed during and immediately following construction, there is still potential for more than 25 mm of time-dependent settlement of the firm portion(s) of the clayey silt deposit. Because of this, depending on the foundation option for the piers, there is a potential for differential settlement between the foundation elements with this option. Therefore, perched abutment footings are not recommended for support of the replacement structure at this site.
- **Driven steel H-piles:** Driven steel H-piles are suitable and feasible for support of new abutments (and would permit integral abutment design), wing walls/retaining walls and piers at this site. There is a risk associated with damaging the piles during driving to or into the layer of shale slabs/cobbles and/or boulders above the shale bedrock. However, the pile driving can be controlled to avoid over-driving the piles once this layer is reached, and a reasonable bearing resistance can be achieved on this layer.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of new abutments, associated wing walls/retaining walls, and the piers at this site. However, pipe piles are considered to have a slightly higher risk than H-piles for hanging up, being damaged or deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders at this site.
- **Caissons:** Caissons are feasible for this site but would require the use of temporary or permanent liners given the potential risks and difficulties associated with the water-bearing silt and sandy gravel deposits through which caissons would be constructed. Due to these risks and potential construction difficulties, caissons are not considered to be a preferred foundation system compared to driven pile foundations for this structure site and therefore are not discussed in detail in subsequent sections of this report. However, the relative advantages and disadvantages of caisson foundations are summarized in Table 1.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments, associated wingwalls/retaining walls and piers for the replacement structure on steel H-pile foundations supported on the very dense layer of shale slabs/cobbles and boulders, immediately above the bedrock surface. Consideration will have to be given to potential conflicts between the new and existing foundation elements, particularly at the centre pier and north abutment based on the currently proposed alignment and layout for the replacement structure. In general, the existing pile caps may be left in place if permitted or removed where necessary to avoid conflicts with new foundation elements. Where existing piles are present within or near the footprint of new pile caps, the existing piles should be cut off below the level of the new pile cap; the existing piles should not be extracted.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, associated wing walls/retaining walls, and new piers, strip or spread footings should be founded below any fill and below any potentially soft near-surface soils, on the stiff to very stiff clayey silt. The following founding elevations are recommended for preliminary design of shallow foundations; as shown in the table below, significant excavation would be required at the abutments and piers to reach these founding levels.

Foundation Element	Borehole No.	Maximum (Highest) Founding Elevation	Founding Stratum	Approximate Excavation Depth
South abutment, south and centre piers	13-01	87.0 m	Stiff clayey silt	2.5 m (Piers) 8 m (Abutment)
North abutment and north pier	13-02	85.0 m	Stiff clayey silt	4 m (Pier 10 m (Abutment)

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with provincial standards to confirm that all existing fill, softened clayey silt soils or other unsuitable material have been removed. The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab be placed on the prepared subgrade within four hours of its inspection and approval, as discussed further in Section 6.6.3.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared subgrade, at or below the preliminary design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) given below.

Founding Stratum	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS*
South abutment, south pier and centre pier	3 m	275 kPa	200 kPa
North abutment and north pier	3 m	350 kPa	300 kPa

* For 25 mm of settlement

The preliminary geotechnical resistances/reactions should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary.*

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutment and pier locations.

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

6.4.1 Founding Elevations

The new abutments, associated wing walls/retaining walls and piers may be supported on steel H-piles or steel pipe (tube) piles driven to found on the very dense layer of shale slabs, cobbles and boulders that was encountered above the shale bedrock. This layer is 1.1 m to more than 3.2 m thick as encountered in Boreholes 14-01 and 14-02, and it is not feasible to attempt to drive the piles through this very dense layer to reach the shale bedrock. The following pile tip elevations may be used for preliminary design purposes, based on interpolation (where more than one reference borehole is considered) of the surface of the end-bearing layer and assuming nominal penetration of approximately 0.5 m to 1 m into the very dense layer of shale slabs, cobbles and boulders:

Foundation Element	Reference Borehole No(s).	Cobble/Boulder Surface Elevation in Boreholes (m)	Estimated Design Pile Tip Elevation (m)
South abutment	2 (1966) 3 (1966)	71.8 68.8	71 (East) 68 (West)
South pier	14-01 1 (1966)	70.2 70.8	70
Centre pier	7 (1966)	66.8	66
North pier	14-02 5 (1966)	68.3 66.9	67
North abutment	14-02 4 (1966)	68.3 66.3	67

As can be seen in the table above, the surface of the very dense ("100-blow") layer varies in the boreholes, and provisions should be made in the Contract Documents to deal with varying pile lengths at the location of the abutments and piers. The pile caps should be constructed at a minimum depth of 1.2 m for frost protection purposes per Provincial Standards.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the presence of cobbles and boulders within the clayey silt and till deposits, and to the very dense layer of shale slabs, cobbles and boulders into which the piles will be driven. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of hanging up, being damaged or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving in accordance with Provincial Standards. For the very dense layer of shale slabs, cobbles and boulders into which piles will be driven at this site, driving shoes are preferred over flange plates.

6.4.2 Axial Geotechnical Resistance/Reaction

For preliminary design for HP 310x110 piles driven to/into refusal in the very dense layer of shale slabs, cobbles and boulders at the estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at ULS may be taken as 1,400 kN, and the axial geotechnical reaction at SLS (for approximately 10 mm of settlement) may be taken as 1,200 kN. The same axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.). These preliminary geotechnical resistances should be re-evaluated and modified as necessary during detail design in consideration of any additional subsurface investigation in the vicinity of the foundation elements for the replacement structure.

Pile installation should be in accordance with OPSS 903 *(Deep Foundations)*. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. As the upper portion of the layer of shale slabs, cobbles and boulders demonstrates some weathering (based on ability to penetrate this layer by augering and/or split-spoon sampling), it is anticipated that the piles will penetrate nominally into the layer (as noted in the preceding section). The piles should be driven to an appropriate set criterion to be established at the time of construction, once the piling equipment is known. However, if "abrupt peaking" is met on stronger materials within the cobble/boulder layer, it is recommended that the hammer energy be immediately reduced, and then gradually increased over a series of blows to set the piles while minimizing damage to the piles.

6.5 Approach Embankments

6.5.1 Subgrade Preparation and Embankment Construction

Based on the proposed alignment for the replacement structure, the existing Seventh Street embankments will require widening to the west at the south approach, and to the east at the north approach. It is recommended that any topsoil/organic material be stripped from the footprint of the proposed approach embankment widening. The depth and extent of stripping should be re-assessed during detail design if additional subsurface information is available for the approach embankment widening areas.

Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. The embankment fill for the realigned Seventh Street should be placed and compacted in accordance with OPSS

206 *(Grading)* and OPSS 501 *(Compacting)*. Benching of the west and east sides of the existing Seventh Street embankment should be carried out to "key in" the new fill materials for the realignment/widening, in accordance with OPSD 208.010 *(Benching of Earth Slopes)*.

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where the fill embankment side slopes are equal to or greater than 8 m in height such that the uninterrupted slope height does not exceed 8 m. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments.

6.5.2 Approach Embankment Stability

Preliminary slope stability analyses have been performed for the proposed widened approach embankments using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed realignment/eastward widening on this underpass, considering the design requirements and the available field and laboratory testing data.

Preliminary stability analyses were completed for a 8.5 m high south approach embankment and a 10.3 m high north approach embankment, based on the subsurface conditions as encountered in Boreholes 13-01 and 13-02, respectively, and also supported by the geotechnical data for the clayey silt deposit from the 1966 investigation. No mid-height bench was assumed in the preliminary stability analyses for these slopes, but the mid-height bench would serve to improve the factor of safety further. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32°	-
Stiff to very stiff clayey silt	20	30°	100
Very stiff to hard clayey silt till	21	32°	-
Dense silt	20	32°	-
Dense sandy gravel	21	32°	-

The analysis results indicate that the 8.5 m and 10.3 m high south and north approach embankments will have a factor of safety of at least 1.3 against global instability, provided that the side slopes are oriented no steeper than 2 horizontal to 1 vertical (2H:1V), and assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. Example static global stability results are provided on Figures 1 and 2 for the south and north embankments, respectively. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on any additional borehole data obtained for the near-surface soils within the footprint of the widened approaches during detail design.

6.5.3 Approach Embankment Settlement

Preliminary analysis of the settlement below the widened approach embankments was carried out using the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli as given in the table below, based on correlations with the SPT "N" values, undrained shear strengths, Atterberg limits testing and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Embankment fill for widening	21	-
Stiff upper portion of clayey silt	20	15
Very stiff to hard portion of clayey silt	21	40
Very stiff to hard clayey silt till	21	40-100

Based on the preliminary settlement assessment, the following comments are provided:

- At the north approach, an eastward widening of up to about 10 m is proposed. Based on the existing site grading, this will require placement of a thickness of up to about 7 m of new fill on top of the existing embankment side slope to achieve the maximum approach height of about 10.3 m. The subsurface conditions generally consist of an upper few metres of stiff clayey silt, underlain by very stiff to hard clayey silt and hard clayey silt till. The settlement of the foundation soils under this north approach widening is estimated to be up to about 70 mm to 90 mm. The majority of this settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments. As it is anticipated that the approach embankment widening will be constructed up to the underside of the pile cap, followed by pile driving and pile cap construction; this will essentially act as a "preload", with the effect that the majority of the settlement will be completed by the time the pile cap is constructed. Based on the estimated settlement and soil behaviour, and this assumed construction sequence, downdrag loads do not need to be taken into account for the north abutment piles, and no other settlement mitigation measures are considered necessary for the northward widening.
- At the south approach, nominal westward widening may be required. A preliminary settlement assessment has been completed based on placement of a 2 m thickness of fill on top of the existing west embankment side slope. For this nominal widening, the settlement of the soils below the embankment will be less than 25 mm; this settlement will occur relatively quickly during and immediately following placement of fill for the widening.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment widening depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.6 Construction Considerations

The following sections identify future construction considerations that should be considered at this stage as they may impact the preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation in the Contract Documents.

6.6.1 Excavation and Temporary Protection Systems

It is understood that Seventh Street will be closed during the replacement of the existing underpass structure, as it will not be possible to stage the construction in two halves given the new alignment relative to the existing structure.

The excavations for pile caps will extend through the existing Seventh Street embankment fill at the abutment locations, and through the QEW embankment fill and potentially into stiff clayey silt at the pier locations. If space and construction activities permit at the abutment pile cap locations, open-cut excavations should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill is classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

At this preliminary stage, it is anticipated that temporary protection systems will be required along the QEW to facilitate removal down to/including the existing pier pile caps and for the new pier foundations, to minimize the footprint of the removals and new construction and therefore minimize impacts on traffic on the QEW. Temporary protection systems may also be required near the abutment locations, if there is insufficient room for open-cut excavations based on construction activities. .The temporary protection systems should be designed and constructed in accordance with OPSS 539, to meet Performance Level 2 adjacent to the QEW lanes.

6.6.2 Groundwater Control

Groundwater seepage into the foundation excavations is anticipated from cohesionless soil interlayers within the clayey silt or till deposits (where these are present), and from groundwater "perched" on top of the clayey silt deposit within existing granular fill. The seepage volume is expected to be relatively small, such that the water inflow can be handled by pumping from filtered sumps placed at the base of the excavation. Based on these small seepage volumes, it is not expected that a Permit to Take Water (PTTW) would be required for the temporary groundwater control system at this site.

6.6.3 Subgrade Protection

The existing fill materials or native clayey silt (and any interlayers, if present) that will be exposed at the abutment and pier pile cap level will be susceptible to disturbance from construction traffic during piling operations, and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP, which can be developed during the detail design stage.

6.6.4 Driving Piles into Shale Slab/Cobble and Boulder Layer

Deep foundations are proposed to be driven to refusal in a very dense layer that has been interpreted to consist of shale slabs and limestone cobbles and boulders, based on the results from the 2014 investigation. The piles should be fitted with driving shoes and pile driving should be carefully controlled to minimize damage to the piles.

6.6.5 Conflicts with Existing Foundation Elements

As noted in Section 6.2, consideration will have to be given to potential conflicts between the new and existing foundation elements, particularly at the centre pier and north abutment based on the currently proposed alignment and layout for the replacement structure. In general, the existing pile caps may be left in place if permitted or removed where necessary to avoid conflicts with new foundation elements. Where existing piles are present within or near the footprint of new pile caps, the existing piles should be cut off below the level of the new pile cap; the existing piles should not be extracted.

6.6.6 Vibration Monitoring During Pile Installation

It is understood that the existing Seventh Street underpass will be demolished prior to construction of the replacement, so there will be no requirement for vibration monitoring of the existing structure. However, there is an industrial building in the southeast quadrant of the structure site, and the requirements for monitoring of vibrations at this site during construction should be evaluated during the detail design stage. An NSSP should be included in the Contract Documents at the detail design stage for a vibration monitoring plan that would include appropriate review and alert levels for vibrations for the existing building.

6.7 Recommendations for Further Work During Detail Design

A total of eleven boreholes have been advanced to "refusal" and, in some cases, cored into a cobble/boulder layer or shale bedrock, as part of Golder's 2013/2014 investigation and the previous 1966 investigation at this site. The surface elevation of the shale slab/cobble and boulder layer, to which deep foundations will be driven, is variable, and the boreholes are not located precisely at each of the foundation elements. If it is desired to reduce the risk of varying pile lengths, then additional borehole investigation at the proposed foundation elements may be warranted to confirm the pile tip elevations beyond the interpolated values given in this report.

It is recommended that the following items be assessed further in detail design:

- Further assessment of the potential conflicts between existing underground foundations/structures and the new abutments and piers.
- Further assessment of the settlement under the north approach embankment widening to confirm the magnitude of settlement and that the majority of settlement will be completed by the time the north abutment piles are driven, based on the planned construction method/schedule for the north abutment and approach.
- Assessment of vibration thresholds for the commercial/industrial building in the southeast quadrant, and if warranted development of an NSSP for a vibration monitoring plan.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Mehdi Mostakhdemi, M.Sc., P.Eng., with technical input and review by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 572	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP 105S21	Amendment to OPSS 501 – Construction Specification for Compacting

**TABLE 1 – COMPARISON OF FOUNDATION OPTIONS
SEVENTH STREET UNDERPASS**

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on stiff clayey silt deposit	• May be feasible but the geotechnical resistances are likely not high enough for support of abutments, piers, wingwalls/ retaining walls	• Limited groundwater control as excavation will be within relatively impermeable clayey silt deposit • Allows for semi-integral abutments	• Requires relatively deep excavations to approximately 2 m to 5 m below QEW grade, with associated temporary excavation support • Low geotechnical resistances as compared with deep foundations; potential for differential settlement between foundation elements • Precludes use of integral abutments; potentially greater maintenance required at abutments	• Conventional excavation and construction techniques, although deep foundations and protection systems would be required	• Less expensive than deep foundations, although must account for significant excavation depth and protection systems, plus bridge maintenance costs will be higher due to non-integral abutment configuration
Spread/strip footings perched on compacted granular pad in approach fill	• Not considered feasible at this site due to predicted settlement under new/widened embankment loading	• Abutment pile caps could be maintained higher than footings founded on clayey silt deposit, reducing depth of excavation and temporary excavation support requirements adjacent to existing Seventh Street embankment	• Up to about 70 mm to 90 mm of settlement predicted under north approach widening • Potential for differential settlement between abutments and pier due to settlement of soils under approach embankment loading • Precludes use of integral abutments; potentially greater maintenance required at abutments	• Conventional excavation and construction techniques	• Not assessed as this option is not considered appropriate at this site

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found on or in the shale bedrock	• Feasible for support of abutments, wing walls/retaining walls and piers	• Pier pile caps could be maintained higher than footings founded on clayey silt deposit, reducing depth of excavation and temporary excavation support requirements adjacent to existing Seventh Street embankment and QEW • Limited groundwater control required • Allows for integral abutment construction • Higher axial resistance than for shallow foundations	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles "hanging up" and lower geotechnical resistances	• Conventional construction methods for H-pile foundations	• Higher relative cost compared with spread/strip footing option • Lower relative cost compared with caisson option
Steel pipe (tube) piles, driven to found on or in shale bedrock	• Feasible for support of new abutments, wing walls/retaining walls, and piers	• Abutment pile caps could be maintained higher than footings founded on clayey silt, reducing depth of excavation and temporary protection system requirements adjacent to Seventh Street and QEW • Limited groundwater control required • Allows for semi-integral abutment configuration • Would minimize differential settlement between foundation elements	• Greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in piles "hanging up" and lower geotechnical resistances	• Conventional construction methods	• Higher relative cost compared with spread/strip footing option • Costs for steel pipe (tube) piles slightly higher than for steel H-piles

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded in shale bedrock	• Feasible but not recommended for support of abutments, wing walls/retaining walls and centre pier	• Abutment pile caps could be maintained higher than footings founded on clayey silt deposit, reducing depth of excavation and temporary excavation support requirements adjacent to existing Seventh Street embankment • Higher capacity than for steel H-piles or pipe piles, so reduced number of deep foundation elements compared to steel piles	• Potential for loss of ground in water-bearing cohesionless deposits • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Conventional construction methods with temporary liners required	• Higher cost compared with shallow foundations or steel H-piles

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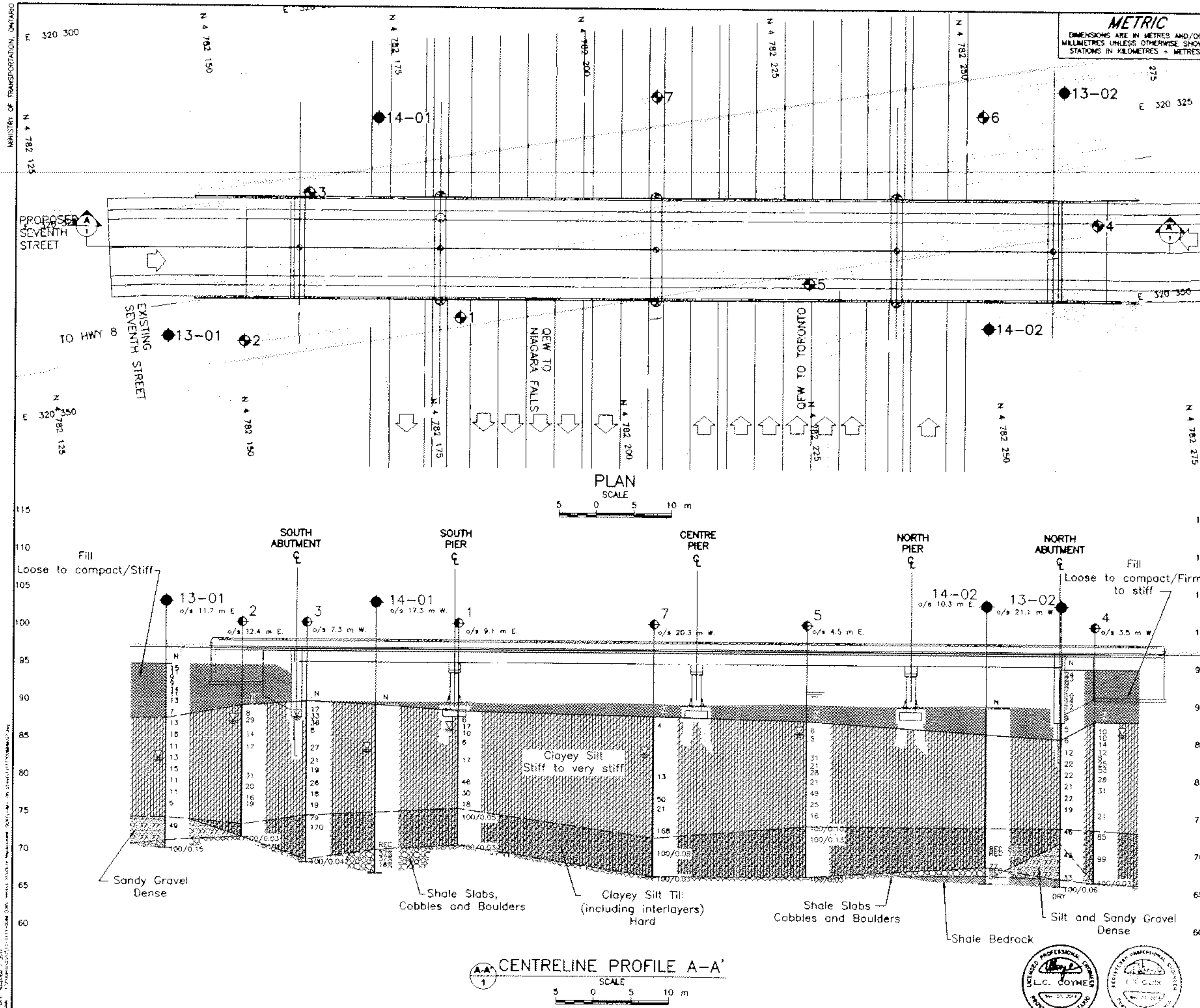

CONT No.
WP No. 2177-08-00

SEVENTH STREET UNDERPASS
QUEEN ELIZABETH WAY
BORHOLE LOCATIONS AND SOIL STRATA

SHEET

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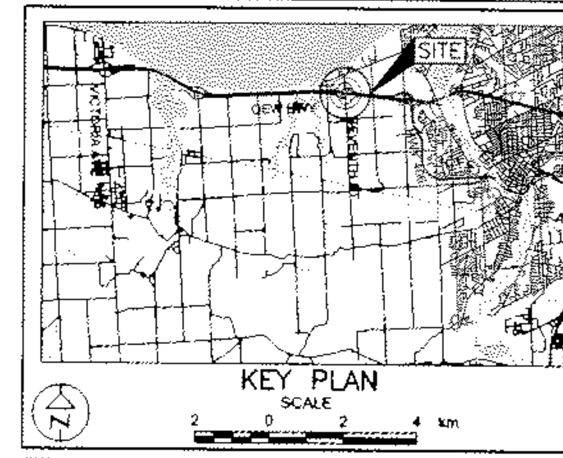


LEGEND

- Borehole – Current Investigation
- Borehole – 1996 Investigation – Location is approximate
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 0% Rock Quality Designation (RQD)
- WL upon completion of drilling
- REC Recovery

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
13-01	94.7	4782141.5	320340.5
13-02	94.8	4782263.8	320321.3
14-01	89.5	4782172.5	320314.8
14-02	89.5	4782250.3	320351.4

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 Preliminary Design Report.

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

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

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base-Ak.dwg and X-Contours.dwg, received July 30, 2013 on GA and Profile file No. Draft_Seventh Street_GA.dwg, received August 6, 2013.

NO.	DATE	BY	REVISION

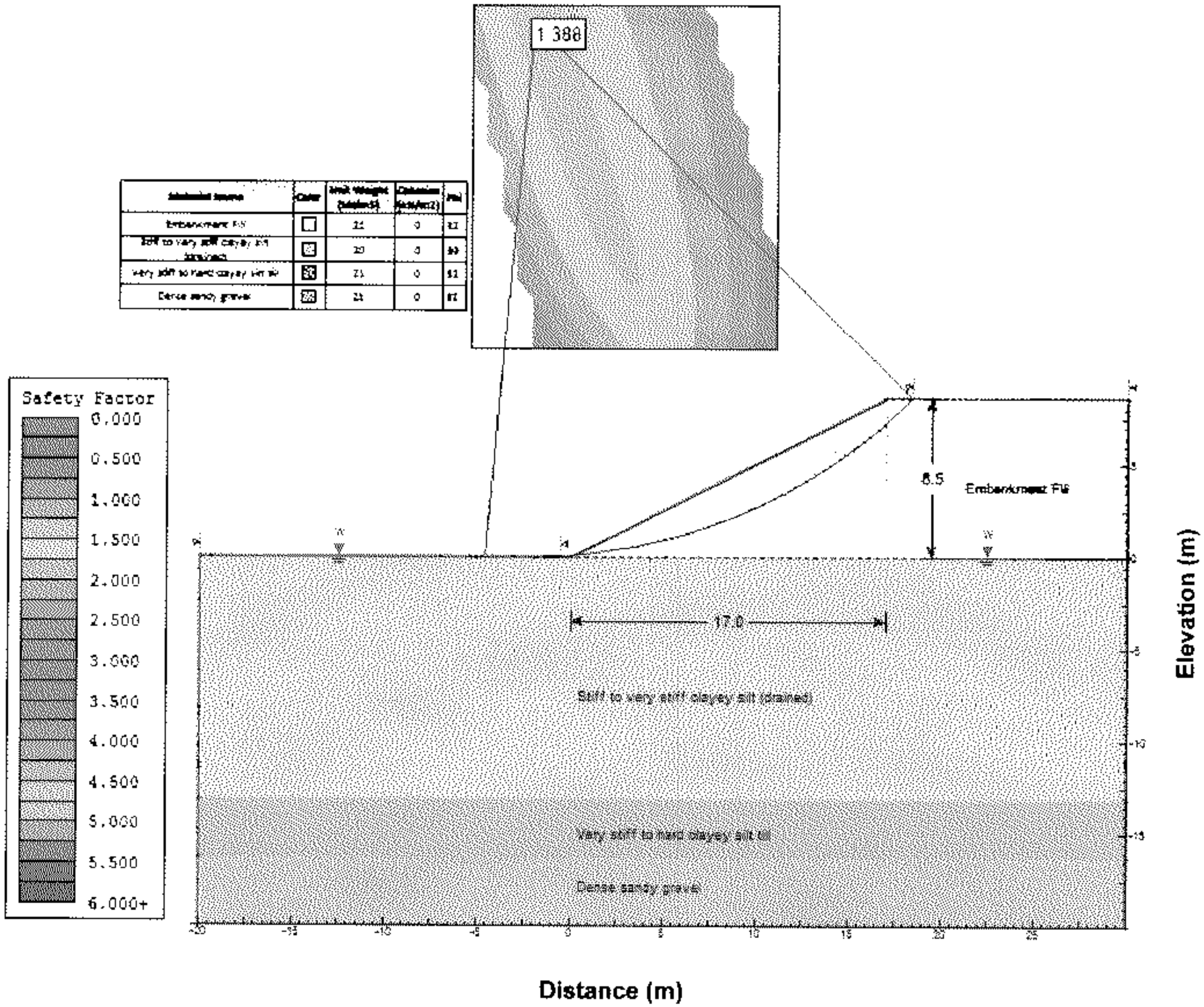
Geocres No. 30M3-278

HWY. QEW		PROJECT NO. 12-1111-0088	DIST. CENTRAL
SUBM'D. ARV	CHKD. LCC	DATE: 11/7/2014	SITE:
DRAWN: JFC	CHKD. LCC	APPD. TJG	DWG. 1

L.C. COYNE

Static Global Stability – Seventh Street South Approach Embankment

Figure 1



Date: September 2013
Project No: 12-1111-0088-1

Analysis By: MM **Reviewed By:** TJG

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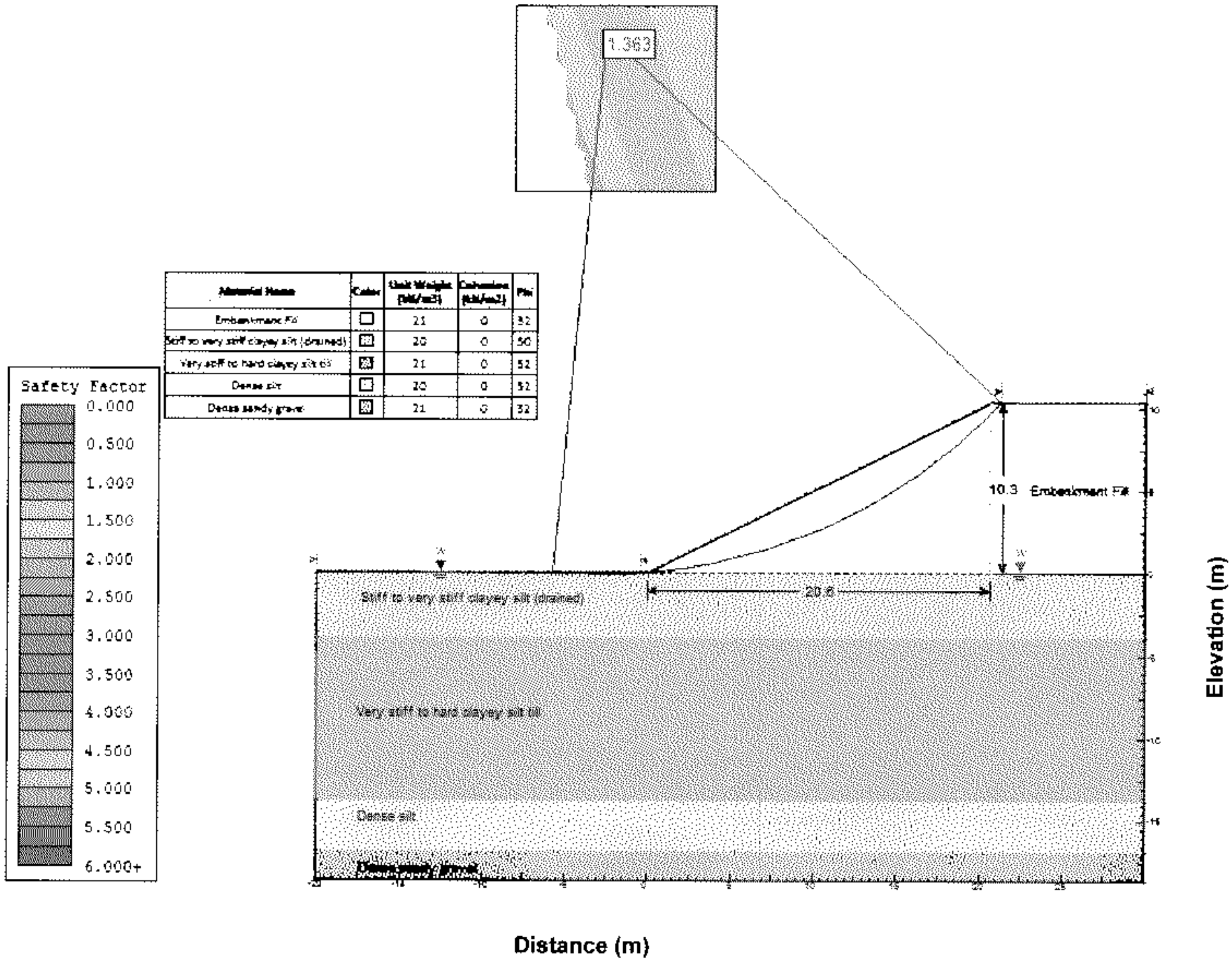
Static Global Stability – Seventh Street North Approach Embankment

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Embankment Fill		21	0	32
Stiff to very stiff clayey silt (drained)		20	0	30
Very stiff to hard clayey silt till		21	0	32
Dense silt		20	0	32
Dense sandy gravel		21	0	32

Date: September 2013
Project No: 12-1111-0088-1

Analysis By: MM **Reviewed By:** TJG

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APPENDIX A

Borehole Records

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ. Unit weight symbol is γ 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 (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c'	effective cohesion
c_u, 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. shear strength = (compressive strength)/2

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
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	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

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

CORE CONDITION

Total Core Recovery (TCR)

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 varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

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

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

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Golder Associates

RECORD OF BOREHOLE No 13-02

SHEET 1 OF 3 — METRIC

PROJECT 12-1111-0088
G.W.P. 2177-08-00
LOCATION N 4782263.6 ;E 320321.3
ORIGINATED BY SB
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY AV
DATUM Geodetic
DATE April 22, 2013
CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT / REMARKS (GR SA SI CL)
94.8	GROUND SURFACE				
0.0	ASPHALT				
94.5 0.3	Sand and gravel, some silt, trace to some clay (FILL) Compact to loose Brown Moist	1	SS	24	
		2	SS	23	
		3	SS	9	32 41 20 7
92.4 2.4	Clayey silt, trace to some sand, trace gravel (FILL) Firm to stiff Brown Moist	4	SS	7	
		5	SS	10	
		6	SS	14	0 9 70 21
90.3 4.5	Silt and sand, trace gravel, containing silty clay pockets (FILL) Compact Brown Moist	7	SS	12	
89.2 5.6	Clayey silt with sand, trace gravel, containing rootlets (FILL) Stiff to firm Brown Moist	8	SS	9	
		9	SS	5	
86.1 8.7	Silt and sand, trace clay, containing rootlets (FILL) Loose Brown Moist	10A	SS	6	
85.4 9.4	CLAYEY SILT, trace sand, trace gravel Firm to stiff Grey to brown Moist	10B			
		11	SS	12	
		12	SS	22	
81.5 13.3	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Grey Moist	13	SS	22	1 22 63 14

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (94 to 80); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Field vane values plotted near elev. 84.5–85: 3, 3

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2012\12-1111-0088 (URS, VARIOUS STRUCTURE REPLACEMENT, QEW)\LOG\12-1111-0088.GPJ GAL-GTA.GDT 11/07/14

RECORD OF BOREHOLE No 13-02

METRIC

PROJECT 12-1111-0088

G.W.P. 2177-08-00 LOCATION N 4782263.6 ;E 320321.3 ORIGINATED BY SB

DIST Central HWY QEW BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers COMPILED BY AV

DATUM Geodetic DATE April 22, 2013 CHECKED BY LCC

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —						
	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Grey Moist	14	SS	21	79		
		15	SS	22	78	H	
		16	SS	19	77, 76		
					75, 74		
		17	SS	46	73	H	4 23 50 23
71.6 23.2	SILT, some sand Dense Grey Moist to wet				72, 71		
		18	SS	43	70, 69		
68.6 26.2	Sandy GRAVEL, trace clay, trace silt, containing shale fragments Dense Red-brown Wet	19	SS	33	68, 67		
66.8 28.0	Shale (bedrock slab or cobble/boulder layer) Weathered Reddish brown to grey				66		
65.8 29.0		20	SS	100/0.05			

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

Continued Next Page

3, 3 Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

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RECORD OF BOREHOLE No 13-02

METRIC

PROJECT 12-1111-0088
G.W.P. 2177-08-00
LOCATION N 4782263.6 ;E 320321.3
ORIGINATED BY SB
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY AV
DATUM Geodetic
DATE April 22, 2013
CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	— CONTINUED FROM PREVIOUS PAGE —										
	END OF BOREHOLE NOTE: 1. Borehole dry on completion of drilling										

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

RECORD OF BOREHOLE No 14-01

SHEET 1 OF 2 METRIC

PROJECT 12-1111-0088

G.W.P. 2177-08-00 LOCATION N 4782172.5 ;E 320314.8 ORIGINATED BY RA

DIST Central HWY QEW BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, 102 mm O.D. Tricone and NQ Core Barrel COMPILED BY ARV

DATUM Geodetic DATE August 13 to 14, 2014 CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
89.5	GROUND SURFACE										
0.0	No soil sample was taken in Borehole 14-01 from ground surface to a depth of 19.2 m. For general soil descriptions, refer to Record of Borehole 13-01						89 88 87 86 85 84 83 82 81 80 79 78 77 76 75				

Continued Next Page

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

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RECORD OF BOREHOLE No 14-01

SHEET 2 OF 2 **METRIC**

PROJECT 12-1111-0088
G.W.P. 2177-08-00
LOCATION N 4782172.5 ;E 320314.8
ORIGINATED BY RA
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, 102 mm O.D. Tricone and NQ Core Barrel
COMPILED BY ARV
DATUM Geodetic
DATE August 13 to 14, 2014
CHECKED BY LCC

-- CONTINUED FROM PREVIOUS PAGE --

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	No soil sample was taken in Borehole 14-01 from ground surface to a depth of 19.2 m. For general soil descriptions, refer to Record of Borehole 13-01				74, 73, 72, 71
70.2		1	SS	*	
70.2 19.4	Limestone COBBLES and BOULDERS, containing shale pieces	2	RC	REC 37%	70
		3	RC	REC 37%	69
	Layers of coarse sand encountered at a depth of 20.7 m.	4	RC	REC 16%	68
67.0 22.5	END OF BOREHOLE				67

NOTES:

1. *Split-Spoon sampler refusal.
2. Drillers experienced difficulty coring through the layer of cobbles and boulders.

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

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 14-02

SHEET 1 OF 2

METRIC

PROJECT 12-1111-0088
G.W.P. 2177-08-00
LOCATION N 4782250.3 ;E 320351.4
ORIGINATED BY JL
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers and NQ Core Barrel
COMPILED BY ARV
DATUM Geodetic
DATE July 28 to 30, 2014
CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_p / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_l / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
89.5	GROUND SURFACE										
0.0	No soil sample was taken in Borehole 14-02 from ground surface to a depth of 18.9 m. For general soil descriptions, refer to Record of Borehole 13-02						89, 88, 87, 86, 85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75				

Continued Next Page

3, 3. Numbers refer to Sensitivity

3% STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\2012\12-1111-0088 (URS, VARIOUS STRUCTURE REPLACEMENT, QEW)\LOG\12-1111-0088.GPJ GAL-GTA.GDT 11/07/14

Golder Associates

RECORD OF BOREHOLE No 14-02

SHEET 2 OF 2 **METRIC**

PROJECT 12-1111-0088

G.W.P. 2177-08-00 LOCATION N 4782250.3 ;E 320351.4 ORIGINATED BY JL

DIST Central HWY QEW BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers and NQ Core Barrel COMPILED BY ARV

DATUM Geodetic DATE July 28 to 30, 2014 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---							
	No soil sample was taken in Borehole 14-02 from ground surface to a depth of 18.9 m. For general soil descriptions, refer to Record of Borehole 13-02						74	
							73	
							72	
71.2 / 18.3	CLAYEY SILT, containing shale fragments (TILL)		1	SC	REC 60%		71	
	No recovery in the core barrel between depths of 18.9 m and 20.4 m.		2	SC	REC 15%			
							70	
							69	
			3	SS	72			
68.3 / 21.2	COBBLES and BOULDERS (Limestone)		4	RC	REC 21%		68	
67.8 / 21.7	Shale pieces/slabs							
67.2 / 22.3	Shale (BEDROCK)						67	
	Bedrock cored between depths of 22.3 m to 23.3 m		1	RC	REC 36%			RQD = 0%
66.2 / 23.3	Refer to Record of Drillhole 14-02 for bedrock coring details.							
	END OF BOREHOLE							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

NOTEs:

1. On July 28, 2014 coarse sand stuck in the inner tube during coring in the residual soil layer. The coring operation was terminated due to time constraints and continued on the following night (July 29, 2014).

2. The drillers experienced difficulty in drilling/coring through the residual soil, cobbles and boulders, and switched between augering and rock coring techniques.

3. On July 30, 2014, the augers were advanced to refusal at a depth of 22.3 m and coring was started.

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

RECORD OF DRILLHOLE: 14-02

PROJECT: 12-1111-0088
LOCATION: N 4782250.3 ;E 320351.4
INCLINATION: -90° AZIMUTH: --

DRILLING DATE: July 28 to 30, 2014
DRILL RIG: CME 75 Truck-Mount
DRILLING CONTRACTOR: Walker Drilling

SHEET 1 OF 1
DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	ROCK STRENGTH INDEX	WEATHERING INDEX	FEATURES	PIEZOMETER
		Continued from Record of Borehole 14-02		67.20												
23	NQRC	Slightly weathered, red, weak to medium strong Shale		22.30	1											
		END OF BOREHOLE		66.20 23.30												
24																
25																
26																
27																
28																
29																
30																
31																
32																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: JL
CHECKED: LCC

GTA-RCK 043 T:\PROJECTS\2012\12-1111-0088 (URS, VARIOUS STRUCTURE REPLACEMENT, QEW)\LOG\12-1111-0088.GPJ GAL-MISS.GDT 11/07/14

APPENDIX B

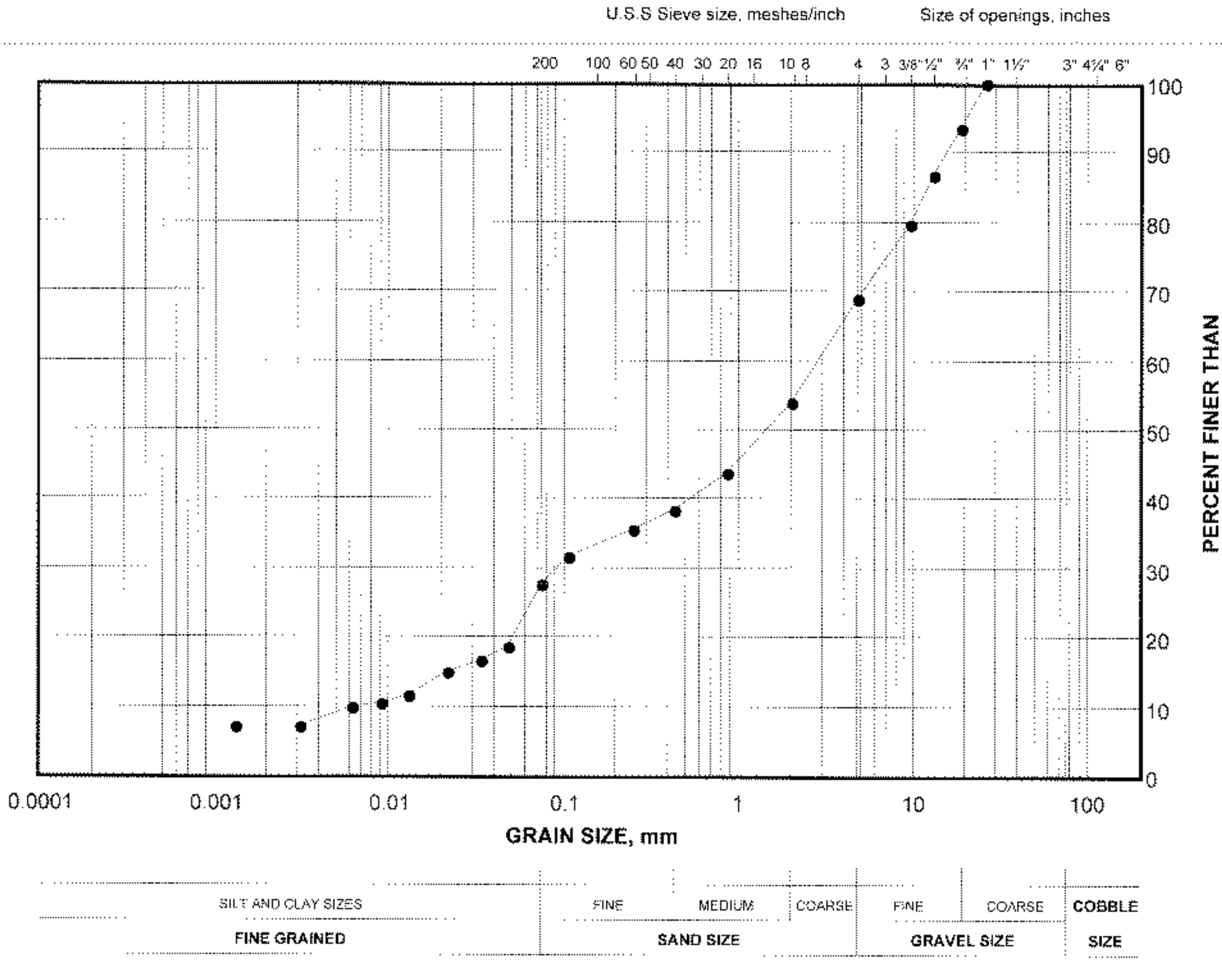
Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sand and Gravel Fill

Seventh Street Underpass, GWP 2177-08-00

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-02	3	93.0

Project Number: 12-1111-0088-1

Checked By: MM

Golder Associates

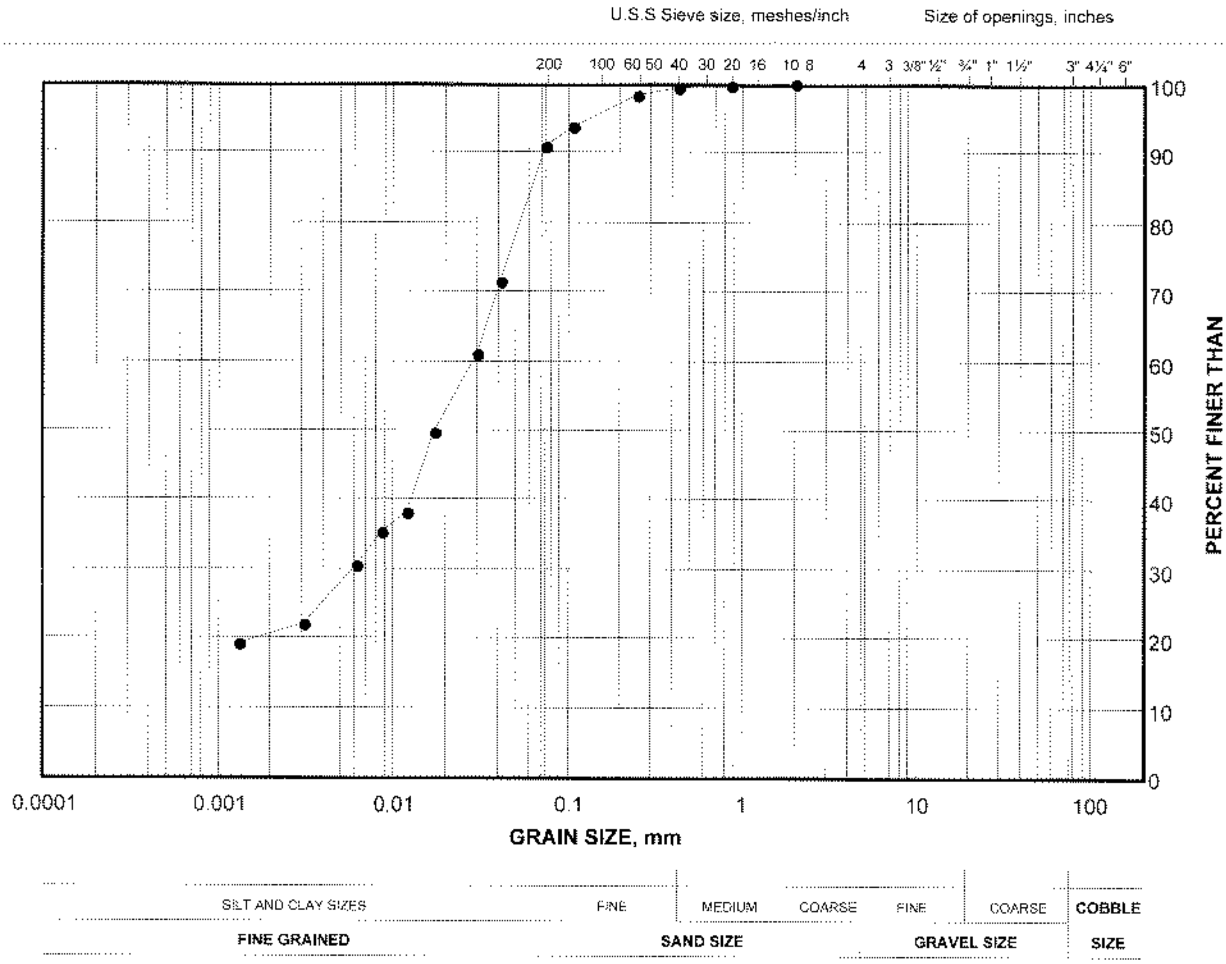
Date: 07-Aug-13

GRAIN SIZE DISTRIBUTION

Clayey Silt Fill

Seventh Street Underpass, GWP 2177-08-00

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-02	6	90.7

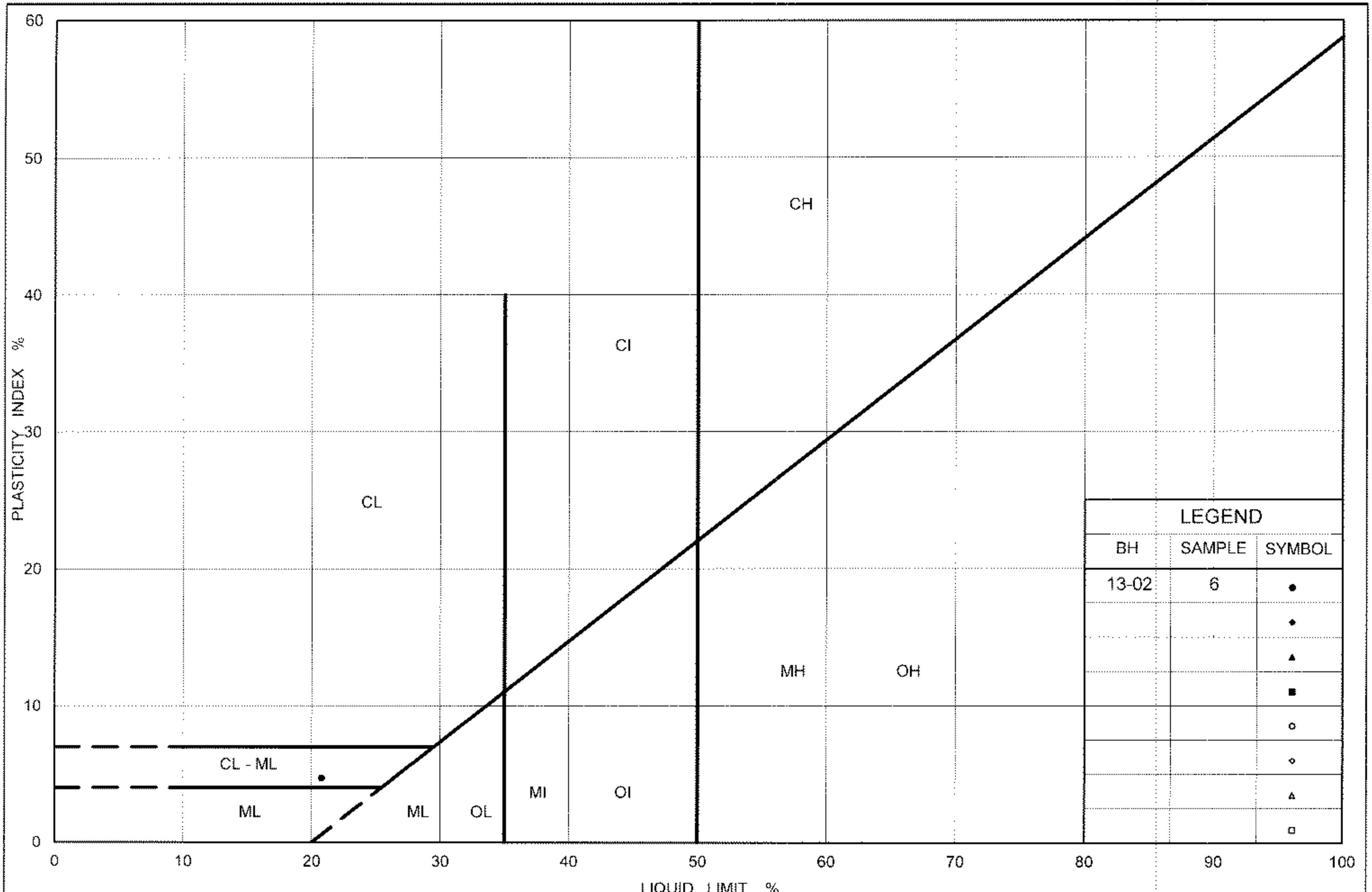
Project Number: 12-1111-0088-1

Checked By: MM

Golder Associates

Date: 07-Aug-13

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt Fill

Seventh Street Underpass, GWP 2177-08-00

Figure No. B3

Project No. 12-1111-0088-1

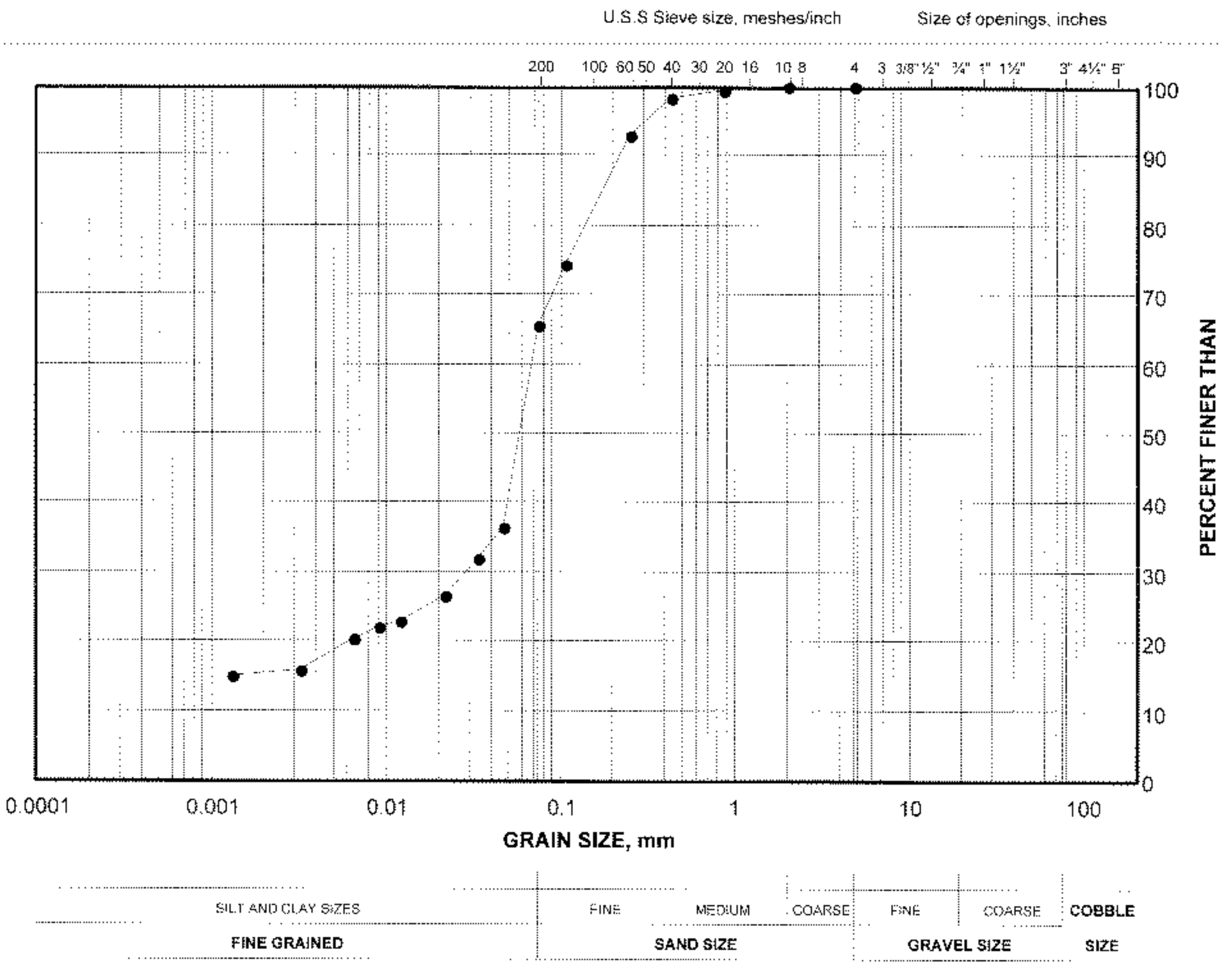
Checked By: MM

GRAIN SIZE DISTRIBUTION

Silt and Sand Fill

Seventh Street Underpass, GWP 2177-08-00

FIGURE B4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-01	8	88.3

Project Number: 12-1111-0088-1

Checked By: MM

Golder Associates

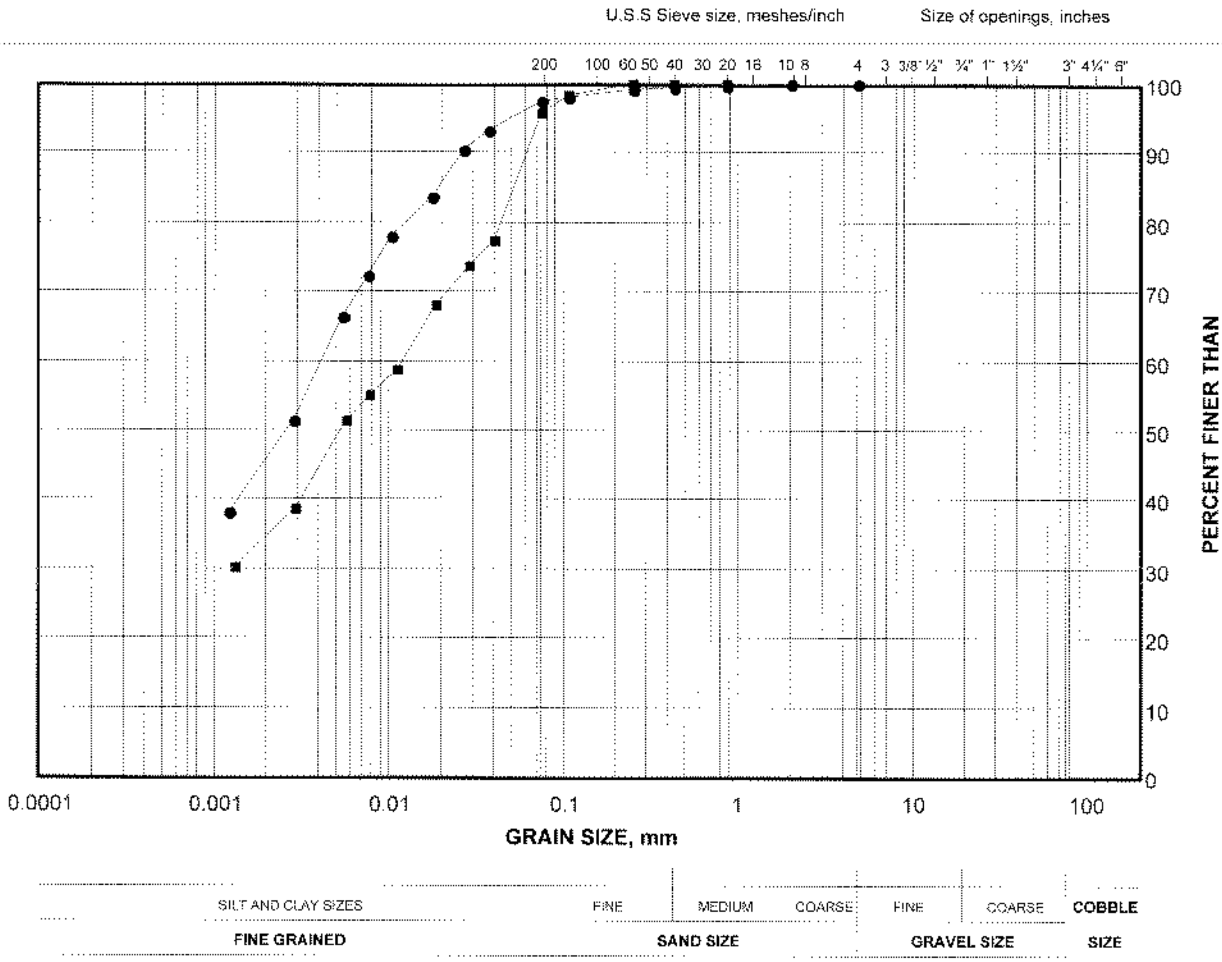
Date: 07-Aug-13

GRAIN SIZE DISTRIBUTION

Clayey Silt

Seventh Street Underpass, GWP 2177-08-00

FIGURE B5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-01	14	79.2
■	13-01	9	86.8

Project Number: 12-1111-0088-1

Checked By: MM

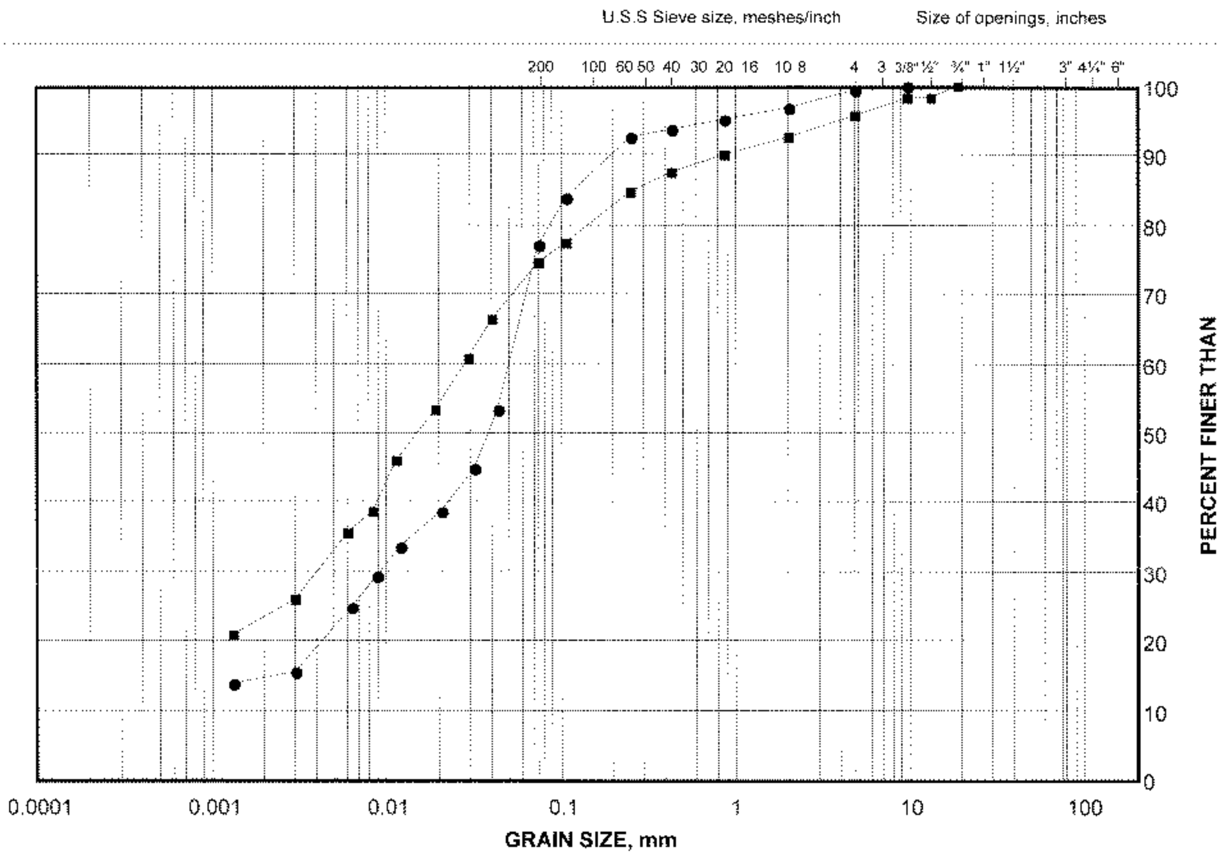
Golder Associates

Date: 07-Aug-13

GRAIN SIZE DISTRIBUTION

Clayey Silt Till
Seventh Street Underpass, GWP 2177-08-00

FIGURE B7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-02	13	80.8
■	13-02	17	73.2

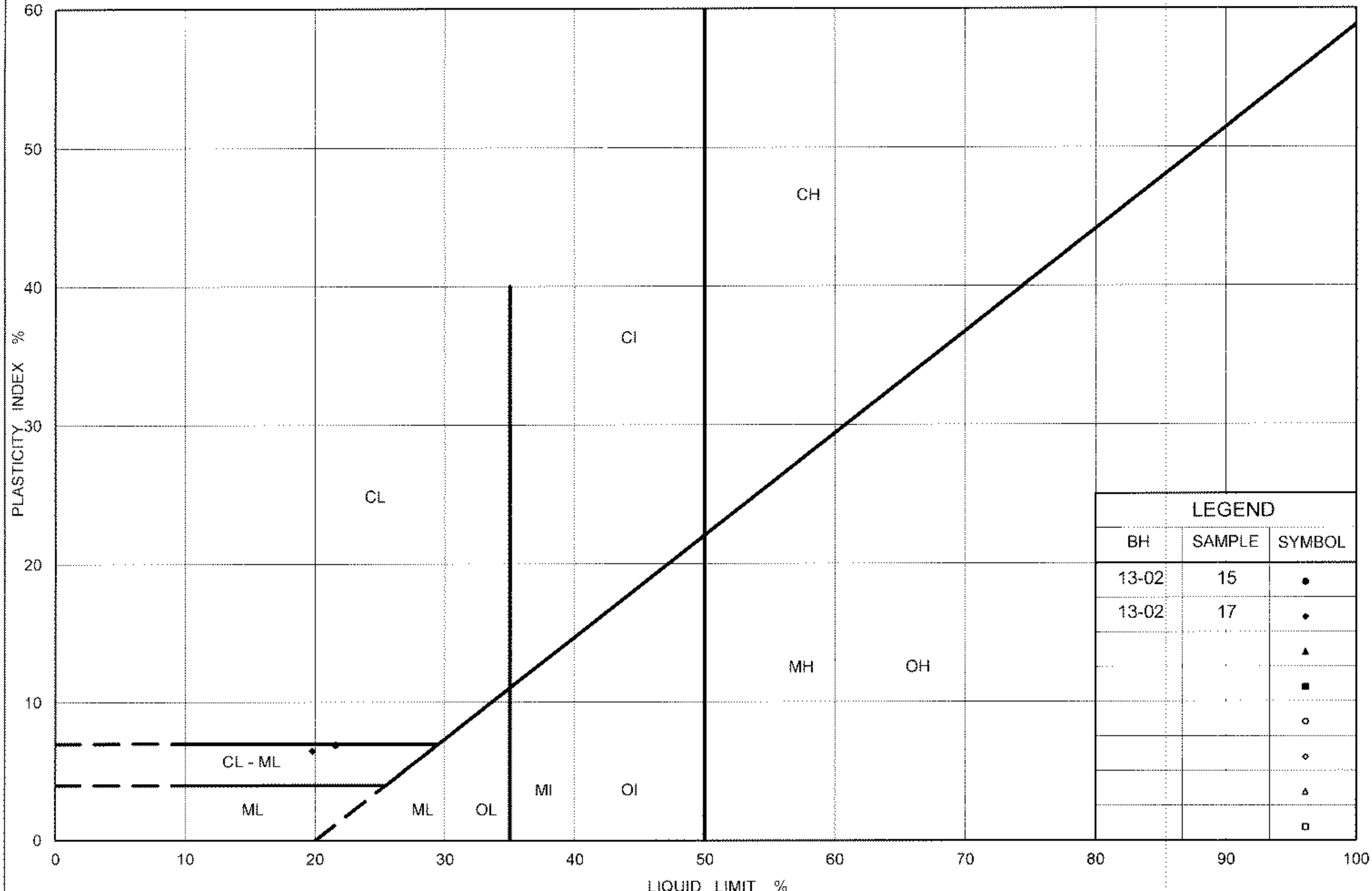
Project Number: 12-1111-0088-1

Checked By: MM

Golder Associates

Date: 07-Aug-13

Oct 75, FF-S-21



Ministry of Transportation Ontario	**PLASTICITY CHART** Clayey Silt Till Seventh Street Underpass, GWP 2177-08-00	Figure No. B8 Project No. 12-1111-0088-1 Checked By: MM

APPENDIX C

Records of Boreholes from Previous Investigation (GEOCRES No. 30M03-016)

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-65 — LOCATION CBK - 7th St. Sta. 30/90. O/S 33' Lt. — ORIGINATED BY V.K.

W.P. 212-63 — BORING DATE June 21, 1966 — COMPILED BY V.K.

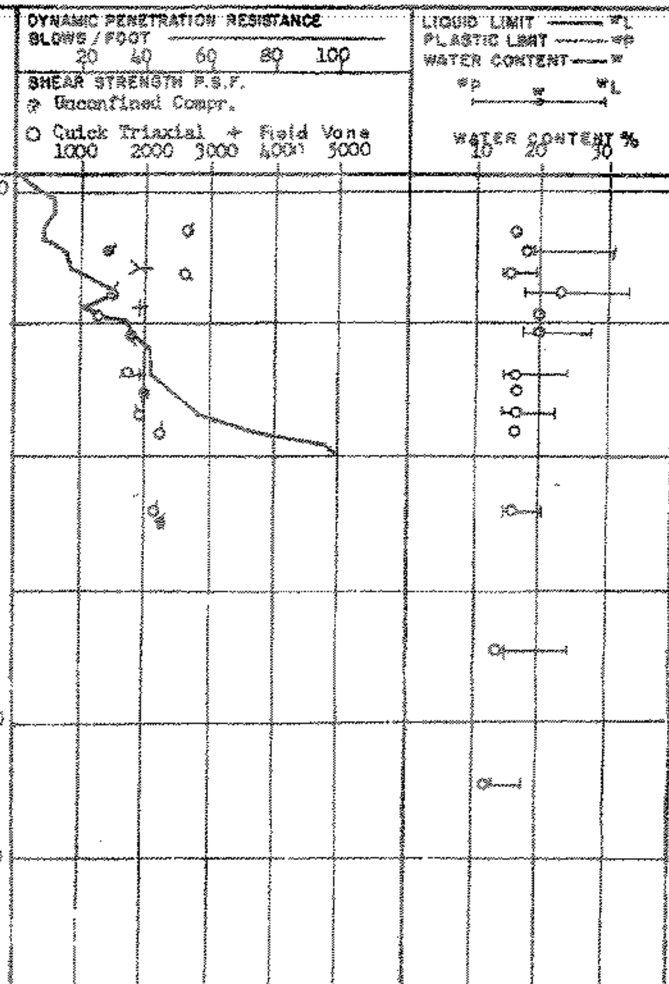
DATUM Geodetic — BOREHOLE TYPE Penndrill — CHECKED BY M.D.

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
291.0	88.7m Groundlevel
87.3m 286.5 / 4.5	Silt to clayey silt & sand with organics - firm.
75.7m 248.5 / 42.5	Clayey silt, traces of sand and gravel. Stiff to hard.
70.6m 232.4 / 58.6	Heterogeneous mixture of clayey silt, sand and gravel. (Glacial Till)
	Weathered Shale
	End of borehole.

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	6
1A	TW	P
2	SS	17
2A	TW	P
3	SS	10
3A	TW	P
4	SS	6
5	TW	P
6	TW	P
7	SS	17
8	TW	P
9	SS	46
10	SS	30
11	SS	12
12	SS	100 for 2"
13	SS	100 for 1"



BULK DENSITY γ P.C.F.

134, 127, 134, [illegible], [illegible], 126, 135, 135, 135, 135, 135

REMARKS

Sa10%Si180% Cl 10%

W.L. El. 283.0

Sa4%Si54% Cl 42%

Gr1%Sa11% Si53%Cl 35%

Gr1%Sa21% Si51%Cl 27%

Refusal

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

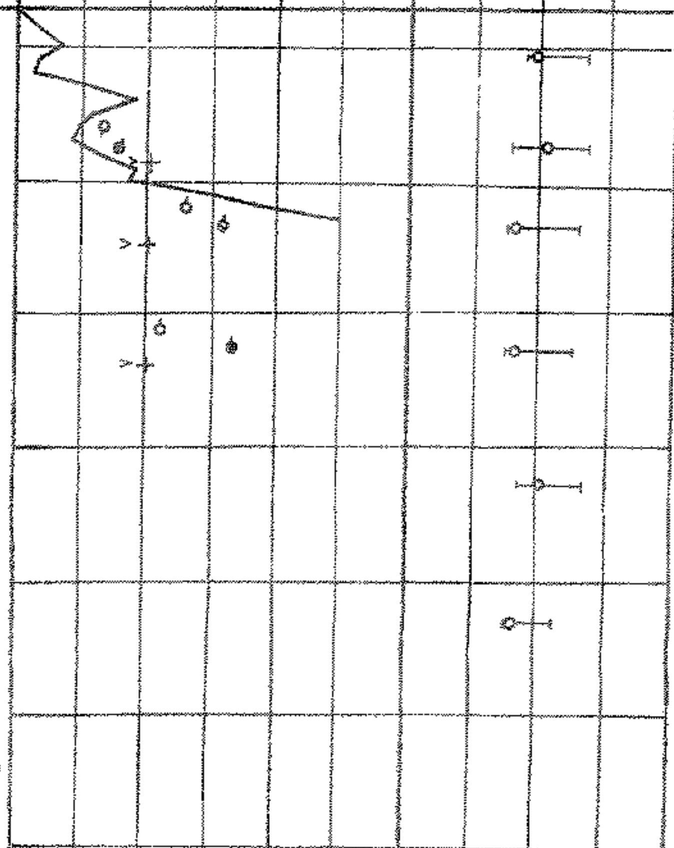
RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-45 LOCATION QEW - 7th St. Sta. 31/85, 29' Lt. ORIGINATED BY V.K.
W.P. 212-63 BORING DATE June 22, 1966 COMPILED BY V.K.
DATUM Geodetic BOREHOLE TYPE Penndrill CHECKED BY M.D.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS
89.3m 293.0	Groundlevel						
	Silt to clayey silt and sand, with organics - compact.	1	SS	8	290		
		2	SS	29			
86.6m 284.0 / 9.0	Clayey silt, traces of sand and gravel. (Stiff to hard)	3	TW	P		124 130	▽ WL El. 286 Sa1% Si63% Cl 36%
		4	SS	14	280		
		5	TW	P		136 136	
		6	SS	17			
					270		
		7	TW	P		134 137	Gr 2% Sa 10% Si57% Cl 31%
		8	SS	31	260		
		9	SS	20			
		10	SS	16	250		
		11	SS	19			
73.5m 241.0 / 52.0	Heterogeneous mixture of clayey silt, sand & gravel. (Glacial Till)				240		
235.5 / 57.5	Weathered Shale	12	SS	100 for 1"			
71.8m	End of borehole.				230		Refusal

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ● Unconfined Compr. ○ Quick Triaxial + Field Vane 1000 2000 3000 4000 5000
LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w
WATER CONTENT % 10 20 30



OFFICE REPORT ON SOIL EXPLORATION

FORM 08-MT-186 (FORMERLY 08-ML-226)

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-65 LOCATION QEW - 7th St. Sta. 31/47, O/S 30.5' Rt. ORIGINATED BY V.K.

W.P. 212-63 BORING DATE June 23, 1966 COMPILED BY V.K.

DATUM Geodetic BOREHOLE TYPE Penndrill CHECKED BY M.D. ///

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
84.9m 295.0	Groundlevel						
	Silt to clayey silt & sand with organics. Very stiff		1	SS	17	290	Gr6%Sa70% Si&Cl 24%
87.3m 286.5			2	SS	33		WL El. 288.0
8.5			3	SS	36		
			4	SS	8	280	
			5	TW	P		
			6	SS	27		
	Clayey silt, traces of sand and gravel. (Stiff to hard)		7	SS	21	270	Sa 12% Si 58% Cl 30%
			8	SS	19		
			9	SS	26	260	
			10	SS	18		
			11	SS	19	250	
74.7m 245.0							
50.0	Heterogeneous mixture of clayey silt, sand and gravel. - hard. (Glacial Till)		12	SS	79		Gr 36% Sa 49% Si&Cl 16%
			13	SS	170	240	
						230	
68.8m 225.6	Weathered Shale		14	SS	100 for 1.5"		
70.4	End of borehole.					220	Refusal

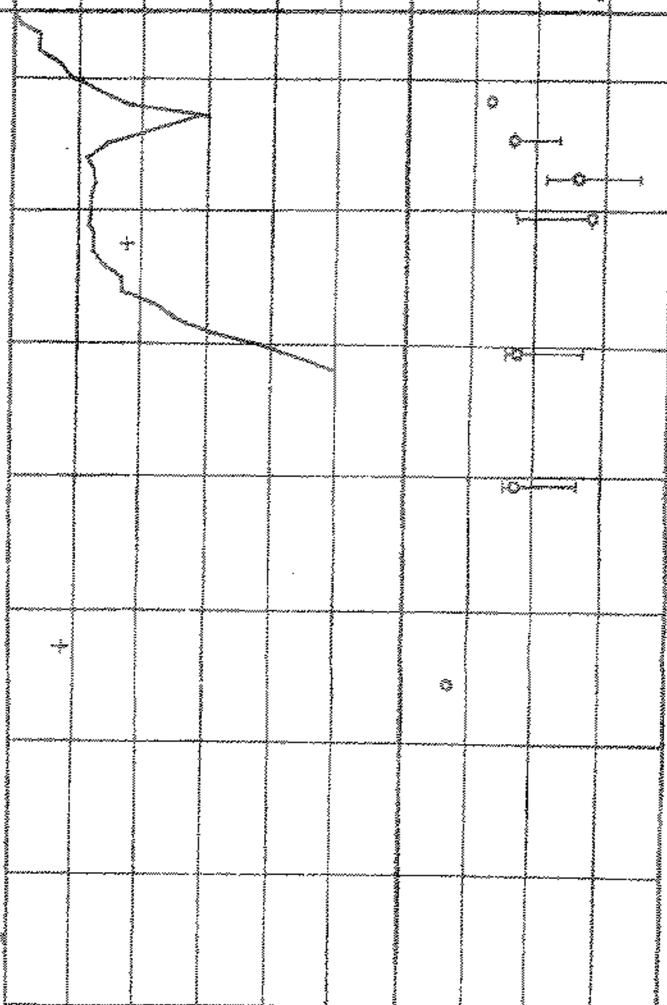
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. + Field Vane. 1000 2000 3000 4000 5000

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.



DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

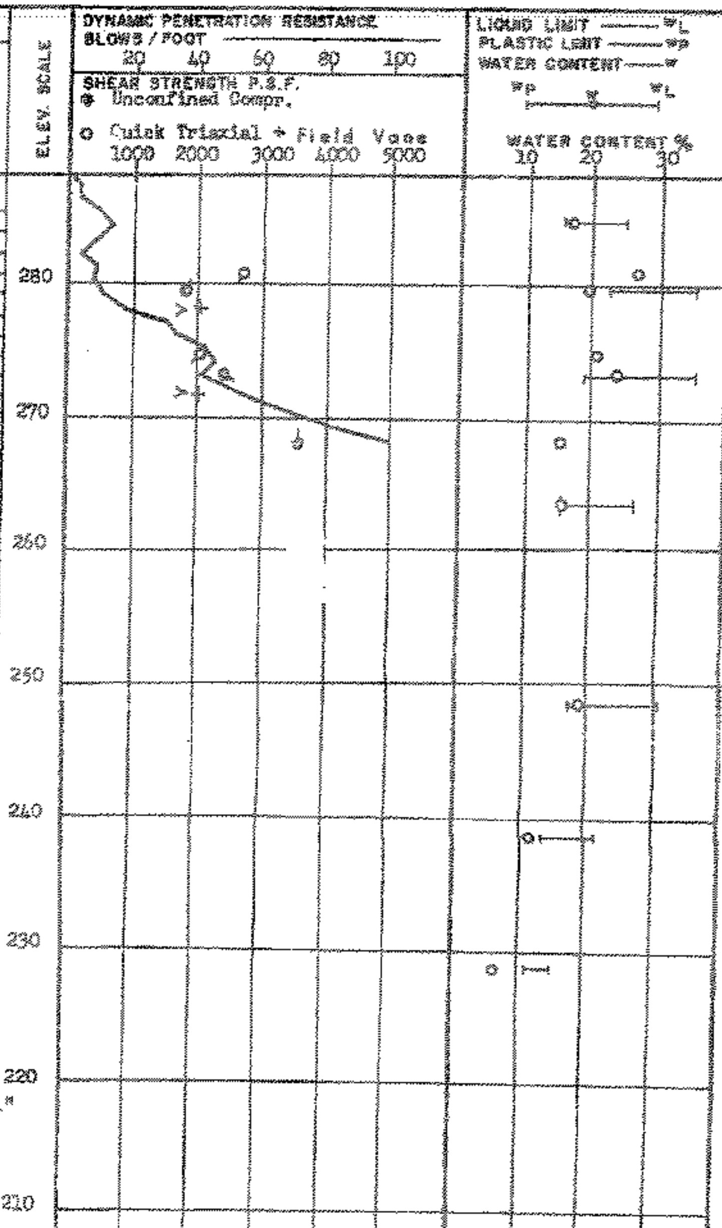
JOB 66-F-65	LOCATION QEW - 7th St. Sta. 28+09, O/S 35' Lt.	ORIGINATED BY V.K.
W.P. 212-63	BORING DATE June 24, 1966.	COMPILED BY V.K.
DATUM Geodetic	BOREHOLE TYPE Penndrill	CHECKED BY M.D.

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
87.8m 288.0	Groundlevel
86.3m 283.0	Silt to clayey silt & sand & organics - stiff
5.0	Clayey silt, traces of sand and gravel. (Stiff to hard)
71.0m 233.0 55.0	Heterogeneous mixture of clayey silt, sand and gravel. - hard (Glacial Till)
66.3m 217.5	Weathered Shale
70.5	End of borehole.

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	10
2	SS	10
2A	TW	P
3	SS	14
4	SS	12
4A	TW	P
5	SS	8
6	SS	25
6A	TW	P
7	SS	53
8	SS	28
9	SS	31
10	SS	21
11	SS	85
12	SS	89
13	SS	100/1"



BULK DENSITY γ P.C.F. / REMARKS

BULK DENSITY γ P.C.F.	REMARKS
	Sa60%Si25% Cl 15%
WL	▽ El 282.5
123 127	Sa2%Si50% Cl 48%
126 126	
138	Gr1%Sa10% Si57%Cl 32%
	Gr3%Sa10% Si56%Cl 31%
	Sa2%Si69% Cl 29%
	Gr18%Sa42% Si33%Cl 7%
	Refusal

68-1881

DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 66-F-65 LOCATION CBW - 7th St. Sta. 29/37, D/S 41.5' Lt. ORIGINATED BY V.K.

W.P. 212-63 BORING DATE June 28, 1966 COMPILED BY V.K.

DATUM Geodetic BOREHOLE TYPE Penndrill CHECKED BY M.D.

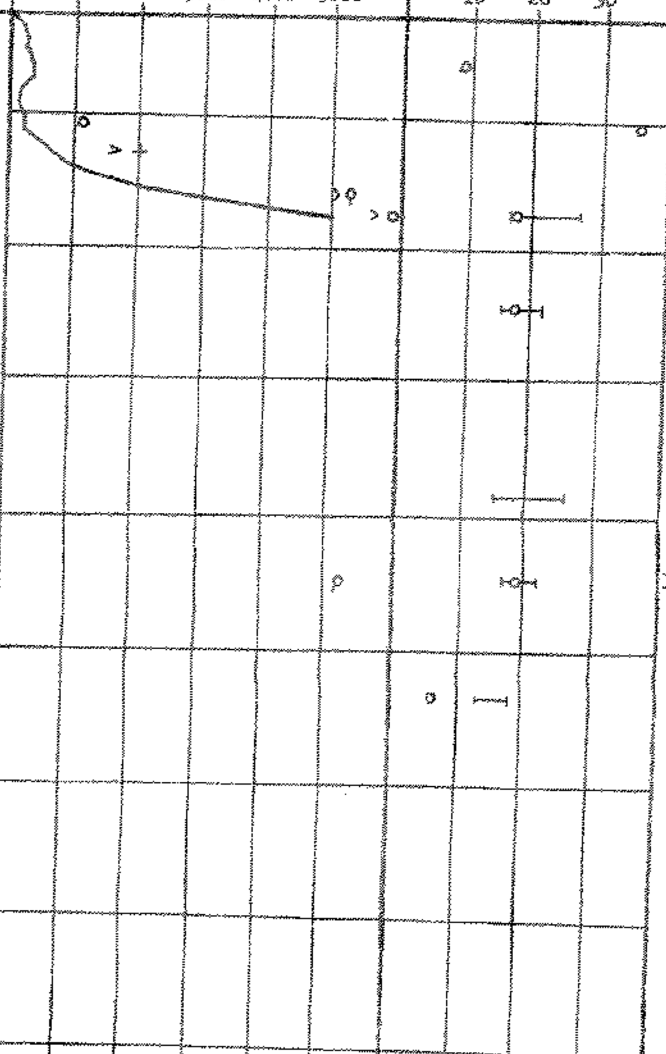
ELEV. DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY P.C.F.	REMARKS
87.6m 287.5	Groundlevel						
86.1m 282.5	Silt to clayey silt & sand with organics – firm	1	SS	6			Gr2%Sa68% Si&Cl 30%
5.0		2	SS	5	280		WL El. 282.0
		3	TW	P		110	Org. 3.5%
		4	TW	P		135	Gr1%Sa13%
		5	SS	31	270	135	Si54%Cl 32%
		6	SS	21			
		7	SS	28			
	Clayey silt, traces of sand and gravel.	8	SS	21	260		
	(V. stiff to hard)	9	SS	49			Sa25%Si56% Cl 19%
		10	SS	25	250		
		11	SS	16			
73.6m 241.5		12	TW	P		135	Sa2%Si82% Cl 16%
46.0	Heterogeneous mixture of clayey silt, sand and gravel – hard (Glacial Till)	13	SS	100 for 4"	240		
		14	SS	100 for 5"			Gr29%Sa45% Si20%Cl 6%
66.9m 219.5	Weathered Shale				230		
68.0	End of borehole.	15	SS	100 for 1"	220		Refusal
					210		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ● Unconfined Compr. ○ Quick Triaxial + Field Vane 1000 2000 3000 4000 5000

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT % 10 20 30



FORM CB-MT-126 (FORMERLY CB-ML-126) 68-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

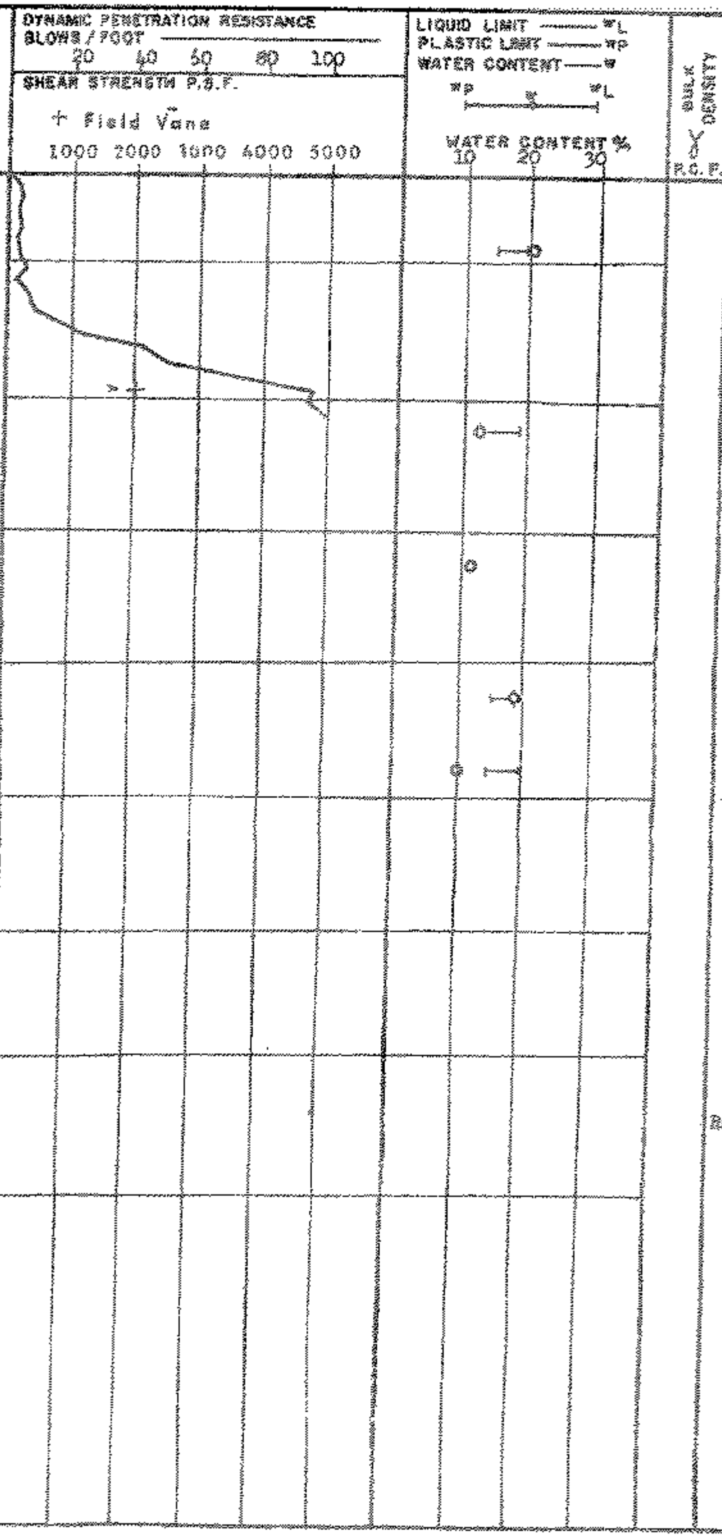
FOUNDATION SECTION

JOB 66-F-65 LOCATION QEW - 7th St. Sta. 28/51 O/S 19' Rt. ORIGINATED BY V.K.

W.P. 212-63 BORING DATE June 29, 1966. COMPILED BY V.K.

DATUM Geodetic BOREHOLE TYPE Penndrill CHECKED BY M.D.

ELEV. DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
87.3m 286.5	Groundlevel					
0.0	Silt to clayey silt and sand, with organics - firm.					Sa26%Si58% Cl 16%
75.7m		1	SS	5	280	W.L. El. 280.5
278.5 / 8.0		2	SS	17		
		3	TW	P	270	
	Clayey silt and sand, traces of gravel.	4	SS	30		Sa18%Si60% Cl 22%
		5	SS	33	260	
	(V. stiff to hard)	6	SS	39		Gr1%Sa42% Si47%Cl 10%
		7	SS	35	250	
74.5m		8	SS	26		
244.5 / 42.0	Heterogeneous mixture of clayey silt, sand and gravel.	9	SS	150	240	
	Glacial Till	10	SS	103	230	
66.1m					220	
217.0	Weathered Shale	11	SS	102 for 1"		
69.5	End of borehole.				210	Refusal



FORM DS-MT-186 (FORMERLY DS-ML-184)
65-100

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 66-F-65
W.P. 212-63
DATUM Geodetic

LOCATION QEW - 7th St. Sta. 29+92 O/S 49' Rt.
BORING DATE July 5, 1966
BOREHOLE TYPE Penndrill

ORIGINATED BY Y.K.
COMPILED BY Y.K.
CHECKED BY K.D.

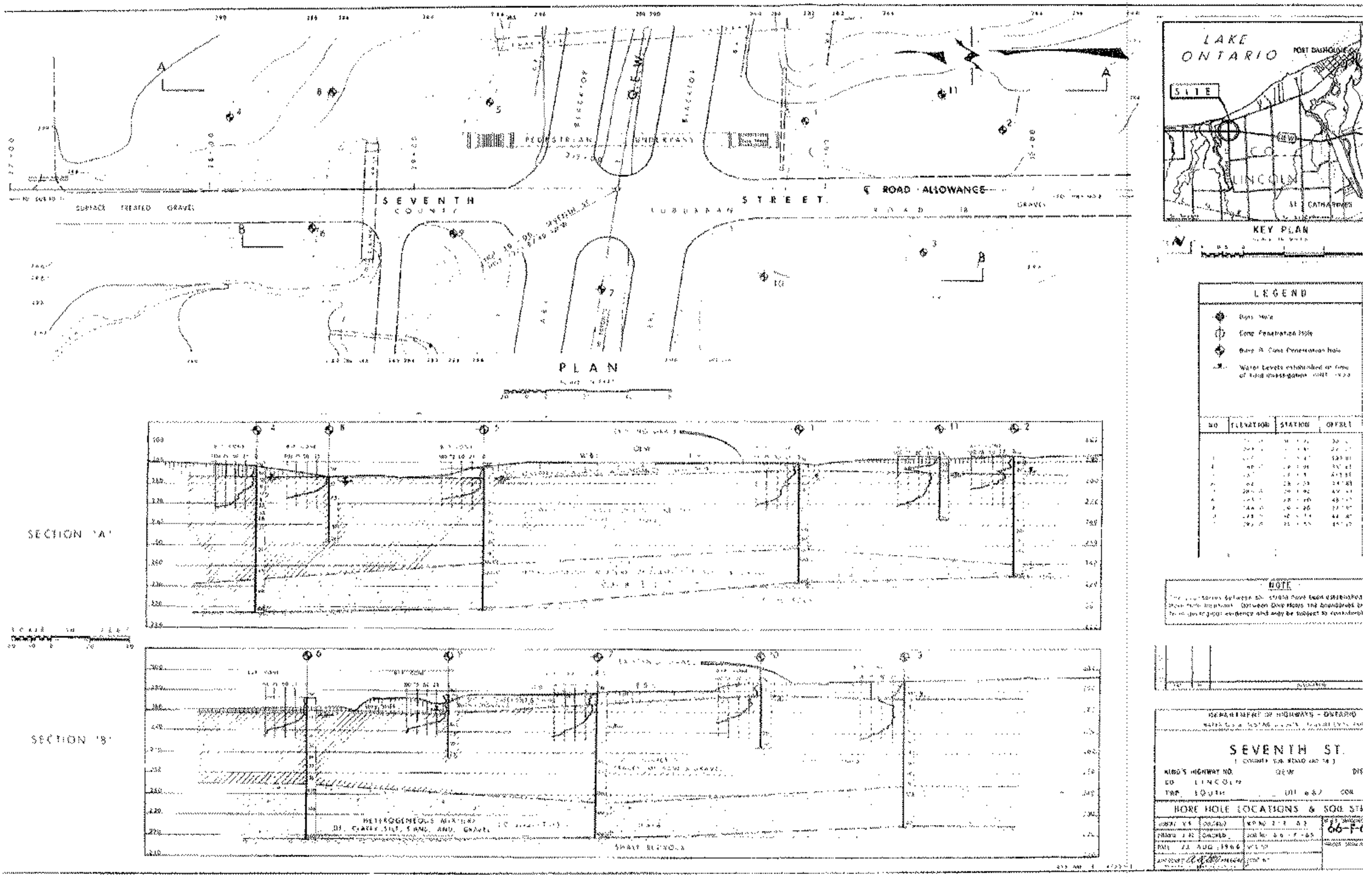
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ● Unconfined Compr. ○ Quick Triaxial + Field Vane 1000 2000 3000 4000 5000
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w
WATER CONTENT % 10 20 30
BULK DENSITY γ P.C.F.

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT
88.2m 289.0	Groundlevel	
86.6m 284.0 / 5.0	Silt to clayey silt & sand, with organics - firm.	
	Clayey silt, traces of sand and gravel. (Stiff to hard)	
71.9m 236.0 / 53.0	Heterogeneous mixture of clayey silt, sand and gravel - hard (Glacial Till)	
66.8m 219.0 / 70.0	Weathered Shale	
	End of borehole.	

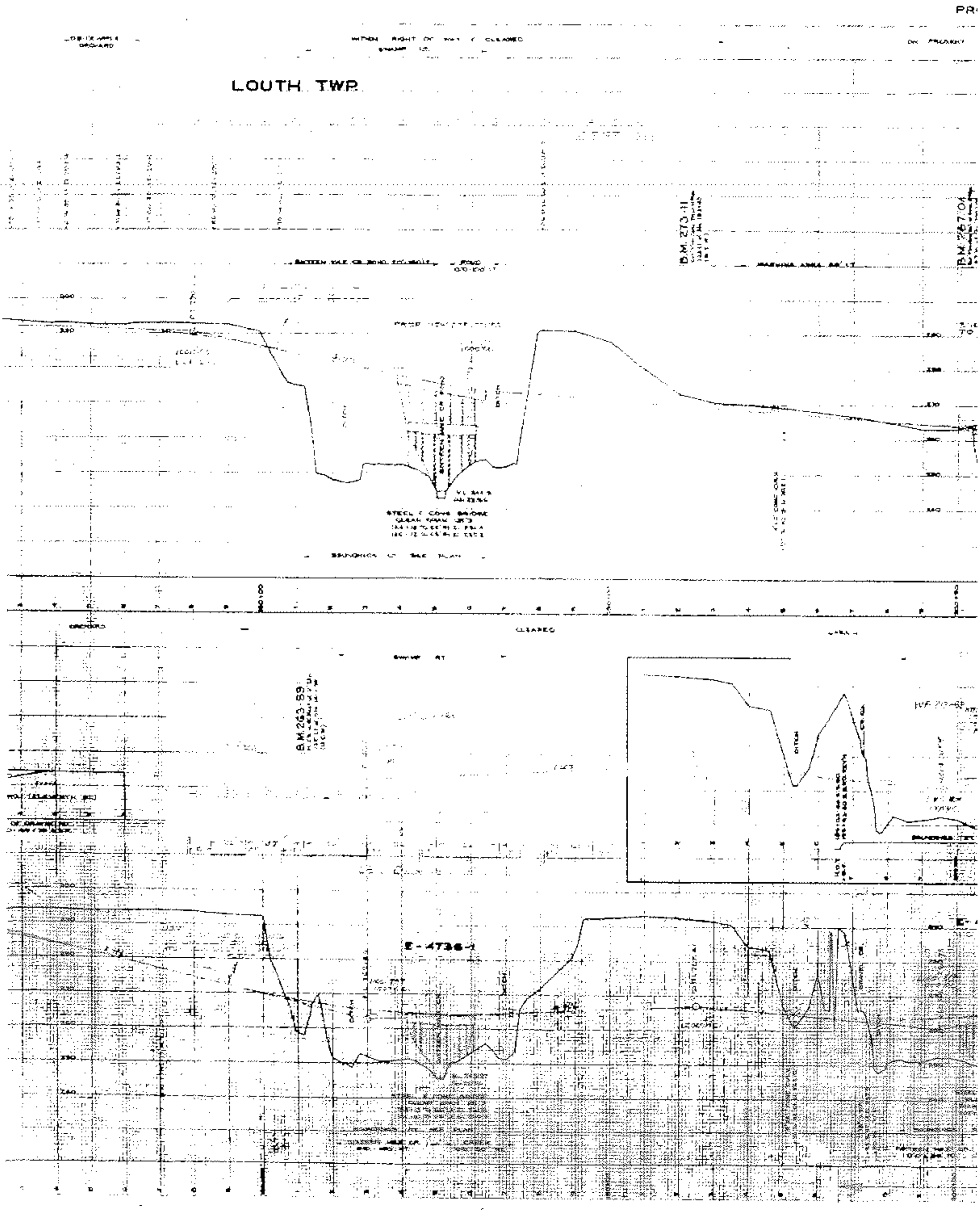
NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
1	SS	4		
2	TW	P	122 / 123	Si 46% Cl 54%
3	TW	P		Gr2% Sa10% Si55% Cl 33%
4	TW	P	134.5 / 134	W.L. El. 272.5
5	TW	P		
6	SS	13		
7	TW	P	137 / 136	
8	SS	30		
9	SS	21		Sa 4% Si 66% Cl 30%
10	SS	168		
11	SS	100 for 3"		Gr27% Sa53% Si&Cl 20%
12	SS	100 for 1"		Refusal

ELEV. SCALE: 280, 270, 260, 250, 240, 230, 220, 210

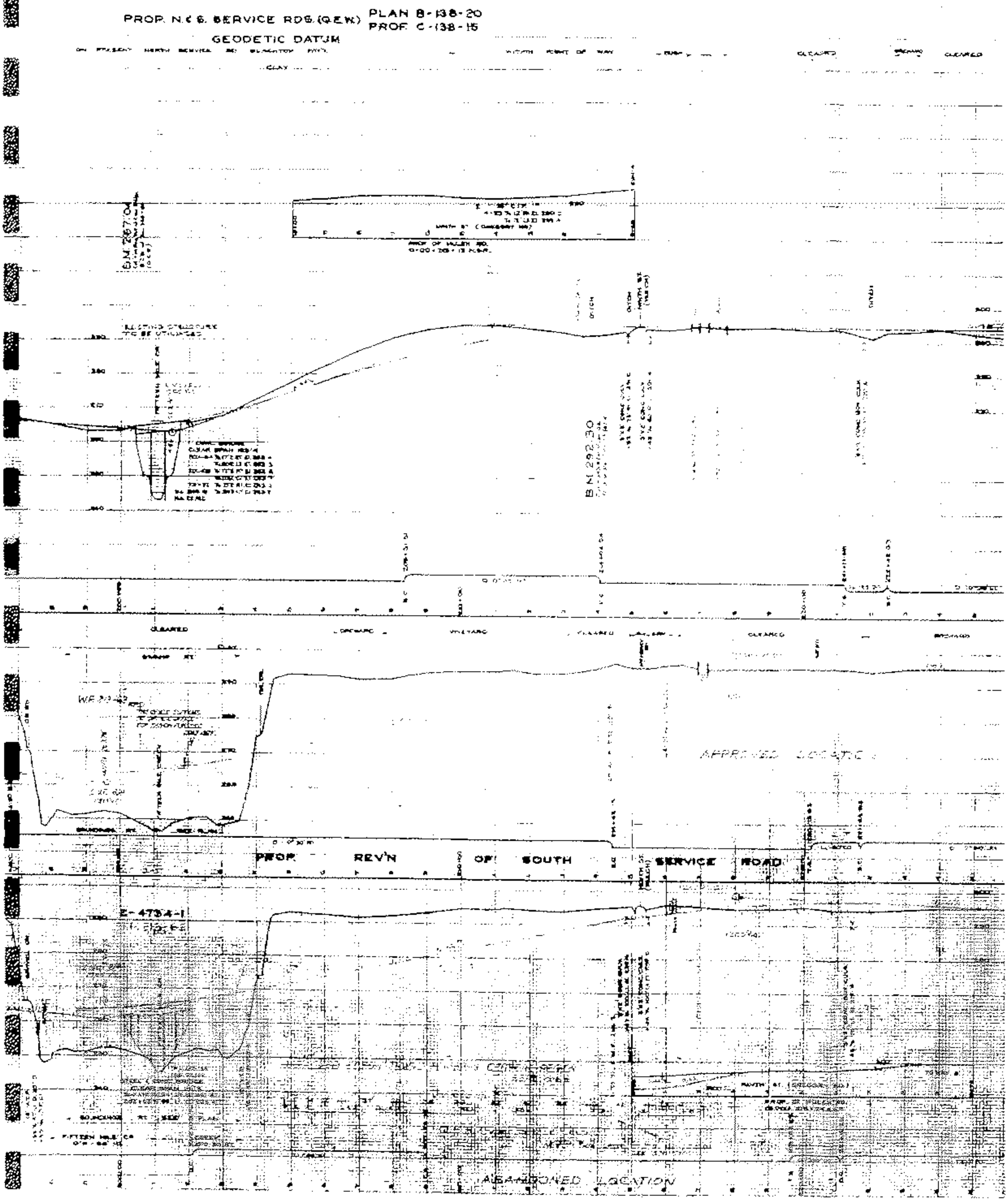
#66-F-65
W.P. #212-63
Q.E.W. &
SEVENTH ST.
UNDERPASS



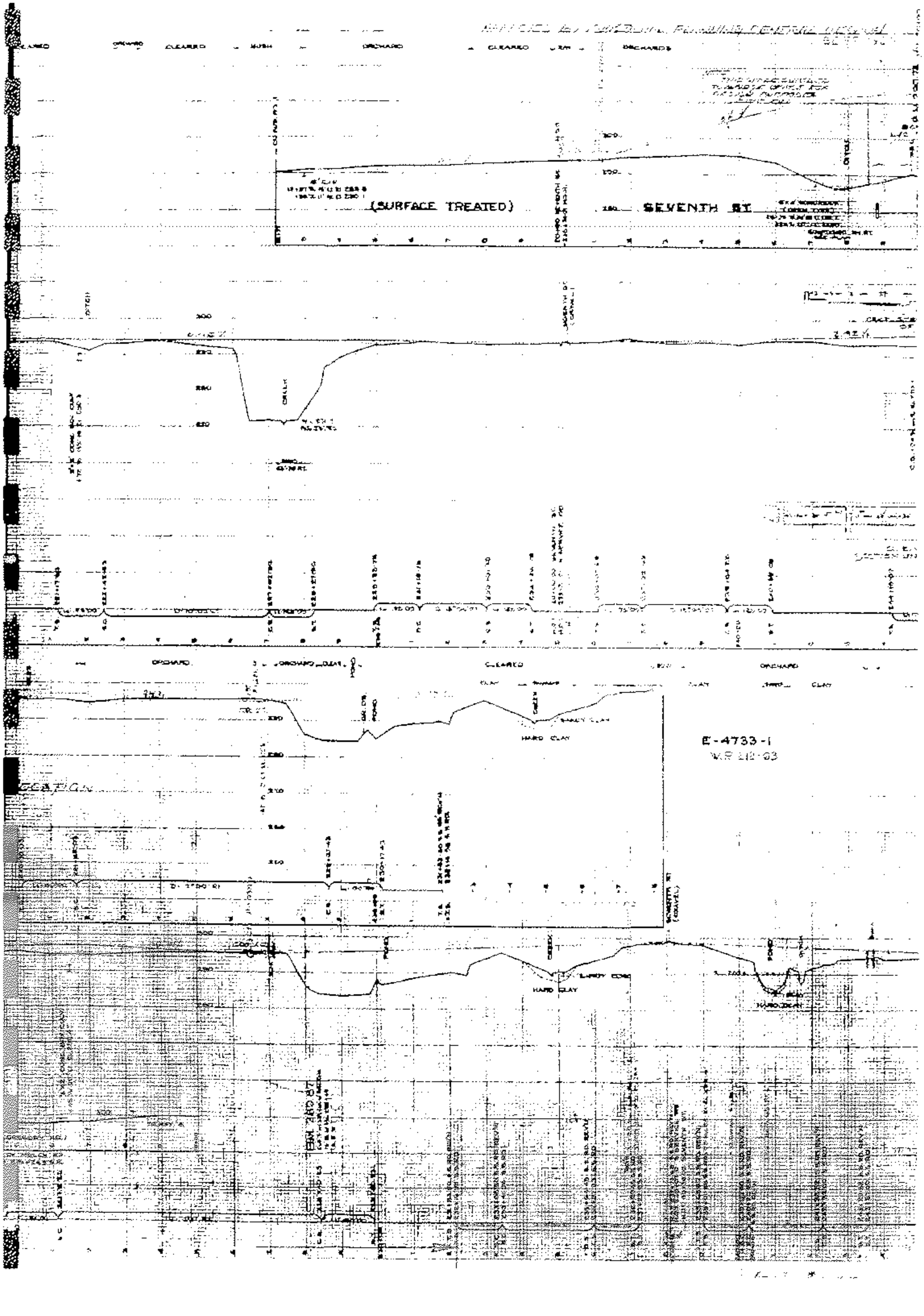
LAKE ONTARIO
PORT DALHOUSIE
SITE
LINCOLN
ST. CATHARINES
KEY PLAN
LEGEND
Bore Hole
Cone Penetration Hole
Bore & Cone Penetration Hole
NO.
ELEVATION
STATION
OFFSET
NOTE
SEVENTH
COUNTY
STREET
ROAD ALLOWANCE
SURFACE TREATED GRAVEL
PEDESTRIAN UNDERPASS
Q.E.W.
PLAN
SECTION 'A'
SECTION 'B'
SCALE
HETEROGENEOUS MIXTURE OF CLAYEY SILT, SAND, AND GRAVEL
SHALE BEDROCK
DEPARTMENT OF HIGHWAYS - ONTARIO
SEVENTH ST.
KING'S HIGHWAY NO.
Q.E.W.
CO. LINCOLN
TWP. LOUTH
LOT 6 & 7
BORE HOLE LOCATIONS & SOIL STRATA
AUG. 1966
66-F-



PR
LOUTH TWP.
B.M. 273·11
B.M. 267·04
B.M. 263·89
E-4736-1



SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS



CLEARED
ORCHARD
BUSH
(SURFACE TREATED)
SEVENTH ST.
DITCH
CREEK
ORCHARD
CLEARED
CLAY
HARD CLAY
SANDY CLAY
POND
E-4733-1