



REPORT

Foundation Investigation and Design Post Road Underpass Replacement Site No. 31-179 Highway 401, 5 km West of Highway 138 *W.P. 257-00-01*

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PART A

Foundation Investigation Report
Post Road Underpass Replacement
Site 31-179
Highway 401, 5 km West of Highway 138
Township of Cornwall
W.P. 257-00-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Limited (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the Design-Build of bridge and culvert replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project. Foundation investigation and design input to several bridge replacements was added to the overall scope of work following award of the project. This report presents the results of the detailed foundation investigation conducted for the replacement of the Post Road (County Road 36) underpass, Site No. 31-179 located on Highway 401 about 5 km west of the intersection of Highway 138 in Cornwall, Ontario.

The purpose of the foundation investigation was to assess the subsurface conditions for the proposed bridge replacement and proposed retained soil system (RSS) walls by drilling a total of nine boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2012 and the work was carried out in accordance with Golder's change proposals to WSP, dated July 10, 2014 and November 2, 2016.

2.0 SITE DESCRIPTION

The Post Road underpass is located on Highway 401 at Post Road, approximately 5 km west of the intersection with Highway 138 in Cornwall, Ontario. Post Road is part of the County Road network and is also known as County Road 36.

The existing bridge consists of a four-span continuous concrete T-beam structure overlain with an asphalt wearing surface. The superstructure is founded on spread footings at the abutments and piers. The existing structure is aligned approximately northeast-southwest, and is about 82 m long and 10.5 m wide. The bridge alignment is skewed at about 25 degrees from perpendicular to Highway 401. It is understood that the structure was built in 1961, previously rehabilitated in 1979, and had minor girder repairs completed in 2017.

Highway 401 in this area is a four-lane, divided highway. Post Road is a two-lane county road. In the area of the bridge, Post Road has been constructed on embankments that are on the order of about 7 m in height above Highway 401, with the Post Road pavement surface at about Elevation 82.9 m in the centre of the bridge. Based on the original design drawings, it is understood that Highway 401 is in a cut section with the original natural ground surface at Elevations between about 78 and 80 m. The Post Road embankment side slopes are oriented between about 2 horizontal to 1 vertical and 3 horizontal to 1 vertical (i.e., 2H:1V and 3H:1V). Based on visual observation at the time of the site investigations, the existing embankment slopes appear to be performing satisfactorily.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed bridge replacement was carried out in two stages. First, three boreholes (numbered 14-721, 14-722, and 14-723) were put down between November 3 and 10, 2014. Two additional boreholes (numbered 16-724 and 16-725) were subsequently put down between December 20 and January 6, 2017. Four additional boreholes (numbered 17-726 to 17-729, inclusive) were put down between November 7 and 9, 2017. The nine boreholes were advanced at the locations shown on Drawing 1. At Borehole 14-721, a temporary sand and gravel pad was constructed to provide suitable access to the borehole location. Following completion of the work, the temporary pad was removed, and the existing conditions were reinstated.

Boreholes 14-721, 14-722, 14-723, 16-724, and 16-725 were advanced with 200 mm diameter continuous-flight hollow-stem augers and/or rotary drilling during rock coring with a truck-mounted drill rig, supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. Boreholes 17-726 to 17-729 were advanced with portable drilling equipment, supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths ranging from about 4.0 to 24.7 m below the existing ground surface. Following penetration of the overburden soil, boreholes 14-721, 14-722, 14-723, and 16-724 were cored between 3.0 and 5.3 m into the bedrock.

Soil samples in the boreholes were obtained at regular intervals using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

Installation of a standpipe piezometer was attempted in Boreholes 14-723 to monitor the groundwater level at the site. The standpipe consisted of a 32 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed with bentonite pellet backfill. However, artesian conditions encountered during drilling persisted and the standpipe piezometer was removed after completion of the investigations.

Monitoring wells were installed in Boreholes 17-727 and 17-728 to monitor the groundwater levels at the site. The monitoring wells consisted of a 32 mm diameter rigid PVC pipe with a 3.0 m long slotted screen section, installed within silica sand backfill and sealed with bentonite pellet backfill. The groundwater levels were measured on November 15, 2017.

A 60 mm inside diameter rigid PVC casing was grouted for the full advancement depth (i.e., through the overburden and into the bedrock) at Borehole 16-724 to allow for Vertical Seismic Profile (VSP) testing to be carried out.

The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock, except as indicated previously for the standpipe piezometer and VSP casing. The site conditions were substantially restored following completion of work.

The field work was supervised by members of Golder's technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock samples.

The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory in Ottawa for further examination by the project engineer. Index and classification tests consisting of grain size distribution and water content testing were carried out on selected soil samples, and unconfined compressive strength tests were carried out on selected rock core samples obtained during the investigation. All of the laboratory tests were carried out to MTO LS and/or ASTM standards as appropriate.

Following the borehole investigation, VSP testing was carried out at Borehole 16-724 on January 13, 2017, by personnel from the Golder Associates' Mississauga and Ottawa offices. Details of the methodology and results of the testing are included in Appendix C.

The coordinates and existing ground surface elevations at the borehole locations were determined by Golder personnel using a Trimble R8 GPS survey unit. The geodetic reference system used for the survey is the North American datum of 1983 (NAD83). The borehole coordinates, based on the Modified Transverse Mercator (MTM Zone 8) coordinate system, and elevations are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)
14-721	South Abutment	4992749.1	200046.3	75.9 ⁽³⁾
14-722	North Abutment	4992782.8	200077.9	75.8
14-723	Central Pier (Within the median of Highway 401)	4992766.0	200060.7	76.0
16-724	Existing South Approach Embankment	4992729.3	200018.3	82.2
16-725	Existing North Approach Embankment	4992809.9	200088.5	82.2
17-726	Northwest Proposed RSS Wall	4992801.9	200067.9	79.7
17-727	Northeast Proposed RSS Wall	4992792.9	200094.0	78.7
17-728	Southwest Proposed RSS Wall	4992745.2	200012.9	80.3
17-729	Southeast Proposed RSS Wall	4992734.3	200039.7	78.9

Notes: 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 8) coordinate system.

2) Ground surface elevations shown are relative to Geodetic Datum.

3) The existing ground surface at borehole 14-721 was surveyed prior to construction of the temporary access pad.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the physiographic region known as the Lancaster flats, just east of the Glengarry till plain, as delineated in The Physiography of Southern Ontario.¹

The Lancaster flats region is characterized by poorly drained clay to fine grained sand deposits over glacial till plains.¹

4.2 Site Stratigraphy

The detailed subsurface soil conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B6 contained in Appendix B.

An interpreted stratigraphic section projected along the centreline of the proposed bridge alignment is shown on Drawing 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the location of the proposed bridge replacement consist of grade and embankment fill overlying glacial till and limestone bedrock.

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

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

4.2.1 Grade and Embankment Fill

To provide subsurface information at the proposed abutment locations during the initial 2014 field investigation, Boreholes 14-721, 14-722, and 14-723 were put down at the Highway 401 level, adjacent to the eastbound and westbound shoulders, respectively.

During the subsequent 2016/2017 field investigation, Boreholes 16-724 and 16-725 were advanced through the approach embankments from the travelled lanes of Post Road.

During the 2017 field investigation, Boreholes 17-726 to 17-729 were advanced through the approach embankments, between the abutment walls and the shoulder of Highway 401.

Grade fill was encountered at the existing ground surface at the boreholes put down at the Highway 401 grade (Boreholes 14-721, 14-722, and 14-723). The grade fill generally consists of silty sand to sand and gravel extending to depths ranging from about 0.5 to 1.7 m below ground surface (Elevations 74.3 to 75.3 m).

A Standard Penetration Test (SPT) "N" value measured in the fill encountered at Borehole 14-723, in the Highway 401 median of 17 blows for 0.3 m of penetration indicates a compact state of packing. The measured natural water content of one sample of the fill was about 5 percent. The results of grain size distribution testing carried out on the sample of the grade fill encountered in the Highway 401 median (Borehole 14-723) are provided on Figure B1 in Appendix B.

The pavement structure of Post Road at boreholes 16-724 and 16-725 consists of about 0.1 m of asphaltic concrete over 0.5 and 0.2 m of gravelly sand to gravel and sand base layer, respectively.

Embankment fill was encountered at the ground surface at Boreholes 17-726 to 17-729 and below the pavement structure in Boreholes 16-724 and 16-725. The embankment fill generally consists of sand or sandy silt to silty sand with varying amounts of gravel, cobbles and boulders. At Borehole 17-727 a layer of organic silt fill was encountered within the embankment fill at a depth of about 1.7 m. At the borehole locations, the embankment fill was inferred to extend to depths ranging from about 3.1 to 6.3 m below the existing ground surface (Elevations ranging from 75.0 to 76.7 m).

The SPT "N" values measured in the embankment fill at Boreholes 16-724 and 16-725 ranged from 6 to greater than 50 blows per 0.3 m of penetration, indicating a loose to very dense state of packing.

The results of grain size distribution testing carried out on nine samples of the embankment fill (Boreholes 16-724, 16-725, and 17-726 to 17-729) are provided on Figure B2 in Appendix B.

The measured natural water contents of 10 selected samples of the embankment fill ranged from about 7 to 10 percent.

The measured organic content of one sample of the organic silt embankment fill was about 7 percent.

4.2.2 Glacial Till

Glacial till was encountered beneath the fill in all boreholes. In general, the glacial till is a heterogeneous mixture of gravel and cobbles in a matrix of sand and silt. The surface of the deposit was encountered at Elevations ranging from 74.3 to 76.7 m. The deposit was fully penetrated in all boreholes, except Borehole 16-725 and 17-726 to 17-729. At the borehole locations, the glacial till had a thickness between about 11.2 and 13.3 m.

The SPT “N” values measured in the upper portion of the glacial till deposit (above about Elevation 68.2 to 69.9 m) range from 21 to greater than 50 blows per 0.3 m of penetration, indicating a compact to very dense state of packing. Within the lower portion of the glacial till deposit (below about Elevation 68.2 to 69.9 m), blow counts greater than 50 blows per 0.3 m of penetration or complete refusal of advancement of the split-spoon sampler were encountered. At borehole 14-722, rotary diamond drilling techniques were required to advance through the glacial till deposit between about Elevation 65.0 and 64.1 m. At borehole 14-723, rotary diamond drilling techniques were required to advance through the glacial till deposit between about Elevation 66.3 and 65.7 m.

The results of grain size distribution testing carried out on 12 samples of the upper, compact to very dense glacial till, and five samples of the lower, very dense glacial till are provided on Figures B3 and B4, respectively, in Appendix B. These test results do not reflect the cobble/boulder or full gravel content of the material, since the samples were retrieved using a 50 mm outside diameter split-spoon sampler. The measured natural water contents of 20 selected samples of the till ranged from about 5 to 16 percent.

A layer of sand containing some gravel, and trace silt was encountered within the glacial till in Borehole 16-724 at a depth of about 12.2 m and has a thickness of about 0.4 m. The results of grain size distribution testing carried out on one sample of the sand layer are provided on Figure B5 in Appendix B.

4.2.3 Bedrock

At all borehole locations that penetrated the glacial till, bedrock was encountered. The bedrock was cored for lengths between 3.0 to 5.3 m. Photographs of the recovered bedrock core are presented in Appendix D. The following table summarizes the bedrock surface depths and elevations as encountered at the four borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
14-721	75.9	12.4	63.5
14-722	75.8	12.7	63.1
14-723	76.0	12.9	63.2
16-724	82.2	19.4	62.9

The bedrock encountered in the boreholes typically consists of grey limestone with thin black shale interbeds and partings and is thinly to medium bedded. The bedrock is slightly weathered to fresh and typically strong to very strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples range from 38 to 100 percent but are typically greater than 75 percent, indicating a good to excellent quality rock.

The discontinuities observed in the rock core are associated with the joints and bedding of the bedrock.

Laboratory unconfined compressive strength testing was carried out on selected specimens of the bedrock core. The results of the testing are summarized on Figure B6 in Appendix B. The results of the unconfined compressive strength testing on three samples of the bedrock gave values ranging from 33 to 167 MPa.

4.2.4 Groundwater Conditions

Based on the field condition of the soil samples recovered from the boreholes, the water table at the time of the investigation was estimated to be between about 4.6 and 6.1 m depth below the existing ground surface (Elevations 71.4 m and 69.7 m). During the 2014 field investigation, a standpipe piezometer was installed within the Highway 401 median at Borehole 14-723 with the tip at 5.2 m depth (Elevation 70.8 m). At that time, the stable groundwater level was about 0.9 m above the existing ground surface. Similar artesian groundwater conditions were encountered during drilling at the other two boreholes put down during the 2014 investigation (Boreholes 14-721 and 14-722). The depth at which artesian conditions were encountered ranged between about 5 and 7 m below the existing ground surface.

At borehole 14-721, the flow of groundwater at the ground surface in the open borehole was estimated to be in the order of about 2 to 5 L/min.

Artesian groundwater conditions were also encountered during the original geotechnical investigation for the bridge carried out in May 1960 by others (GEOCRETS Report #31G-123, prepared by Associated Geotechnical Services Ltd. titled, "*Foundation Investigation Report, Proposed Structure: Hwy. No 401 and Road to St. Andrews, Lots 21 & 22, Con. 5, Twp. of Cornwall, WP 78-59, District No. 9*" dated June 1960). At that time, artesian conditions were encountered at depths that generally decreased to the east from about 7 m (Elevation 68.9 m) near the Post Road alignment, to about 12 m (Elevation 61.0 m) at the bedrock surface in the lowland basin to the east.

During the 2017 field investigation program, monitoring wells were installed in Boreholes 17-727 and 17-728 to monitor the groundwater levels at the site. The monitoring wells consisted of a 32 mm diameter rigid PVC pipe with a 3.0 m long slotted screen section, installed within silica sand backfill and sealed with bentonite pellet backfill. The groundwater levels were measured on November 15, 2017 and summarized in the table below.

Borehole Number	Borehole Location	Screened Interval	Groundwater Level Depth (m)	Groundwater Elevation (m)	Date of Reading
17-727	Northeast RSS Wall	3.5 – 6.6	2.4	76.4	November 15, 2017
17-728	Southwest RSS Wall	4.0 – 7.1	2.5	77.8	November 15, 2017

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Alex Meacoe, P.Eng. and Mr. Matt Kennedy, P.Eng., and was reviewed by Mr. Bill Cavers, P.Eng. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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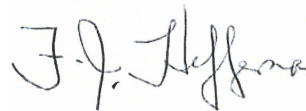
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PART B

Foundation Design Report
Post Road Underpass Replacement
Site 31-179
Highway 401, 5 km West of Highway 138
Township of Cornwall
W.P. 257-00-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Post Road underpass on Highway 401. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations at the site. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the replacement structure. It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC). In accordance with Section 4.4.2 of the CHBDC, the proposed bridge structure is understood to have an importance category of *other* bridge.

The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the detail design of the project, and for which special provisions may be required in the contract documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge consists of a two-lane, four-span, continuous concrete T-beam structure that was originally constructed in 1961. The two middle spans are about 22.1 m long, and the two outer spans are about 18.9 m long. The bridge alignment is skewed at about 25 degrees from perpendicular to the Highway 401 alignment. It is understood that the preferred alternative for the proposed replacement consists of a shorter two-span structure, with span lengths of about 38 m, on the same alignment as the existing bridge with no significant change in width. The new underpass structure will be founded on abutments located inside the existing abutment foundations with one central pier. The proposed Post Road pavement grades will be up to about 1.8 m higher than the existing pavement grades.

6.2 Seismic Design

6.2.1 Seismic Zone and Importance Category

The site falls within the Western Quebec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, the proposed bridge structure has been given an importance category of other bridge.

6.2.2 Seismic Site Classification

Vertical Seismic Profile (VSP) geophysical testing was carried out at the existing bridge location to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The shear wave velocities measured are presented in a technical memorandum (see results in Appendix C) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy, measured from the existing surface of the southern embankment (Borehole 16-724) is 741 m/s. Based on the results of the testing, the average shear wave velocity calculated in a deeper 30 m interval (i.e. below spread footings or pile caps constructed at depth) is expected to be larger. As such, the corresponding Seismic Site Class is a Site Class C.

6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridge (latitude 45.07 and longitude -74.83), the following are the reference Site Class C (reference) peak seismic hazard values (based on the 5th generation seismic hazard maps published by the GSC) that can be used for the design of the proposed bridge structure.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.14	0.22	0.38
PGV (m/s)	0.08	0.14	0.25
Sa (0.2) (g)	0.21	0.34	0.59
Sa (0.5) (g)	0.11	0.18	0.31
Sa (1.0) (g)	0.05	0.08	0.15
Sa (2.0) (g)	0.02	0.04	0.07
Sa (5.0) (g)	0.01	0.01	0.02
Sa (10.0) (g)	0.00	0.00	0.01

The fundamental period of the structure is expected to be less than 0.5 s which, in consideration of its *other* importance category and the site-specific seismic hazard values given above, would indicate that the bridge structure falls in Seismic Performance Category 3 in accordance with Table 4.10 of the CHBDC. Based on this Seismic Performance Category and the *irregular* geometry of the bridge, it is understood that the structure will be designed using a “performance-based approach” as defined in the CHBDC. However, if the fundamental period of the structure is greater than 0.5 s, the bridge structure would fall in Seismic Performance Category 2, and the structure would be designed using a “force-based approach” as defined in the CHBDC.

6.2.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength

can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with the “simplified” approach outlined in the CHBDC and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using the data collected at the boreholes, and the results of the VSP testing. The design groundwater level was taken to be at the ground surface for each borehole in the model. The CRR with depth was calculated at each borehole location using the parameter, $(N_1)_{60cs}$, that is based on the SPT “N” blow counts obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment indicate that the site soils have a low potential for liquefaction during 2,475-year design earthquake and may be considered to be non-liquefiable for design.

6.2.5 Seismic Loading on Abutment Walls

Section C4.6.4 of the Commentary to the CHBDC provides guidance on appropriate methods to represent the backfill passive pressure force resisting movement at the abutments during seismic loading. For vertical bridge abutment walls up to about 1.7 m high, a spring stiffness based on the near-field conditions immediately behind the abutment wall is recommended (Caltrans, 2013). For abutment walls that are taller than 1.7 m, consideration of the influence of the far field conditions on the backfill stiffness is recommended (Carvajal, 2011). It is understood that the proposed abutment walls at the Post Road underpass site will be about 5 m high (from underside of approach slab to the underside of the abutment “pile cap”).

To provide an estimate of the seismic loads on the integral abutment walls under the 2,475-year design ground motions, dynamic soil soil-structure interaction (SSI) analyses of the bridge structure, abutment backfill, and embankments were undertaken. The two-dimensional SSI analyses were carried out considering the earthquake records, which provide the mean and maximum response with respect to the cyclic shear stresses under the design ground motions. The earthquake records for the mean and maximum response were obtained from the results of site-specific one-dimensional ground response analyses.

The results of the SSI analyses were compared to those calculated using the simplified analysis methodology proposed by Carvajal that considers the near field and far field response on the earth pressures due to shaking, as well as the results using the Caltrans methodology.

More detailed discussion of the analysis methodology and results were provided to WSP under separate cover in the memorandum titled, “*Soil-Structure Interaction Analyses, Post Road Underpass (Site 31-179), Highway 401, Ontario*”.

The total force vs. displacement computed using the Carvajal approach agreed well with the results of the dynamic SSI analyses. Therefore, it is recommended that the combined near and far-field response of the soils calculated using the Carvajal approach be considered to represent the soil passive reaction at the abutment. The maximum passive pressure that can be mobilized with the lateral movements of the abutment was also estimated using the Richards & Elms (1979) approach based on the Mononobe-Okabe (M-O) formulation. The soil reaction at the abutment should be limited to this maximum passive force.

Based on the Carvajal approach, the combined near and far-field response of the soils may be approximated using a linear elastic spring with a stiffness of 1,520 kN/mm for displacements up to about 16 mm. The soil spring should be applied at mid-height, perpendicular to the abutment wall.

6.3 Bridge Foundations – General

6.3.1 Existing Foundations

The existing Post Road underpass is a four-span structure with non-integral abutments. The existing bridge is understood to be in poor condition. The deterioration of the substructure of the bridge includes severe corrosion of the rocker bearings, and delaminations and spalling of the concrete at the abutments and piers. The rotation of the rocker bearings is understood to be greater than expected and not consistent with thermal movements of the bridge deck.

Based on the 1960 design drawings (Drawings TWP #31-179-1-A to TWP #31-179-5-A, included in Appendix F), the existing abutment and pier foundations are understood to consist of spread footings skewed from the bridge alignment to roughly match the Highway 401 alignment. In plan, the abutment footings are rhomboids with sides angled to match the bridge alignment. The north abutment footprint is about 10.8 m long and 3.5 m wide. The south abutment is about 10.8 m long and 2.7 m wide. The three pier footings are rectangular and about 9.8 m long and 2.7 m wide.

The original design drawings indicate that the pier footings were to be founded within the glacial till at elevations ranging from about 73.0 m to 73.9 m. The north and south abutment footings were to be founded within the approach embankments at Elevation 76.0 m and 77.7 m, respectively. The ground surface profile that existed prior to the original bridge construction shown on Drawing TWP #31-179-1-A is relatively flat and ranges from about Elevation 78.2 m at the north abutment to about Elevation 80.0 m at the south abutment. Based on the results of the original geotechnical investigation for the bridge carried out in May 1960 (GEOCRE Report #31G-123), it is considered likely that the abutment footings are founded on native glacial till that comprises the lower portion of the approach embankments.

6.3.2 Foundation Options

Based on the subsurface conditions, both shallow and deep foundation options have been considered for the replacement of the existing Post Road 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.

- **Spread footings founded on glacial till:** Spread footings founded on or within the compact to very dense native till are feasible for support of the replacement structure. This foundation type would not permit the use of integral abutments but may still be used at the pier. Some minimal settlement (less than about 25 mm) of the abutment and/or pier footings may occur for footings founded on the glacial till. Artesian groundwater conditions were encountered at depths as shallow as about 5 m below the Highway 401 grade at the

foundation locations. Excavations to significant depth at the site would likely require active dewatering to control groundwater flows during construction. Excavation for spread footings should be maintained at a suitably shallow depth to avoid penetration into overburden with artesian groundwater pressures.

- **Driven steel H-piles:** Steel H-piles driven through the glacial till to refusal on the limestone bedrock could also be considered for support of the replacement bridge structure. This option would allow the pile caps to be maintained at a higher elevation than for spread footings at the abutments, thus minimizing excavation depth, protection system requirements and groundwater control requirements, while achieving relatively higher geotechnical resistances and minimizing settlement. Steel H-pile foundations would also allow for the construction of integral abutments. If the piles are driven, the use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which contains cobbles and boulders) and seating onto the limestone bedrock. It is anticipated that groundwater may flow up the H-piles to the ground surface during driving. In this case, a fill blanket consisting of Granular A may be placed at ground surface to act as a filter to minimize the potential for loss of fines from the subsurface strata.
- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments, and this foundation option would have similar advantages to steel H-piles in terms of minimizing excavation depth, protection system requirements and groundwater control requirements. However, pipe piles are considered to have a higher risk than H-piles for “hanging up”, damage, or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the till deposit. Open-ended pipe piles would provide a greater challenge to management of artesian groundwater flows during driving.
- **Drilled concrete caissons:** Caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from the water-bearing cohesionless till soils during construction. In addition, the caissons must be socketed into the bedrock a sufficient length to provide the required bearing resistance. The presence of cobbles and boulders may require churn drilling and possibly rock coring techniques to penetrate obstructions where encountered in the glacial till. The caisson sockets would have to be advanced by rock coring and/or chisel drilling into the strong to very strong limestone bedrock.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments and pier for the underpass replacement on spread footings founded on the compact to very dense glacial till. It is anticipated that the spread footings could be maintained at an elevation high enough to avoid any significant groundwater control and protection system implications associated with artesian groundwater conditions.

6.3.3 Feasibility of Integral Abutments

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

The shallow spread footing foundations described in Section 6.4 would not meet MTO's foundation criteria for flexible foundation elements. The flexible pile-supported abutment foundations discussed in Section 6.5 would substantially meet the requirements for integral abutments, but the MTO criteria also discuss the requirement for rigorous analysis considering the loading effects associated with a skew angle greater than 20 degrees.

6.3.4 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed bridge structure and foundation system may be classified as having medium traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in the sections below.

6.4 Shallow Foundations

6.4.1 Founding Elevations

If adopted for the replacement structure, spread footings should be founded on the compact to very dense glacial till and below any existing fill or soil containing organic matter. Shallow foundations bearing directly on the underlying limestone bedrock are not considered to be practical due to the significant depth of excavation and groundwater control issues at depth that would be required.

The following table provides the maximum (highest) founding elevations recommended for design of footings founded on the compact to very dense glacial till deposit. Excavation would be carried out to depths of up to about 7.0 m below the existing Post Road grade at the abutments and to about 1.7 m below the existing grade at the pier location. The groundwater level was measured in the well installed at the site at Elevation 76.9 m (at 0.9 m above the ground surface) in November 2015 and, therefore, dewatering of the excavations to the founding elevations presented below may be required depending on groundwater level encountered at the time of construction. A Non-Standard Special Provision has been provided in Appendix E to address this requirement.

Foundation Element	Borehole Number	Founding Stratum	Footing Founding Elevation (m)
North Abutment	14-722, 16-725	Compact to very dense till	Below 75.3*
Central Pier	14-723	Dense to very dense till	Below 74.3
South Abutment	14-721, 16-724	Dense to very dense till	Below 75.3*

Note: Compacted Granular A fill could be used to raise the foundation level in order to minimize the concrete requirements while still maintaining the required earth cover of 1.7 m for frost protection purposes.

Excavations for spread footings deeper than about Elevation 74.2 m should be avoided to reduce the potential for heave of the bottom of the excavation during construction due to the artesian groundwater pressure at this site. However, it is understood that the excavations at the central pier would be expected to extend below the elevation of the existing footing, which is at about Elevation 73.0 m. In this case, where excavations for footing construction are to extend deeper than about Elevation 74.2 m, pressure-relief wells drilled to below the artesian groundwater level may be used to manage the groundwater pressure and flow during construction.

To minimize the concrete requirements and/or reduce the duration of dewatering excavations below Elevation 74.2 m, the shallow footings could be founded on a pad of compacted Granular A fill to raise the foundation level. The granular pad should be placed directly on the prepared native glacial till, in 300 mm thick lifts compacted to 95 percent of the material's standard Proctor maximum dry density.

Spread footings should be founded deep enough to accommodate frost protection requirements. The footings should be provided with a minimum 1.7 m of soil cover for frost protection as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the footing.

The footing subgrade should be inspected in accordance with OPSS 902 (Construction Specification for Excavating and Backfilling – Structures) to check that all existing fill, organic deposits, and other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing, or a granular pad to support the footing, is not placed within four hours of the inspection and approval of the subgrade. A Non-Standard Special Provision has been provided in Appendix E to address this requirement.

6.4.2 Geotechnical Resistance

Spread footings placed on the properly prepared glacial till deposit, at or below the design elevations given in the preceding section, should be designed based on a factored geotechnical resistance of 450 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 300 kPa at Serviceability Limit States (SLS, for 25 mm of settlement). Similar factored geotechnical resistances may be used for bearing on a pad of engineered fill placed directly on the prepared native glacial till, in 300 mm thick lifts compacted to 95 percent of the material's standard Proctor maximum dry density.

The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and subsoils should be calculated in accordance with Section 6.10.5 of the CHBDC. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the glacial till or a pad of engineered fill, the coefficient of friction, $\tan \delta$ or $\tan \phi'$, may be taken as follows:

- Cast-in-place footing to concrete working slab: $\tan \delta = 0.6$
- Cast-in-place concrete working slab to glacial till: $\tan \phi' = 0.60$
- Cast-in-place footing to pad of engineered fill (i.e. Granular A): $\tan \phi' = 0.60$

The resistance to lateral loads could be increased by constructing a shear-key at the bottom of the footing. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments.

The above values assume that the subgrade materials will not be disturbed by construction activities or groundwater inflow.

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

6.5.1 Founding Elevations

Alternatively, the replacement structure may be supported on steel H-piles driven to found on the limestone bedrock or closed-ended steel pipe (tube) piles founded within the very dense glacial till. Based on the borehole results from the investigation, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, the following pile tip elevations are recommended for design of steel H-piles or pipe piles.

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
North Abutment	14-722	63.1	63.0
Central Pier	14-723	63.1	63.0
South Abutment	14-721, 16-724	62.9 to 63.5	62.8 to 63.4

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

If integral abutments are adopted, the upper portion of the piles would need to be cased in a sand-filled, corrugated steel pipe (or similar) to provide suitable flexibility of steel H-piles.

The borehole logs at the abutments indicate that there is a very dense layer within the glacial till containing cobbles and boulders below about Elevation 70 m. At the south extent of the site (Borehole 16-724), very dense glacial till was encountered at about Elevation 74.0 m. Diamond drilling was required to penetrate portions of this deeper, very dense glacial till in Boreholes 14-722 and 14-723. Steel H-piles reinforced at the tip with a driving shoe are expected to penetrate this layer and continue to the surface of the bedrock. However, if the H-piles reach effective refusal in the glacial till, a reduced axial geotechnical resistance may be used (see Section 6.5.2).

Due to the potential presence of cobbles and boulders within the till deposit, steel H-piles are preferred over closed-ended steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. Pipe piles will also be more susceptible to damage, and thicker walled pipe piles may be required and/or allowance made for pile damage. The piles should be reinforced at the tip with a driving shoe to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving in accordance with OPSS 903 (*Deep Foundations*). If steel pipe piles are used, driving shoes should be in accordance with OPSP 3001.100 Type II (*Steel Tube Pile Driving Shoe*).

Due to the potential of artesian groundwater conditions at Post Road, it is anticipated that groundwater may flow up the H-piles to the ground surface during driving, depending on the construction surface elevation at the time of pile driving. If the piles are driven from about Elevation 77.0 or below, a fill blanket consisting of Granular A should be placed at ground surface to act as a filter to reduce the potential for loss of fines from the subsurface strata. It is understood that sand-filled CSP casings are to be installed to about Elevation 75.8 m to allow sufficient flexibility for integral abutments. As described in Section 6.4.1, if excavation or augering for the CSP installation is carried out no deeper than about Elevation 74.2 m, artesian conditions and potential for basal heave would be avoided during CSP installation. Therefore, a fill blanket consisting of uncompacted Granular A fill with a minimum

thickness of 500 mm should be placed at the bottom of the CSP casings in the pre-augered CSP holes prior to driving of the piles. If the contractor opts to open-trench for installation of the CSPs, dewatering may be required. The volume of groundwater inflows into such an excavation may be high and an NSSP for dewatering is provided in Appendix E.

6.5.2 Axial Geotechnical Resistance

For design of HP 310x174 piles driven to the estimated tip elevations provided in Section 6.5.1, the factored axial geotechnical resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm. However, it is noted that based on the presence of cobbles and boulders within the till deposit, and the very dense nature of portions of the till deposit, some steel H-piles or pipe piles may not reach the bedrock. Provided that these piles meet practical refusal in the very dense glacial till at depth, the factored axial geotechnical resistance at ULS for such piles may be taken as 1,600 kN and the axial geotechnical resistance at SLS may be taken as 1,300 kN.

Based on the relatively dense and incompressible nature of the native glacial till, downdrag loads on the piles may be considered to be negligible for design.

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

6.5.3 Lateral Geotechnical Resistance

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the *Commentary* to the CHBDC.

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

The foundation lateral soil-structure interaction springs required for the dynamic analysis of the bridge pier and abutments were computed based on the available subsurface information on the soil layers surrounding the foundations and the pile dimensions.

The soil-structure interaction between the bridge foundations and the surrounding soils was modeled using the load transfer method. The lateral load-displacement behaviour of the piles can be modeled using p-y curves (CFEM, 2006). P-y curves relate the lateral deflection of a single pile to the corresponding soil and bedrock reactions at any depth below ground surface. The p-y curves were generated internally using the commercially available software program LPILE Plus (Version 2016.09.009), produced by ENSOFT Inc.

For all loading conditions, a pinned connection was assumed between the pile head and the pile cap. Static and cyclical loading conditions were considered in the lateral analyses; therefore, p-y curves were generated for all foundations as described in the LPILE Plus (Version 5.0) Technical Manual (2004) which is based on the studies of Wang (1982) and Long (1984).

The family of static and cyclic p-y curves calculated at 0.5 to 1.0 m increments of depth for a single, vertical 310x174 steel H-pile and a single, vertical 360x132 steel H-pile at the abutments are presented in tabular format and graphically in Appendix G.

For piles arranged in closely spaced groups, the pile-soil-pile interaction causes the individual piles in a group to be less effective than a single pile. These “group effects” can be incorporated into the design using a method that modifies the single pile lateral resistance by some factor (i.e. a p-reduction factor). Generalized p-multipliers (i.e. p-reduction factors) for a range of pile spacings are provided in Section C6.11.3.4 of CHBDC.

6.6 Caisson Foundations

Support of the abutments or central pier may alternatively be provided by caisson foundations. Due to the relatively high water table, the artesian groundwater conditions, and the difficulty in socketing a liner into the strong to very strong bedrock, it may not be feasible to dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. The axial geotechnical resistance for rock socketed caissons should be based primarily on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

The use of a liner or casing will be required in order to advance the caissons through the overburden with minimal loss of ground. The casing should be extended so that it is “seated” a minimum of 300 mm into the bedrock.

Casing installation through the glacial till containing cobbles and boulders may be difficult. Churn drilling and possibly rock coring techniques will be required to advance the caissons through the glacial till. In addition, the bedrock at this site is strong to very strong, and the caisson sockets will likely have to be advanced by rock coring (possibly supplemented with a down-hole hammer) and/or chisel drilling.

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

6.6.1 Axial Geotechnical Resistance

Rock-socketed caissons should be designed based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at ULS of 1,000 kPa, provided that the caisson socket is within competent bedrock (i.e., RQD greater than 75 percent). This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

For a 1.5 m diameter caisson with a 1.2 m diameter socket embedded 3 m in to the competent bedrock, this would equate to a factored axial geotechnical resistance at ULS of about 11,300 kN. SLS resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.6.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soil in front of the caissons, and the reductions due to group effects, may be determined as outlined in Section 6.5.3.

The family of static and cyclic p-y curves calculated at 0.5 to 1.0 m increments of depth for a single vertical 1.5 m diameter concrete caisson at the pier are included in Appendix G.

6.7 Lateral Earth Pressures for Design

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

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.7 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary* to the CHBDC).

6.7.1 Static Lateral Earth Pressures for Design

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

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

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.00

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

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.70	3.70

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

6.7.2 Seismic Lateral Earth Pressures for Design

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

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Seismic Active Pressure Coefficients, K_{AE}

	Design Earthquake	Site PGA	SSM	Granular A	Granular B Type II
Yielding wall	2,475 Yr	0.38 g	0.46	0.39	0.39
Non-yielding wall	2,475 Yr	0.38 g	0.66	0.55	0.55

- The above K_{AE} values for yielding walls are applicable provided that the wall is capable of moving approximately 50 mm.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls}$$

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

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

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

γ is the unit weight of the backfill soil (kN/m³), as given previously;

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

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

6.8 Approach Embankments

It is understood that the overall grade of Post Road will be raised up to about 1.8 m to accommodate an increase in the soffit elevation of the bridge required for clearance above Highway 401. In general, the existing width and alignment of Post Road are to be maintained and, therefore, the existing embankments will require nominal widening to accommodate the proposed grade raise.

Based on the results from the boreholes drilled through the existing Post Road embankments, the road structure is generally underlain by embankment fill consisting of gravelly sand, overlying silty sand fill containing some gravel that is underlain by glacial till and limestone bedrock.

6.8.1 General Embankment Construction

It is recommended that all topsoil or existing loose surficial fill present within the widening footprint be stripped prior to placement of embankment fill.

The new embankment fill associated with the grade raise for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (*Earth Excavation and Grading*) and OPSS 501 (*Compacting*).

Benching of the existing Post Road embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened, in accordance with OPSS 208.010 (*Benching of Earth Slopes*).

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

6.8.2 Global Stability

A slope stability assessment of the embankments has been carried out considering the proposed grade raise of up to about 1.8 m using the commercially available slope stability analysis software package SlopeW™ by GeoSlope International Ltd., to verify that a minimum factor of safety of 1.3 is achieved under static conditions and 1.1 under design seismic conditions. These minimum factors of safety are considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

The stability analyses were carried out considering that embankment side slopes will be maintained at no steeper than 2H:1V. The soil stratigraphy used in the analyses was selected to represent soil conditions with the greatest thickness of overburden soil that may be expected at the site and was based on the information available.

Provided that the approach embankment side slopes are maintained no steeper than 2H:1V, and the existing embankment side slopes are benched in accordance with OPSS 208.010 (*Benching of Earth Slopes*), to “key in” any new fill materials placed on the slopes to accommodate the overall grade, the embankments should have an adequate minimum factor of safety of at least 1.3 under static conditions and 1.1 under design seismic conditions. If side-slopes steeper than 2H:1V are to be considered or the Post Road grade is to be increased more than 1.8 m above the existing grades, the embankment side-slope stability will have to be re-assessed.

A summary of the material parameters and stability analysis results are presented in Appendix H.

6.8.3 Settlement

The additional loading imposed by the proposed 1.8 m grade raise will result in an increase in effective stress within the glacial till that underlies the site. The elastic compression of the existing glacial till is estimated to be less than about 25 mm. As described above, any organic matter encountered within areas of the embankments that are to be widened to accommodate the grade raise should be removed prior to placement of any fill.

Additional settlement of the embankments will occur as a result of compression of the new grade fill and the existing embankment fill. The magnitude of compression of the new fill may range from 0.5 to 1 percent of its thickness, assuming approximately 95 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. Some nominal compression of the existing fill (less than 0.5 percent of its thickness) is expected to occur under the increased loading. Provided that granular fill is used to raise the grade, settlement of the new fill is expected to occur essentially during embankment construction. Similarly, settlement of the existing silty sand embankment fill will be elastic in nature and should occur essentially immediately following placement of the new fill.

6.9 Reinforced Soil System (RSS) Retaining Walls

Due to the geometry of the proposed abutments and approach embankments relative to the Highway 401 right-of-way, retaining walls will be required to extend the width of the retained soil behind and beyond the abutments.

The retaining walls are understood to be designed as Retained Soil System (RSS) walls that extend up to about 13 m in plan beyond the outer limits of the proposed abutments (i.e., perpendicular to Post Road). Based on the available design drawings, it is understood that the RSS retaining wall foundations are to be “perched”, i.e., having foundations within the existing approach embankments above the Highway 401 grade.

The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22.

6.9.1 Founding Elevations

A typical RSS retaining wall has a front facing supported on a strip footing (concrete levelling pad) placed at shallow depth below the ground surface in front of the wall; this footing, and the reinforced soil mass, should be founded below any topsoil, loose fill or unsuitable native soils.

In general, the fill encountered at the boreholes put down near the proposed RSS retaining wall footprints is considered to be suitable for support of the walls, with the exception of Borehole 17-727 where organic silt was encountered extending down to about Elevation 76.7 m. At this location, and where other localized organics or unsuitable fill is encountered, it is recommended that the unsuitable existing soils be subexcavated a further 0.5 m (i.e., the subexcavation should extend to Elevation 76.2 m at Borehole 17-727) and replaced with compacted Granular A or Granular B Type II.

Provided the subgrade is adequately prepared during construction, the performance of the walls are expected to meet the global performance tolerances specified for high performance and appearance.

Because the proposed RSS retaining walls are perched within the embankments, the recommended founding elevations are governed by the global stability of the RSS wall slopes. As described below, walls were found to be stable provided that they have a reinforcement length equal to or greater than 1.0 times the height of the wall (defined as being measured from the bottom of the concrete levelling pad to the top of the retained soil at the front face of the wall), they are supported on foundations with the underside no higher than Elevation 78.0 metres and they are founded with the underside of the concrete levelling pad at least 1.0 metre below the top of the fore slope, as outlined in Section 5.2 of the *MTO RSS Design Guidelines* manual.

The granular levelling pad for the RSS retaining wall foundations should consist of compacted OPSS.PROV 1010 (*Aggregates*) Granular ‘A’ material that should extend at least 1.0 m beyond the outside edge of the facing panels, then outward/downward at 1 horizontal to 1 vertical (1H:1V). The granular levelling pad should be a minimum 300 mm thickness below the base of the concrete levelling pad, as outlined in Section 5.3 of the *MTO RSS Design Guidelines* manual.

6.9.2 Geotechnical Resistance

For the RSS facing panels supported on a concrete footing constructed on a compacted granular pad, the wall design may be completed based on a factored geotechnical resistance at ULS of 150 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

6.9.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the RSS wall footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC. For cast-in-place concrete footings that are cast on top of the compact granular fill, the coefficient of friction, $\tan \delta$ or ϕ' , can be taken as follows:

- RSS wall compacted granular fill to existing or new fill: $\tan \phi' = 0.58$
- Cast-in-place concrete footing to compacted granular fill: $\tan \delta = 0.58$

6.9.4 Global Stability

Based on the available design drawings, the proposed RSS retaining walls range up to about 2.7 m high above the top of the fore slope, with the back slope (above/behind the RSS) and the fore slope (below/in front of the RSS wall) to be sloped at 2 horizontal to 1 vertical. The height of the RSS retaining walls are understood to decrease with distance away from the abutments to match the embankment side slopes.

The external (global) stability of the proposed RSS retaining walls has been assessed assuming a reinforcement length of 1.0 times the wall height using the commercially available slope stability analysis software package SlopeW™ by GeoSlope International Ltd. The walls were found to be stable based on achieving a minimum factor of safety of 1.3 is under static conditions and 1.1 under design seismic conditions. These minimum factors of safety are considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

The RSS retaining walls were found to be stable provided that they have a reinforcement length equal to or greater than 1.0 times the height of the wall (i.e., the supplier may require a greater length of reinforcement for internal stability but it should not be less than the height of the wall), they are founded no higher than Elevation 78.0 metres, and they are founded at least 1.0 metre below the top of the fore slope.

6.10 Construction Considerations

The following sections identify future construction issues that should be considered during the design stage, and for which appropriate provisions should be made in the Contract Documents.

6.10.1 Excavation and Temporary Protection Systems

If spread footings are adopted for support of the replacement structure or if excavations are required for removal of the existing structure, the excavations are expected to extend through the existing embankment fill (consisting of gravelly sand and silty sand fill) and into the dense to very dense sand to silty sand till. The excavations would extend up to about 6.4 m below the existing Post Road grade at the abutments and up to about 2 m below the existing Highway 401 median grade. At the abutments, if deep foundations are adopted, the excavations for pile caps could be maintained at a higher elevation within the approach embankments.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill, and glacial till above the water table would be classified as Type 3 soil, based on the OHSA. According to OHSA excavations that extend to, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Overburden below the water table would be classified as Type 4 soil, based on OSHA and excavations in these materials should be sloped no steeper than 3H:1V.

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

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

- It is considered that either a soldier pile and lagging system or an interlocking sheetpile system would be feasible at this site. The use of an interlocking sheetpile system has an advantage over soldier pile and lagging in that it would aid in groundwater control; however, the presence of cobbles and/or boulders in the glacial till may impact the depth that sheetpiling can be driven and the effectiveness of the system. Therefore, the preferred method of shoring would be soldier piles and lagging, with measures to control seepage and/or mitigate the loss of soil particles through the lagging boards.
- The soldier pile and lagging or sheetpiling would have to be socketed to sufficient depth to provide the necessary passive resistance for the retained soil height. Lateral support to the sheetpiles or soldier piles could be provided in the form of walers, tie-backs and/or internal struts/braces.

The design of a braced temporary protection system should be based on a rectangular earth pressure distribution (CFEM 2006; NAVFAC 1982) using the design parameters given below. Where the support to the wall is provided by anchors, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The supports must be designed to accommodate the loads applied from earth pressures and surcharge pressures from area, line or point loads as may be imposed by construction equipment and materials, as well as the impact of sloping ground behind the system.

Soil Type	Internal Angle of Friction (ϕ)	Unit Weight (γ , kN/m ³)	Coefficients of Earth Pressure		
			Active, K_a	At-Rest, K_0	Passive, K_p
Grade Fill	32	20	0.31	0.47	3.3
Glacial Till (Above Elev. 69 m)	33	21	0.29	0.46	3.4
Glacial Till (Below Elev. 69 m)	36	22	0.26	0.41	3.9

The temporary shoring design should be assessed for both the drained and undrained cases, based on the more conservative earth pressure conditions. The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the earth pressure loading should be adjusted accordingly.

Design of the temporary excavation support system should include an evaluation of base stability ("base heave" or soil squeezing stability) and hydraulic uplift stability as defined in the Canadian Foundation Engineering Manual (CFEM 2006).

The total passive resistance within the temporary protection system below the base of the excavation and below the base of the dewatered area should be calculated based on the values of K_p given above and then reduced by an appropriate factor of safety which considers the allowable wall movement as extrapolated from Figure C6.16 of the Canadian Highway Bridge Design Code (CHBDC 2014) to account for the fact that a large strain would be required for full mobilization of the passive resistance.

6.10.2 Groundwater and Surface Water Control

Based on readings taken at the monitoring well installed in median of Highway 401 and groundwater conditions observed in the boreholes immediately following drilling, the groundwater level is expected to be encountered about 4.6 to 6.1 m below the existing Post Road grade. During the 2014 field investigation, a standpipe piezometer was installed within the Highway 401 median at Borehole 14-723 with the tip at 5.2 m depth (Elevation 70.8 m). At that time, artesian groundwater conditions were encountered with the stable groundwater level at about 0.9 m above the existing ground surface at the time of drilling (about Elevation 76.9 m). During the 2017 investigation, standpipe piezometers were installed within Boreholes 17-727 and 17-728 with tips at Elevations 72.2 m and 73.2 m, respectively. At that time, the measured groundwater levels were measured at Elevations 76.4 m and 77.8 m, respectively.

The excavations required for construction of shallow foundations at the abutments and central pier, pile/caisson caps at the central pier, or RSS wall construction should be limited to no deeper than about Elevation 74.2 m due to the potential for artesian groundwater conditions. Where deeper excavations are required, such as those anticipated at the central pier, suitable shoring and groundwater control during construction will be required. Dewatering is recommended to lower the groundwater level to approximately 0.5 m below the footing founding level, to minimize disturbance of the subgrade. The water-bearing till at this site is relatively fine-grained (silty), and therefore will have a lower to moderate permeability.

According to Ontario Regulation 63/16 and Ontario Regulation 387/04, a Permit to Take Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 L/day is pumped from the excavations. If the volume of water to be pumped will be less than 400,000 L/day, but more than 50,000 L/day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. The anticipated dewatering rate based on the proposed construction methodology should be confirmed by the contractor to confirm if either a PTTW or EASR is required for this site.

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

6.10.3 Subgrade Protection

If the abutments or pier are to be founded on shallow spread footings, and where RSS walls are to be constructed, all embankment fill, topsoil, and soft or loose soils should be removed from below the proposed founding elevations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).

The glacial till that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, ponded water, and/or artesian groundwater conditions. 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. A Non-Standard Special Provision has been provided in Appendix E to address this requirement.

6.10.4 Vibration Monitoring During Pile Installation

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

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

6.10.5 Ground/Groundwater Control and Obstructions for Deep Foundation Installation

Where caissons are adopted, or if pre-augering is required for steel pile installation, the use of temporary or permanent liners will be required to minimize loss of ground through the water-bearing cohesionless till deposit.

The presence of cobbles and boulders in the glacial till could affect the installation of deep foundations or protection system elements. If caissons are to be used, appropriate drilling techniques will be required to advance the caissons through the glacial till. If driven H-piles are used, pre-augering of the pile locations may be required.

A Non-Standard Special Provision is provided in Appendix E, for inclusion in the Contract Documents to alert the Contractor to these conditions.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Mr. Bill Cavers P.Eng. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.



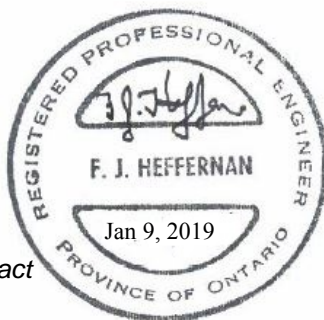
Matt Kennedy, P.Eng.
Senior Geotechnical Engineer



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Fin Heffernan, P.Eng.
Designated MTO Foundations Contact

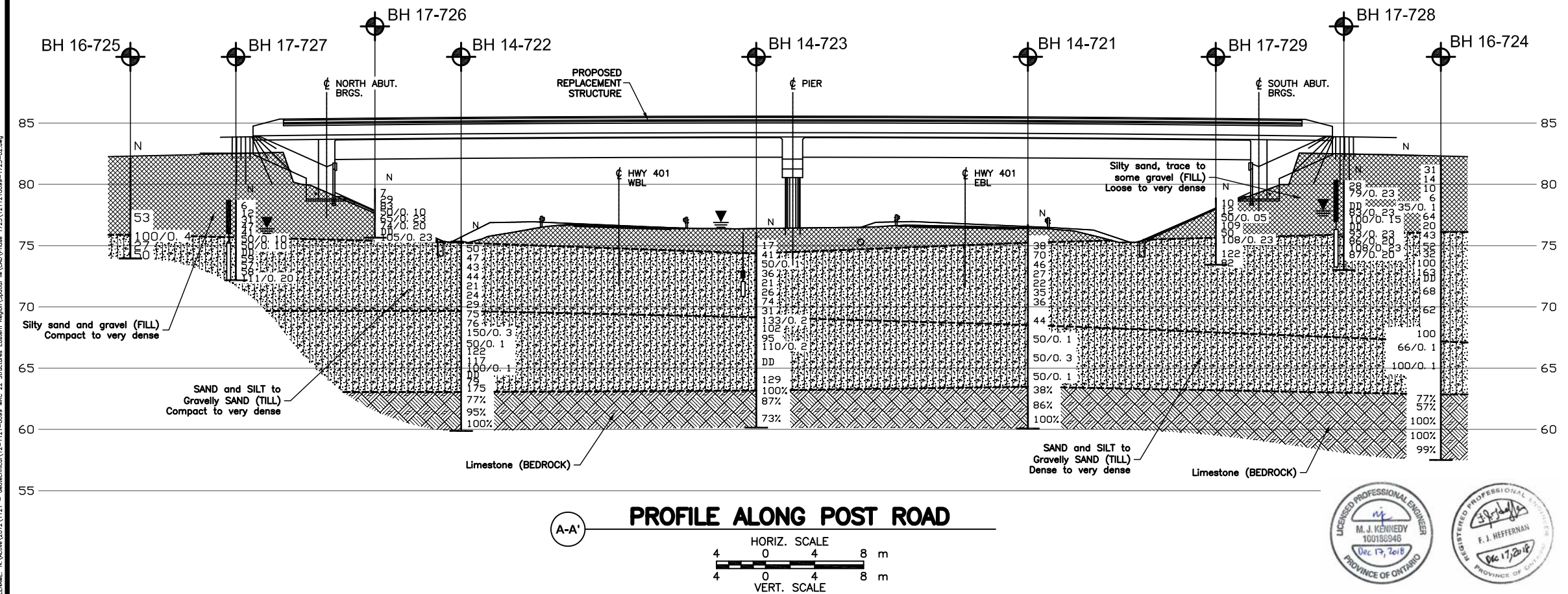
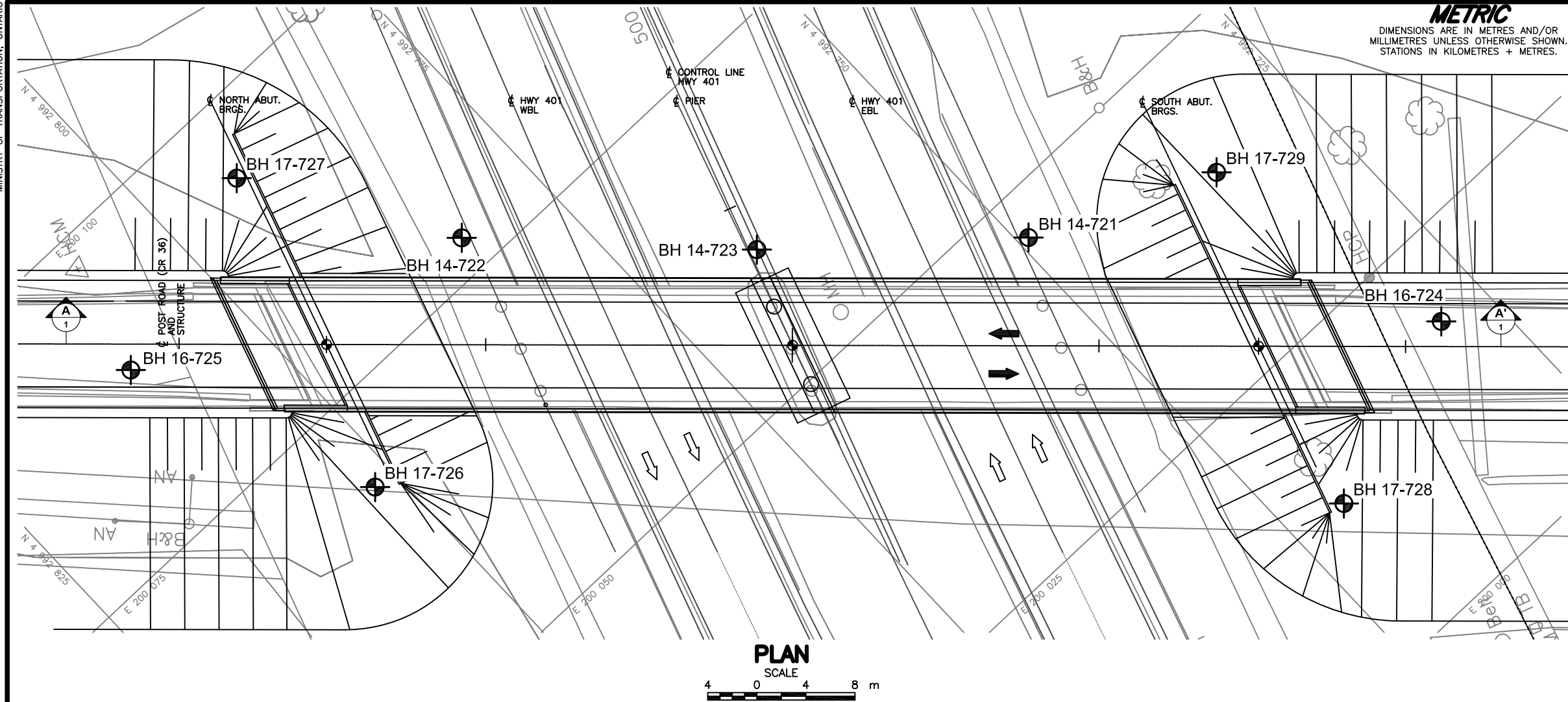
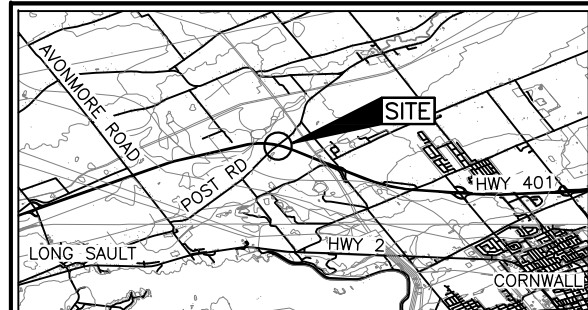


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Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread/strip footings on compact to very dense glacial till	<ul style="list-style-type: none"> Feasible at piers and abutments Preferred option from a foundations perspective 	<ul style="list-style-type: none"> Founding stratum at relatively shallow depth Option of founding on Granular A pad on glacial till to reduce footing depths 	<ul style="list-style-type: none"> Significant excavations to depths of greater than 7m at the abutment locations through the existing embankments Excavation to a depth of about 2m at the pier location, between the travelled lanes of Highway 401, will require temporary protection systems Groundwater control requirements during construction Lower geotechnical resistances as compared with deep foundations; potential for about 25 mm of settlement Precludes use of integral abutments; potentially greater maintenance required 	<ul style="list-style-type: none"> Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration 	<ul style="list-style-type: none"> Risk of instability of existing embankment slopes without appropriate temporary protection measures during excavation at abutments to significant depth
Steel H-piles driven to bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than for footings, reducing depth of excavation and temporary excavation support requirements Higher geotechnical resistances and negligible settlement Preferred foundation option for integral abutment construction 	<ul style="list-style-type: none"> Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles “hanging up” in the glacial till deposit and lower geotechnical resistances Temporary protection systems may be required at the central pier Some groundwater control would still be required at the central pier 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Moderate risk of driven H-piles “hanging up” in glacial till Contingency for pre-augering to penetrate deeper glacial till
Steel pipe (tube) piles, driven to found on bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system Higher geotechnical resistances and negligible settlement 	<ul style="list-style-type: none"> Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in more piles “hanging up”, lower geotechnical resistances, and greater potential for interference with existing piles Temporary protection systems may be required at central pier Some groundwater control would still be required at the central pier 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Moderate risk of pipe piles “hanging up” in glacial till Contingency for pre-augering to penetrate deeper glacial till
Caissons founded on bedrock	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Could eliminate the need for deep foundation cap at the central pier and allow for structural continuity between caissons and piers Construction from existing grade would reduce excavation and groundwater control requirements (reduced impact on Highway 401) 	<ul style="list-style-type: none"> Temporary or permanent liners required to control ground and groundwater in water-bearing till deposit Additional groundwater control required if artesian conditions encountered Rock coring, churn drilling or chisel drilling required to form rock sockets in strong to very strong bedrock 	<ul style="list-style-type: none"> Construction of deep caissons more expensive than alternative foundation options 	

**CONT No. 2017-4057**
WP No. 257-00-01**HWY. 401 - BRIDGE REPLACEMENT**
POST ROAD (CR 36) UNDERPASS
SITE NO. 31X-0179/B0
BOREHOLE LOCATIONS AND SOIL STRATA**SHEET**
20**Golder Associates Ltd.**
OTTAWA ONTARIO, CANADA**KEY PLAN**1 0 2
SCALE KM**LEGEND**

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- Seal
- Piezometer

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
14-721	75.9	4992749.1	200046.3
14-722	75.8	4992782.8	200077.9
14-723	76.0	4992766.0	200060.7
16-724	82.2	4992729.3	200018.3
16-725	82.2	4992809.9	200088.5
17-726	79.7	4992801.9	200067.9
17-727	78.7	4992792.9	200094.0
17-728	80.3	4992745.2	200012.9
17-729	78.9	4992734.3	200039.7

NOTES

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

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

REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. 3412039-7B-302-001_General-Arrangement.dwg, received Oct. 18, 2017.

NO.	DATE	BY	REVISION
Geocres No. 31G-256			
HWY. 401		PROJECT NO. 12-1121-0099	DIST. Eastern
SUBM'D. WAM	CHKD. WAM	DATE: NOV. 2017	SITE: 31X-0179
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1



APPENDIX A

Borehole and Drillhole Records

LIST OF SYMBOLS

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

I. GENERAL		(a) Index Properties (continued)	
π	3.1416	w	water content
$\ln x$,	natural logarithm of x	w_l or LL	liquid limit
\log_{10}	x or log x, logarithm of x to base 10	w_p or PL	plastic limit
g	acceleration due to gravity	I_p or PI	plasticity index = $(w_l - w_p)$
t	time	w_s	shrinkage limit
FoS	factor of safety	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)
II. STRESS AND STRAIN		(b) Hydraulic Properties	
γ	shear strain	h	hydraulic head or potential
Δ	change in, e.g. in stress: $\Delta \sigma$	q	rate of flow
ε	linear strain	v	velocity of flow
ε_v	volumetric strain	i	hydraulic gradient
η	coefficient of viscosity	k	hydraulic conductivity (coefficient of permeability)
ν	Poisson's ratio	j	seepage force per unit volume
	total stress		
σ'	effective stress ($\sigma' = \sigma - u$)	(c) Consolidation (one-dimensional)	
σ'_{vo}	initial effective overburden stress	C	compression index (normally consolidated range)
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, minor)	C_r	recompression index (over-consolidated range)
σ_{oct}	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$	C_s	swelling index
τ	shear stress	C_α	secondary compression index
u	porewater pressure	m_v	coefficient of volume change
E	modulus of deformation	c_v	coefficient of consolidation (vertical direction)
G	shear modulus of deformation	C_h	coefficient of consolidation (horizontal direction)
K	bulk modulus of compressibility	T_v	time factor (vertical direction)
		U	degree of consolidation
III. SOIL PROPERTIES		σ'_p	pre-consolidation stress
(a) Index Properties		OCR	over-consolidation ratio = σ'_p / σ'_{vo}
$\rho(\gamma)$	bulk density (bulk unit weight)*	(d) Shear Strength	
$\rho_d(\gamma_d)$	dry density (dry unit weight)	τ_p, τ_r	peak and residual shear strength
$\rho_w(\gamma_w)$	density (unit weight) of water	ϕ'	effective angle of internal friction
$\rho_s(\gamma_s)$	density (unit weight) of solid particles	δ	angle of interface friction
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)	μ	coefficient of friction = $\tan \delta$
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)	c'	effective cohesion
e	void ratio	C_u, S_u	undrained shear strength ($\phi = 0$ analysis)
n	porosity	p	mean total stress $(\sigma_1 + \sigma_3)/2$
S	degree of saturation	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

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

Notes: 1
2

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
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 cm² 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

Condition	N
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

	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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

<u>Description</u>	<u>Spacing</u>
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

<u>Term</u>	<u>Size*</u>
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.

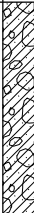
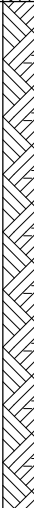
Abbreviations

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	



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

PROJECT <u>12-1121-0099-1720</u>		RECORD OF BOREHOLE No 14-721				SHEET 2 OF 3		METRIC	
G.W.P. <u>257-00-01</u>		LOCATION <u>N 4992749.1; E 200046.3 MTM NAD ZONE (LAT. ; LONG.)</u>				ORIGINATED BY <u>DWM/KE</u>			
DIST <u> </u> HWY <u> </u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core</u>				COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>November 6-10, 2014</u>				CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 25 50 75						
63.5	--- CONTINUED FROM PREVIOUS PAGE --- Silty SAND, trace to some gravel (TILL) Very dense Grey Wet		11	SS	50/0.1												
12.4	Limestone (BEDROCK) Bedrock cored from depths of 13.3 m to 16.7 m For bedrock coring details refer to Record of Drillhole 14-721																
			1	RC	REC 100%											RQD = 38%	
			2	RC	REC 99%											RQD = 86%	
			3	RC	REC 100%											RQD = 100%	
60.1	END OF BOREHOLE NOTES: 1. Artesian ground water flow encountered during drilling at approximately 5.8 m depth (Elev. 70.1 m).																
15.8																	

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\1211210099.GPJ GAL-GTA.GDT 12/18/18 JM

PROJECT: 12-1121-0099-1720

RECORD OF DRILLHOLE: 14-721

SHEET 3 OF 3

LOCATION: N 4992749.1 ;E 200046.3

DRILLING DATE: November 6-10, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES						
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX									
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	W1	W2		W3	W4	W5	W6		
							80 60 40 20 0	80 60 40 20 0																		
		Continued from Record of Borehole 14-721		63.48																						
	Rotary Drill NQ Core	Limestone (BEDROCK), with thin shale partings and interbeds		12.37	1	100																				
		Slightly weathered																								
		Thinly bedded																								
		Very strong																								
13		Limestone (BEDROCK), with thin shale partings and interbeds																								
		Slightly weathered to fresh																								
		Thinly to medium bedded																								
		Very strong																								
14						61.60	2	100																		
		Limestone (BEDROCK), with thin shale partings and interbeds																								
	Fresh																									
	Thinly bedded																									
15		Very strong		14.25	3	100																				
	Limestone (BEDROCK), with thin shale partings and interbeds																									
	Fresh																									
	Thinly bedded																									
		Very strong																								
16		END OF DRILLHOLE		60.07																						
				15.78																						
17																										
18																										
19																										
20																										
21																										
22																										

DEPTH SCALE

1 : 50



LOGGED: DWM/KE

CHECKED: MJK

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PROJECT 12-1121-0099-1720		RECORD OF BOREHOLE No 14-722		SHEET 1 OF 3		METRIC	
G.W.P. 257-00-01		LOCATION N 4992782.8; E 200077.9 MTM NAD ZONE (LAT. ; LONG.)		ORIGINATED BY DWM			
DIST _____ HWY _____		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY JM			
DATUM Geodetic		DATE November 4-6, 2014		CHECKED BY MJK			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	20	40	60		80	100			
75.8	GROUND SURFACE																				
0.0	Gravelly sand (FILL) Grey																				
75.3																					
0.5	Gravelly Silty SAND, contains cobbles (TILL) Dense Grey Moist		1	SS	50																
			2	SS	47																
			3	SS	43																
72.8																					
3.1	SAND and SILT, some gravel (TILL) Dense to compact Grey Moist		4	SS	44																
			5	SS	21																
			6	SS	24																
			7	SS	29																
69.7																					
6.1	Gravelly SAND, some silt (TILL) Very dense Grey Wet		8	SS	75																
69.1																					
6.7	Silty SAND, some gravel (TILL) Very dense Grey Wet		9	SS	76																
68.2																					
7.6	Silty SAND, some gravel, contains cobbles and boulders (TILL) Very dense Grey Wet		10	SS	150/0.3																
			11	SS	50/0.1																
66.7																					
9.1	SAND and GRAVEL, some silt, contains cobbles and boulders (TILL) Very dense Grey Wet		12	SS	122																

Continued Next Page

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

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PROJECT 12-1121-0099-1720		RECORD OF BOREHOLE No 14-722				SHEET 2 OF 3		METRIC	
G.W.P. 257-00-01		LOCATION N 4992782.8; E 200077.9 MTM NAD ZONE (LAT. ; LONG.)				ORIGINATED BY DWM			
DIST _____ HWY _____		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core				COMPILED BY JM			
DATUM Geodetic		DATE November 4-6, 2014				CHECKED BY MJK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L				
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)								
						20 40 60 80 100					25	50	75		GR SA SI CL				
63.6 12.2	--- CONTINUED FROM PREVIOUS PAGE --- SAND and GRAVEL, some silt, contains cobbles and boulders (TILL) Very dense Grey Wet		13	SS	117														
			14	SS	100/0.1														
			15	RC	DD														
			16	SS	79														
			17	SS	175														
63.1 12.7	Sandy SILT, contains sand seams (TILL) Very dense Grey Wet Limestone (BEDROCK) Bedrock cored from depths of 12.7 m to 15.9 m For bedrock coring details refer to Record of Drillhole 14-722		1	RC	REC 100%														
			2	RC	REC 100%														
			3	RC	REC 100%														
59.9 15.9	END OF BOREHOLE NOTES: 1. Artesian ground water flow encountered during drilling at approximately 11.6 m depth (Elev. 64.0 m).																		

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PROJECT: 12-1121-0099-1720

RECORD OF DRILLHOLE: 14-722

SHEET 3 OF 3

LOCATION: N 4992782.8 ;E 200077.9

DRILLING DATE: November 4-6, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³	W1		W2	W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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13	Rotary Drill NQ Core	Continued from Record of Borehole 14-722		63.06																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: DWM

CHECKED: MJK

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PROJECT 12-1121-0099-1720		RECORD OF BOREHOLE No 14-723		SHEET 1 OF 3		METRIC	
G.W.P. 257-00-01		LOCATION N 4992766.0; E 200060.7 MTM NAD ZONE (LAT. ; LONG.)		ORIGINATED BY DWM			
DIST _____ HWY _____		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY JM			
DATUM Geodetic		DATE November 3-4, 2014		CHECKED BY MJK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	W _p	W		W _L						
76.0	GROUND SURFACE																						
0.0	Sand and gravel, trace fines (FILL) Compact Grey																						
			1	SS	17													49	43	5	3		
74.3																							
1.7	SAND and SILT, some gravel, contains cobbles (TILL) Dense to very dense Grey Moist to wet		2	SS	41																		
			3	SS	50/0.1																		
73.0																							
3.1	Gravelly Silty SAND, some gravel, contains cobbles (TILL) Dense to compact Grey Wet		4	SS	36																		
			5	SS	21														23	43	25	9	
			6	SS	26																		
70.7																							
5.3	Silty SAND and GRAVEL (TILL) Very dense Grey Wet		7	SS	74																		
69.9																							
6.1	Gravelly SAND, some silt (TILL) Dense Grey to black Wet		8	SS	31														29	59	10	2	
69.1																							
6.9	Silty SAND, some gravel (TILL) Very dense Grey Moist to wet		9	SS	133/0.2																		
68.6																							
7.4	Silty SAND, some gravel (TILL) Very dense Grey Wet		10	SS	102																		
			11	SS	95															14	55	27	4
66.8																							
9.2	Silty SAND and GRAVEL, contains cobbles and boulders (TILL) Very dense Grey Wet		12	SS	110/0.2																		

Continued Next Page

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

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

PROJECT: 12-1121-0099-1720

RECORD OF DRILLHOLE: 14-723

SHEET 3 OF 3

LOCATION: N 4992766.0 ;E 200060.7

DRILLING DATE: November 3-4, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES				
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 100	DIP w.r.t. CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX						
						TOTAL CORE %	SOLID CORE %				Jr	Ja	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	W1	W2		W3	W4	W5	W6
						80 80																

DEPTH SCALE

1 : 50



LOGGED: DWM

CHECKED: MJK

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PROJECT 12-1121-0099-1720		RECORD OF BOREHOLE No 16-724		SHEET 1 OF 3		METRIC	
G.W.P. 257-00-01		LOCATION N 4992729.3; E 200018.3 MTM NAD ZONE (LAT. ; LONG.)		ORIGINATED BY KM			
DIST _____ HWY _____		BOREHOLE TYPE Power Auger/200 mm Diam. (Hollow Stem), Wash Boring/HW Casing		COMPILED BY JJL			
DATUM Geodetic		DATE December 20, 21, and 23, 2016		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p W W _L				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)											
82.2	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE																			
0.1	Gravelly sand (FILL) Brown						82													
81.6																				
81.4	Sand, some silt, trace gravel (FILL) Brown																			
0.8	Silty sand, trace to some gravel, contains limestone pieces (FILL) Loose to dense Brown Moist		1	SS	31		81													
			2	SS	14															
			3	SS	10		80													
			4	SS	6		79													
			5	SS	35/0.1															
							78													
77.6	Silty sand, some gravel (FILL) Very dense Brown Moist		6	SS	64															
4.6																				
			7	SS	20		77													
76.1	Silty SAND, some gravel to gravelly Silty SAND, contains cobbles (TILL) Dense to very dense Grey Wet		8	SS	43		76													
6.1																				
			9	SS	52		75													
			10	SS	32															
							74													
			11	SS	100															
			12	SS	163		73													

Continued Next Page

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

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PROJECT <u>12-1121-0099-1720</u>		RECORD OF BOREHOLE No 16-724		SHEET 2 OF 3		METRIC	
G.W.P. <u>257-00-01</u>		LOCATION <u>N 4992729.3; E 200018.3 MTM NAD ZONE (LAT. ; LONG.)</u>		ORIGINATED BY <u>KM</u>			
DIST <u> </u> HWY <u> </u>		BOREHOLE TYPE <u>Power Auger/200 mm Diam. (Hollow Stem), Wash Boring/HW Casing</u>		COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>December 20, 21, and 23, 2016</u>		CHECKED BY <u> </u>			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED												
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	Silty SAND, some gravel to gravelly Silty SAND, contains cobbles (TILL) Dense to very dense Grey Wet		13	RC	DD																
			14	SS	68																
70.0																					
12.2	SAND, some gravel, trace silt Compact Grey Wet		15	SS	62								○					12 78 8 2			
69.6																					
12.6	Silty SAND, some gravel, containing cobbles and boulders (TILL) Very dense Grey Wet																				
			16	SS	100								○								
			17	SS	66/0.1																
			18	SS	100/0.1																
63.3																					
18.9	Silty SAND Compact Grey Wet		19	WS	N/A																
62.9																					
19.4			1	RC	REC 100%													RQD = 77%			
			2	RC	REC 92%													RQD = 57%			

Continued Next Page

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

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PROJECT <u>12-1121-0099-1720</u>		RECORD OF BOREHOLE No 16-724				SHEET 3 OF 3		METRIC	
G.W.P. <u>257-00-01</u>		LOCATION <u>N 4992729.3; E 200018.3 MTM NAD ZONE (LAT. ; LONG.)</u>				ORIGINATED BY <u>KM</u>			
DIST <u> </u> HWY <u> </u>		BOREHOLE TYPE <u>Power Auger/200 mm Diam. (Hollow Stem), Wash Boring/HW Casing</u>				COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>December 20, 21, and 23, 2016</u>				CHECKED BY <u> </u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
						<div style="display: flex; justify-content: space-between; font-size: small;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between; font-size: small;"> 25 50 75 </div>						
	--- CONTINUED FROM PREVIOUS PAGE --- Limestone (BEDROCK) Bedrock cored from depths of 19.4 m to 24.7 m For bedrock coring details refer to Record of Drillhole 16-724		2	RC	REC 92%	62										UCS = 33.2 MPa	RQD = 57%
			3	RC	REC 100%	61											RQD = 100%
			4	RC	REC 100%	60											RQD = 100%
			5	RC	REC 100%	59											
						58											RQD = 99%
57.5 24.7	END OF BOREHOLE																

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PROJECT: 12-1121-0099-1720

RECORD OF DRILLHOLE: 16-724

SHEET 3 OF 3

LOCATION: N 4992729.3 ; E 200018.3

DRILLING DATE: December 20, 21, and 23, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY K, cm/sec				WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁵	10 ⁻⁴	10 ⁻³		W1	W2	W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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		Continued from Record of Borehole 16-724		62.86																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	</

UCS =
33.2 MPa

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: KM

CHECKED:

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

PROJECT <u>12-1121-0099-1723</u>		RECORD OF BOREHOLE No 17-726		SHEET 1 OF 1		METRIC	
G.W.P. <u>257-00-01</u>		LOCATION <u>N 4992801.9; E 200067.9 MTM ZONE (LAT. ; LONG.)</u>		ORIGINATED BY <u>KM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable, Wash Boring/NW Casing</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 7, 2017</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
79.7	GROUND SURFACE																			
0.0	Sandy silt, contains organic matter (rootlets) (TOPSOIL)		1	SS	7															
0.1	Dark brown Moist																			
	Sand and silt, some gravel to gravelly, contains cobbles and boulders (FILL)		2	SS	29															
	Compact to very dense Brown Moist																			
			3	SS	83															
			4	SS	50/0.10															
			5	RC	DD															
77.3	Sand and gravel, contains cobbles (FILL)		6	SS	65/0.23															
2.4	Very dense Brown Moist																			
76.7	Silty SAND, some gravel, contains cobbles and boulders (TILL)		7	SS	74/0.20															
3.1	Very dense Grey Wet		8	RC	DD															
			9	SS	105/0.23															
75.7																				
4.0	END OF BOREHOLE																			

PROJECT		12-1121-0099-1723		RECORD OF BOREHOLE No 17-727				SHEET 1 OF 1				METRIC					
G.W.P.		257-00-01		LOCATION				N 4992792.9; E 200094.0 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY KM					
DIST		Eastern HWY 401		BOREHOLE TYPE				Portable, Wash Boring/NW Casing				COMPILED BY ZS					
DATUM		Geodetic		DATE				November 6, 2017				CHECKED BY MJK					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
78.7	GROUND SURFACE							20	40	60	80	100					
0.0	Silty sand (TOPSOIL)																
0.1	Dark brown Moist		1	SS	6												
	Silty sand, some gravel, contains cobbles, boulders and organic matter (rootlets) (FILL)																
	Loose to compact Brown Moist		2	SS	12		78										
77.5	Gravelly silty sand, contains cobbles (FILL)																
1.2	Compact to dense Grey-brown Moist		3	SS	31		77										28 35 30 7
77.0	Organic silt, some sand, trace gravel (FILL)																
1.7	Compact Dark brown Moist		4	SS	47												
76.7	Silty sand to sandy silt, some gravel, contains organic matter (organic silt pockets) (FILL)																
2.0	Dense Grey-brown and dark brown Moist		5	SS	41		76										16 44 31 9
			6	SS	50/0.10												
75.0																	
3.7	Silty SAND, some gravel to gravelly, contains cobbles and boulders (TILL)		7	SS	50/0.10		75										
	Very dense Grey-brown Moist		8	RC	DD												
73.8			9	SS	59		74										
4.9	Gravelly sandy SILT, contains cobbles (TILL)																
	Very dense Grey Moist to wet		10	SS	57												22 28 44 6
			11	SS	58		73										
			12	SS	111/0.20												
72.2	END OF BOREHOLE																
6.6	NOTES: 1. Water level in well screen at a depth of 2.4 m below ground surface (Elev. 76.4 m), measured on Nov. 15, 2017.																

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PROJECT 12-1121-0099-1723		RECORD OF BOREHOLE No 17-728		SHEET 1 OF 1		METRIC	
G.W.P. 257-00-01		LOCATION N 4992745.2; E 200012.9 MTM ZONE (LAT. ; LONG.)		ORIGINATED BY KM			
DIST Eastern HWY 401		BOREHOLE TYPE Portable, Wash Boring/NW Casing		COMPILED BY ZS			
DATUM Geodetic		DATE November 8-9, 2017		CHECKED BY MJK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								20	40	60	80						100	25

80.3	GROUND SURFACE															
0.0	Sandy silt, contains organic matter (rootlets) (TOPSOIL)															
80.1	Dark brown Moist		1	SS	28											
0.2	Silty sand, some gravel, contains cobbles and boulders (FILL)		2	SS	79/0.23											
	Very dense															
	Brown															
	Moist															
			3	RC	DD											
77.9	Sandy gravel, some silt, contains cobbles and boulders (FILL)		4	SS	83/0.23											
2.4	Very dense															
	Brown															
	Moist															
			5	SS	100/0.15											61 26 10 3
76.7	Silty SAND, some gravel to gravelly, contains cobbles and boulders (TILL)		6	RC	DD											
3.6	Very dense		7	SS	93/0.23											24 39 29 8
	Grey															
	Moist to wet		8	SS	86/0.20											
			9	SS	108/0.23											
			10	SS	87/0.20											8 49 35 8
										</						

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PROJECT <u>12-1121-0099-1723</u>		RECORD OF BOREHOLE No 17-729		SHEET 1 OF 1		METRIC	
G.W.P. <u>257-00-01</u>		LOCATION <u>N 4992734.3; E 200039.7 MTM ZONE (LAT. ; LONG.)</u>		ORIGINATED BY <u>KM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable, Wash Boring/NW Casing</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 8, 2017</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE	20	40	60	80	100	● QUICK TRIAXIAL × REMOULDED	W _p		W	W _L		
78.9	GROUND SURFACE						20	40	60	80	100									
0.0	Sandy silt, contains organic matter (rootlets) (TOPSOIL)																			
0.2	Dark brown Moist		1	SS	10															
	Sand and silt, some gravel, contains cobbles and boulders (FILL)		2	SS	12															
	Compact to very dense		3	SS	50/0.05															
	Brown		4	RC	DD															
	Moist		5	SS	109															
76.5	Silty sand and gravel, contains gravelly sand seams, cobbles and boulders (FILL)		6	SS	50															
2.4	Very dense		7	SS	108/0.23															
	Brown																			
	Moist																			
75.5	SAND and SILT, some gravel to gravelly, contains cobbles (TILL)																			
3.4	Very dense		8	SS	122															
	Grey																			
	Moist to wet		9	SS	82															
73.4																				
5.5	END OF BOREHOLE																			

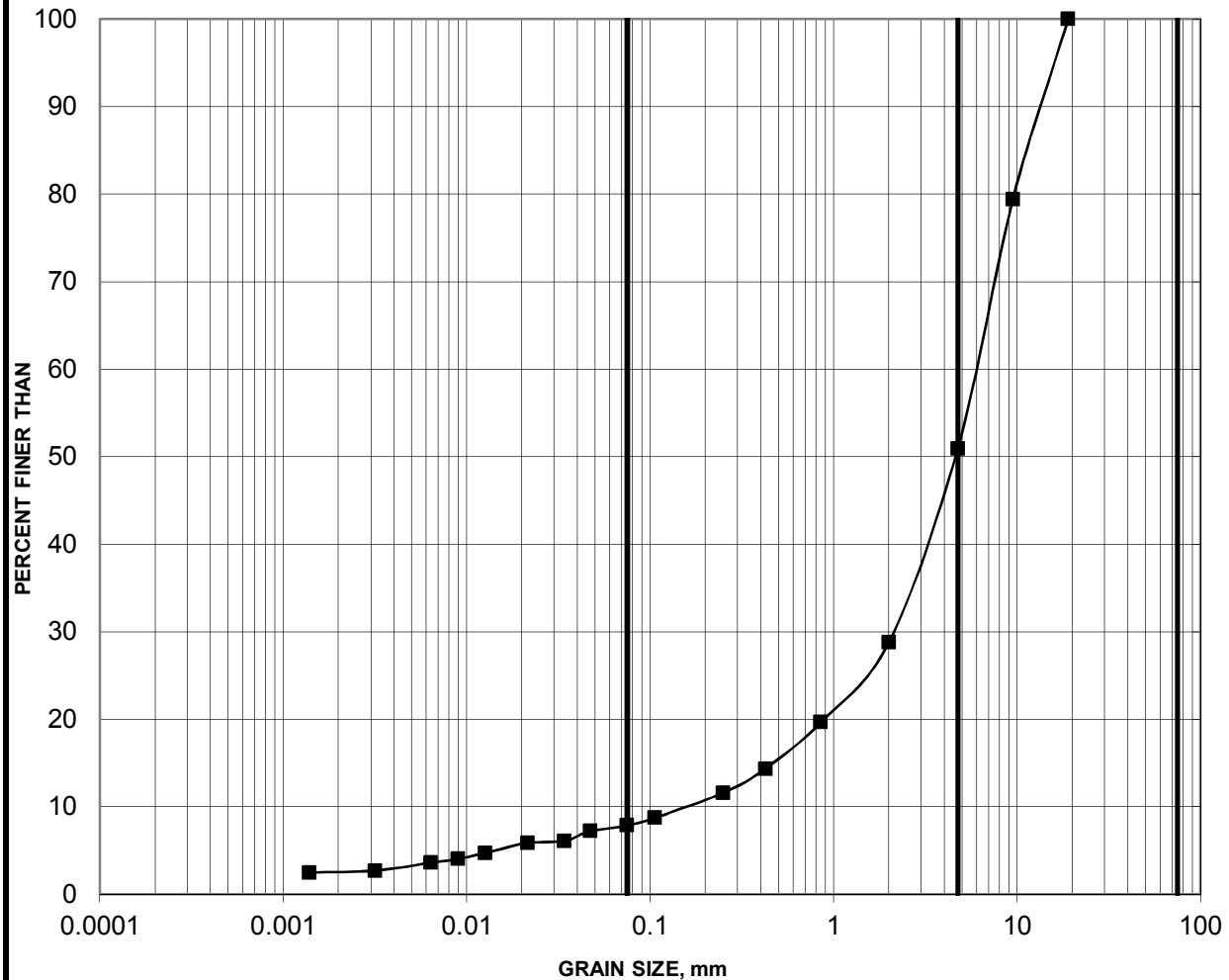
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND AND GRAVEL (MEDIAN GRADE FILL)



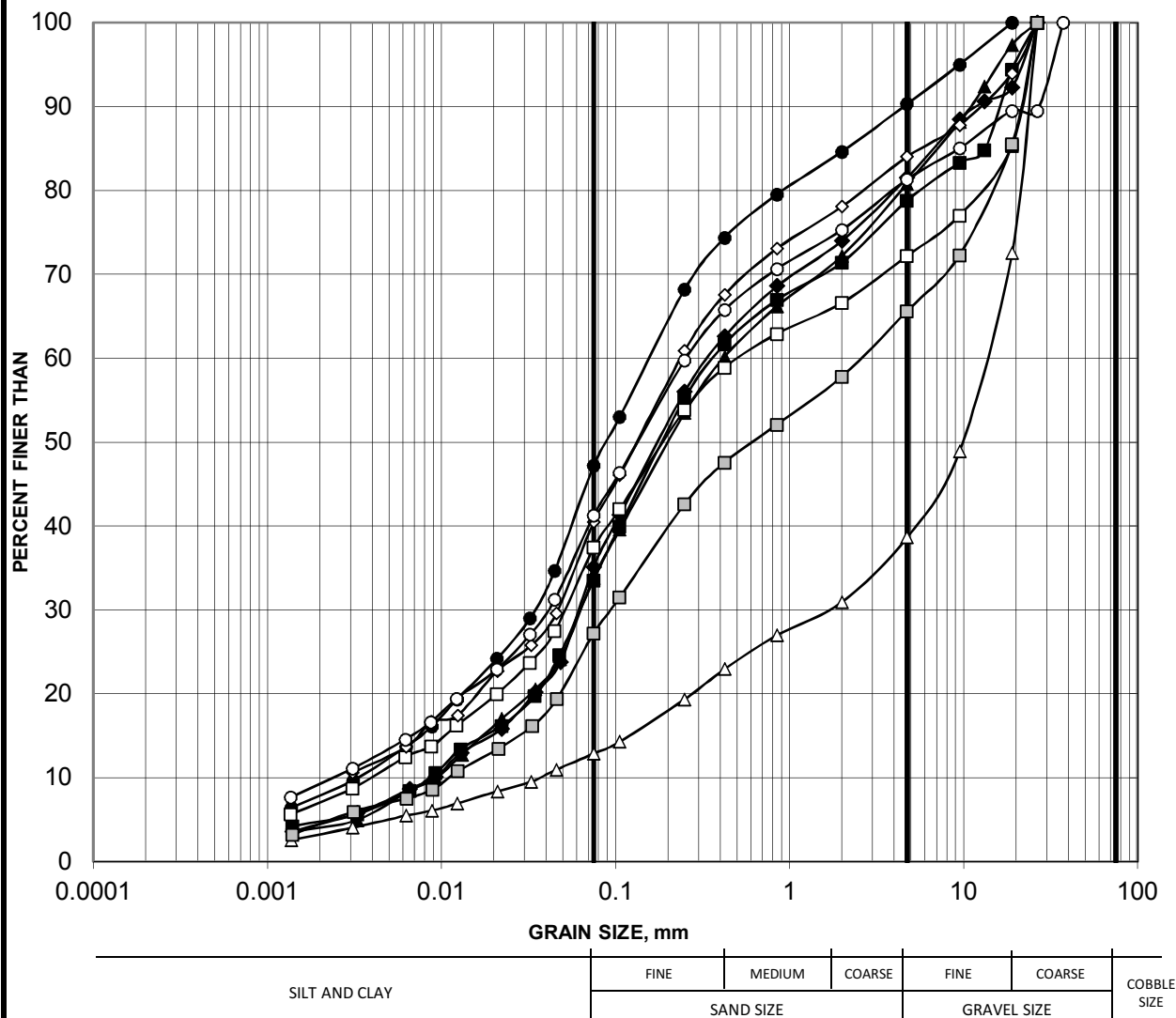
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
14-723	1	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND, SOME GRAVEL (EMBANKMENT FILL)

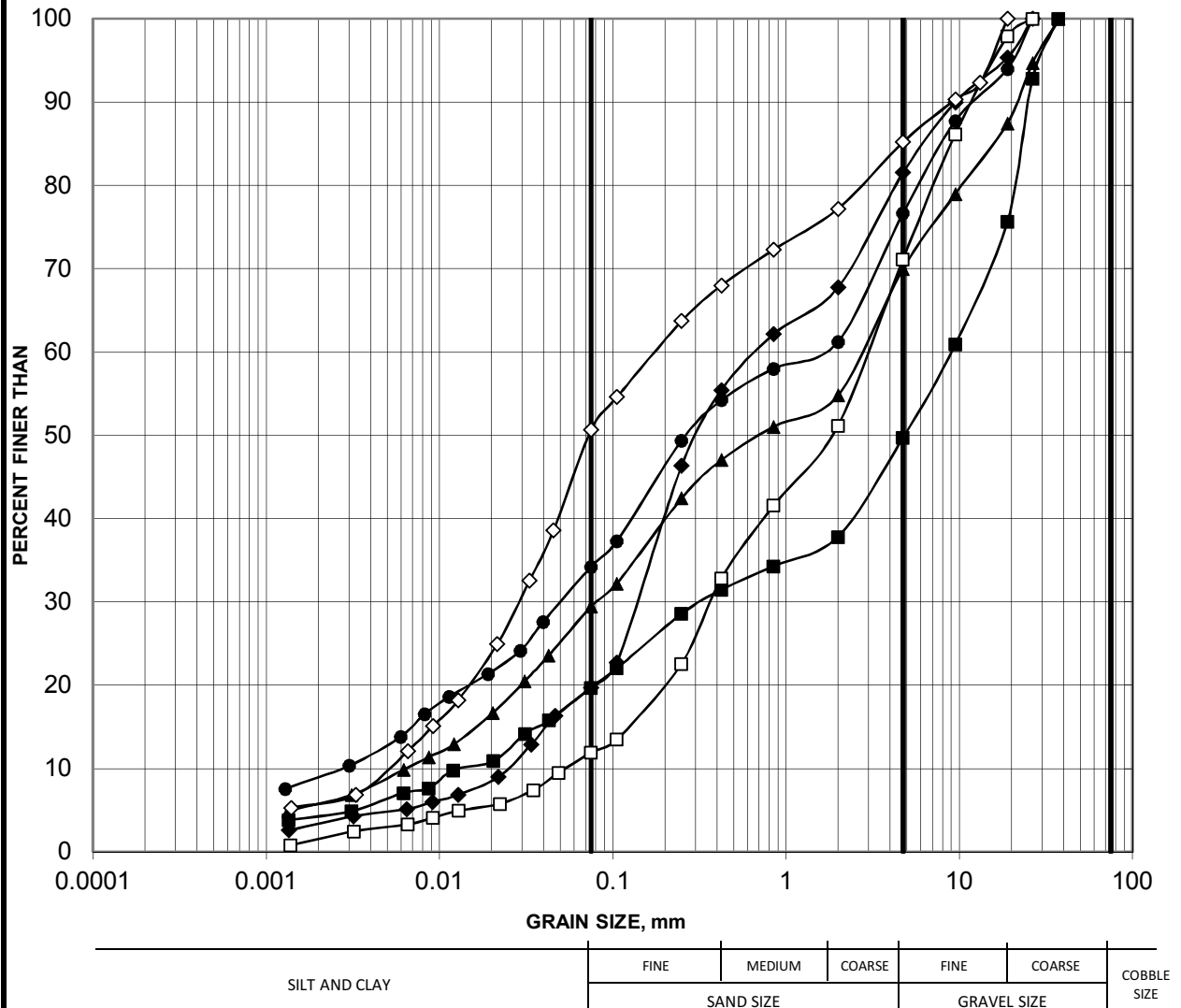


Borehole	Sample	Depth (m)
16-724	4	3.05-3.66
16-724	6	4.57-5.18
16-725	1	4.57-5.18
17-726	3	1.22-1.83
17-727	3A	1.22-1.52
17-727	5	2.44-3.05
17-728	5	3.05-3.51
17-729	2	0.61-1.22
17-729	6	2.44-3.05

GRAIN SIZE DISTRIBUTION

FIGURE B3A

SILTY SAND AND GRAVEL (UPPER GLACIAL TILL)

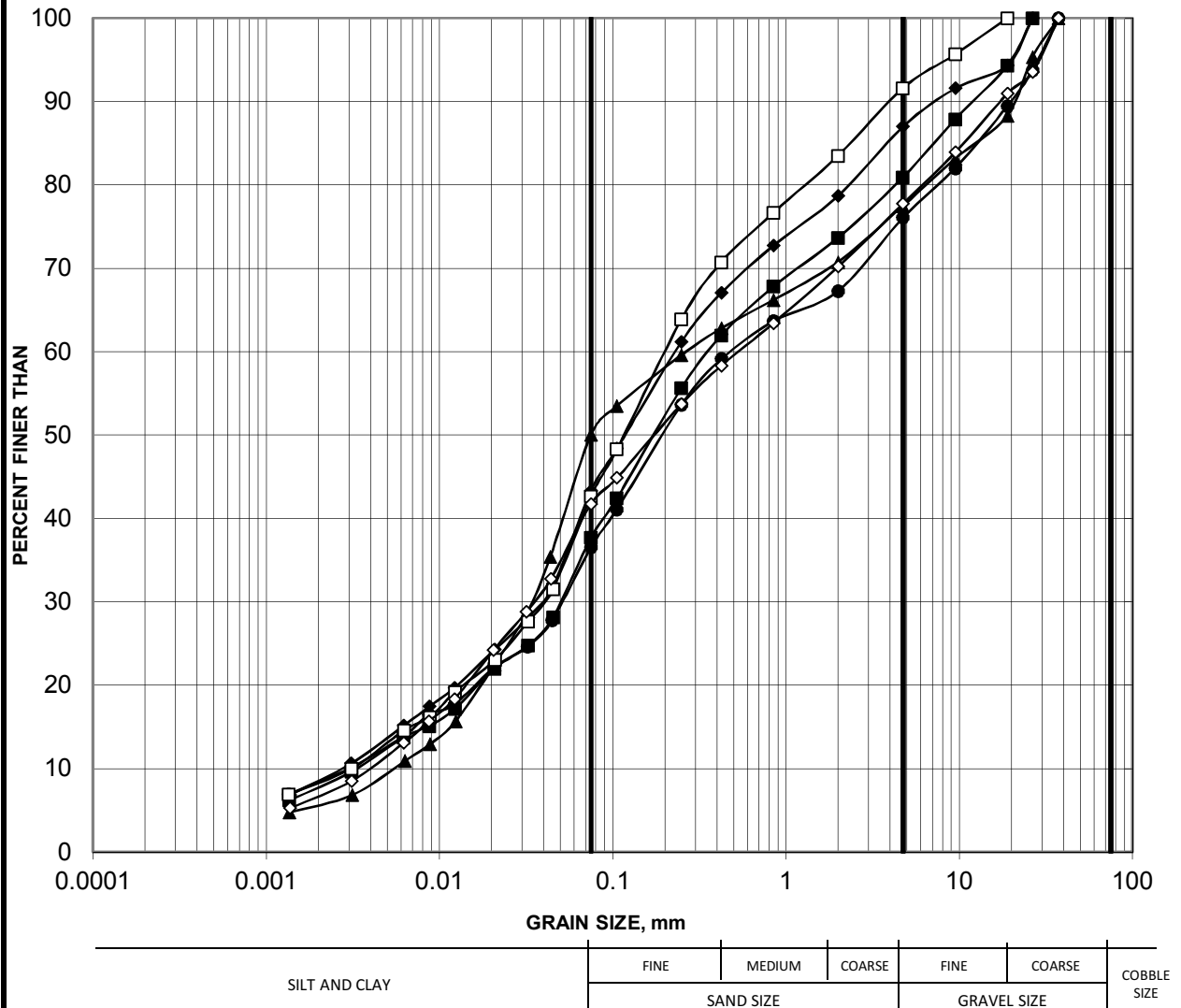


Borehole	Sample	Depth (m)
14-721	2	1.39-2.00
14-721	8	6.72-7.33
14-722	3	2.29-2.90
14-723	5	3.81-4.42
14-723	8	6.10-6.71
16-725	3	6.86-7.47

GRAIN SIZE DISTRIBUTION

FIGURE B3B

SILTY SAND AND GRAVEL (UPPER GLACIAL TILL)

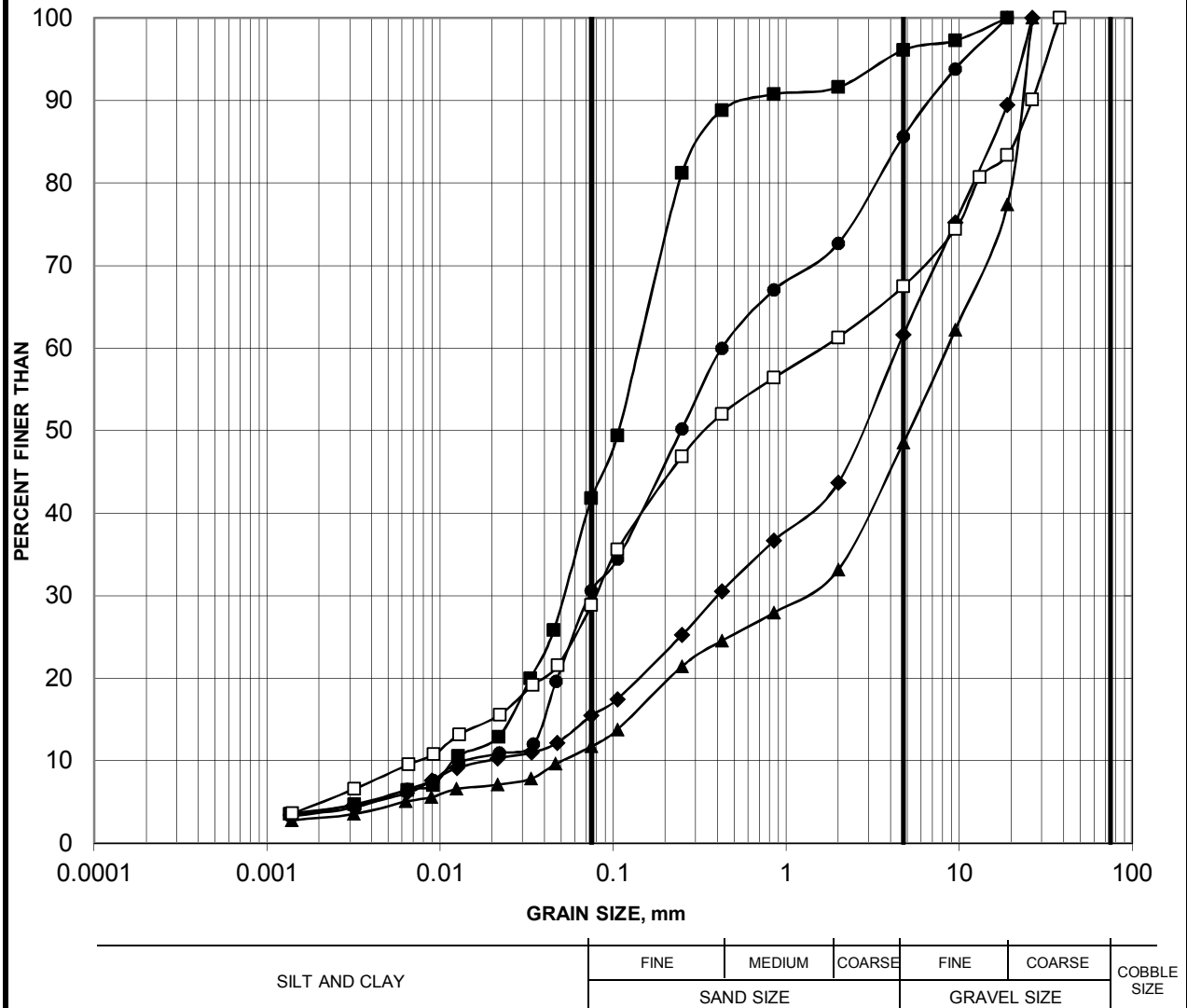


Borehole	Sample	Depth (m)
■ 17-726	7	3.05-3.25
◆ 17-726	9	3.66-4.04
▲ 17-727	10	4.88-5.49
● 17-728	7	3.66-4.04
□ 17-728	10	5.49-5.69
◇ 17-729	9	4.27-4.88

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY SAND AND GRAVEL (LOWER GLACIAL TILL)

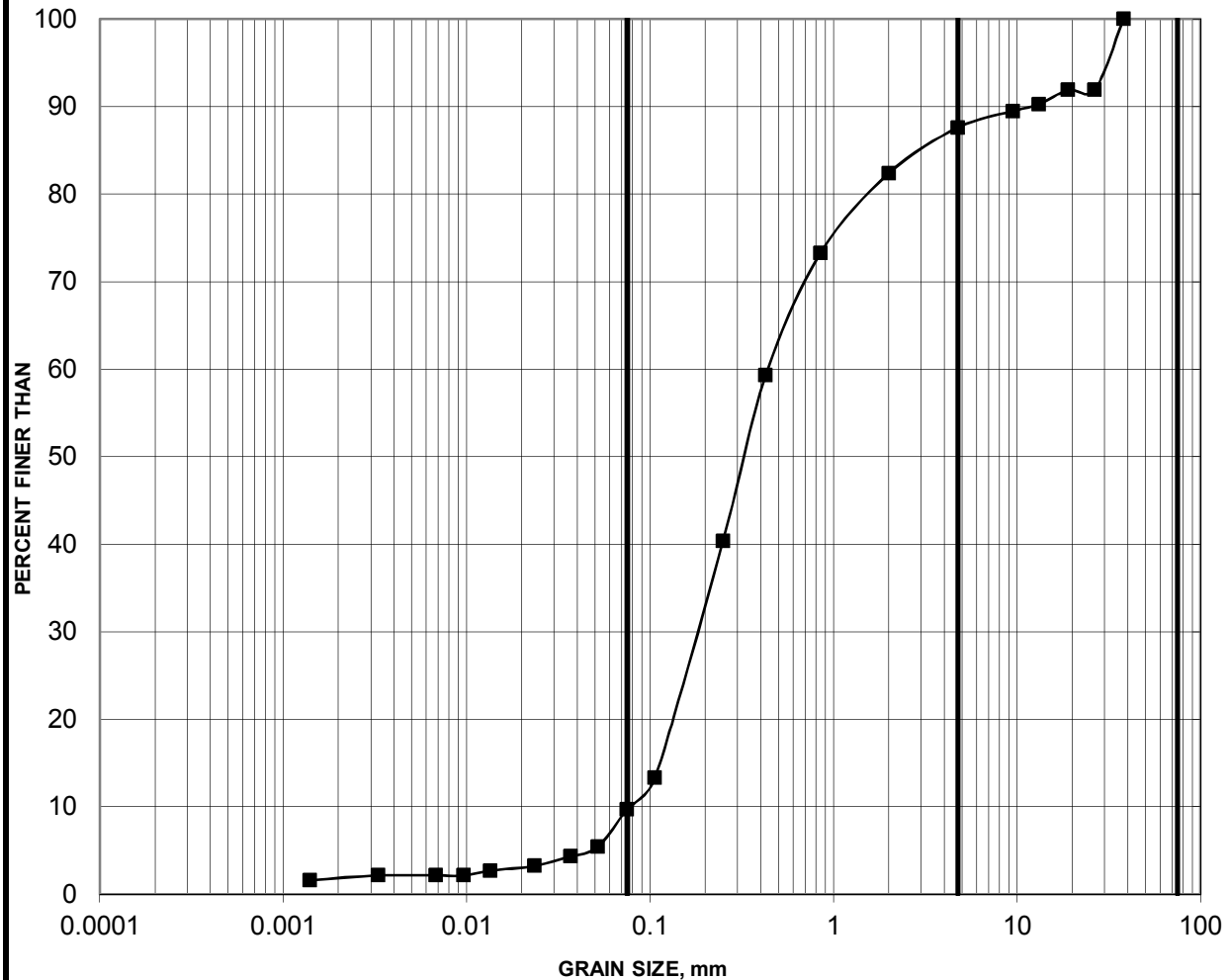


Borehole	Sample	Depth (m)
14-721	10	9.77-10.04
14-722	8	6.10-6.71
14-722	13	9.91-10.37
14-723	11	8.38-8.99
16-724	10	7.62-8.23

GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND, SOME GRAVEL

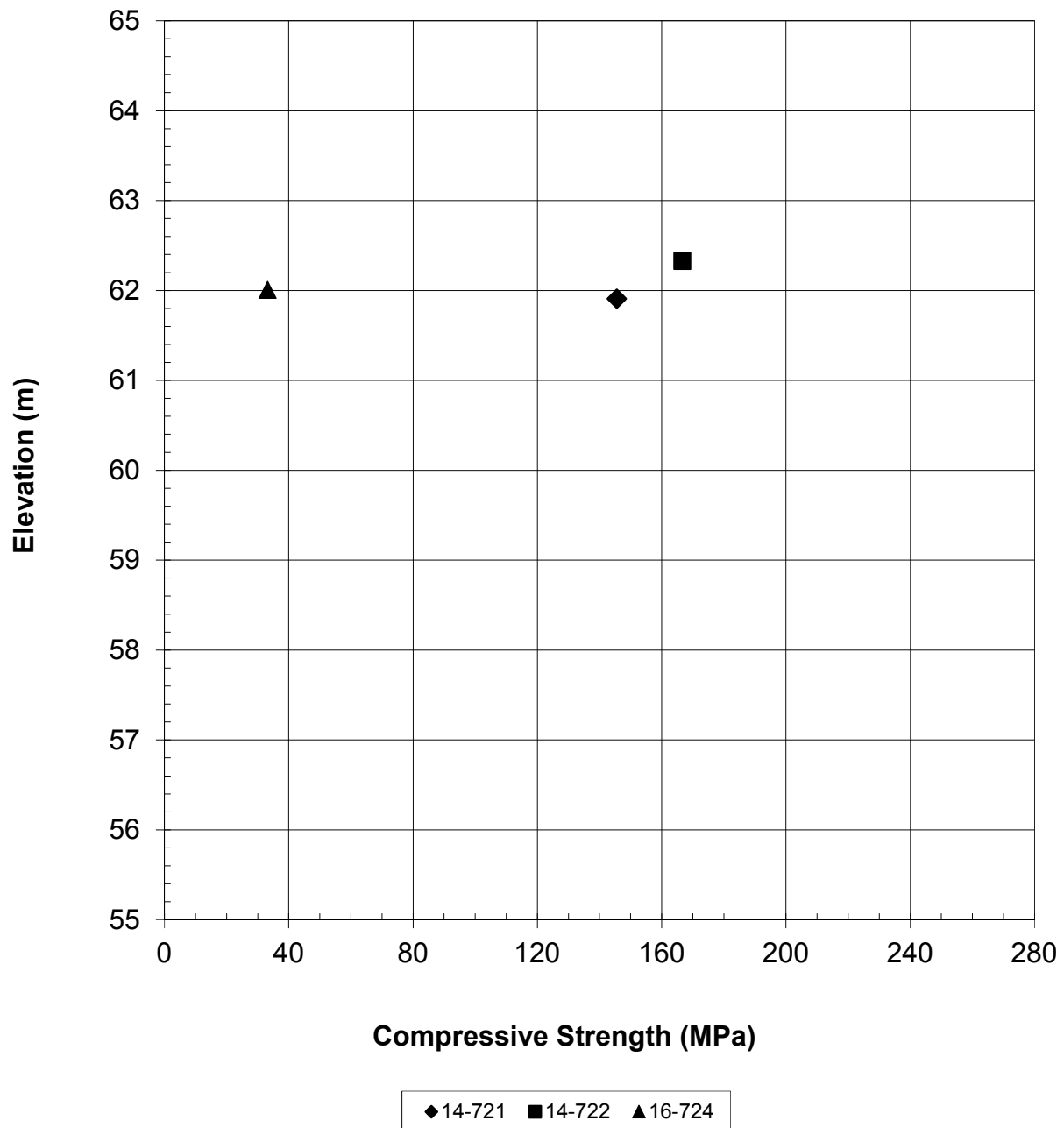


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-724	15	12.20-12.80

**SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
UNCONFINED COMPRESSION TESTS**

FIGURE B6



APPENDIX C

Vertical Seismic Profiling Test Results

DATE January 20, 2017**PROJECT No.** 12-1121-0099/1720**TO** Susan Trickey
Golder Associates Ltd.**FROM** Stephane Sol, Christopher Phillips**EMAIL** ssol@golder.com, cphillips@golder.com**VERTICAL SEISMIC PROFILING TEST RESULTS
HWY 401 AND HWY 36, CORNWALL, ONTARIO**

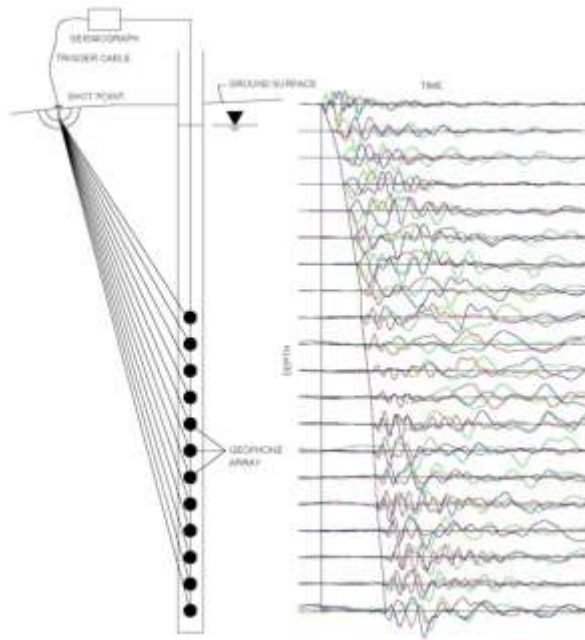
This memorandum presents the results of the Vertical Seismic Profiling (VSP) testing carried out at the intersection of HWY 401 and HWY 36 near Cornwall Ontario. VSP testing was completed in borehole BH16-724. VSP testing was carried out on January 13, 2017. BH16-724 was drilled to an approximate depth of 24.7 metres below the existing ground surface and then cased with a PVC pipe grouted in place. The overburden consisted of approximately 6.1 metres of silty sand Fill and 13.2 m of Silty sand. The shale bedrock started at 19.35 mbgs with one metre of weathered shale at the top.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the 2010 National Building Code of Canada.





Example 1: Layout and resulting time traces from a VSP survey.

Fieldwork

The fieldwork was carried out on January 13, 2017, by personnel from the Golder Mississauga and Ottawa offices.

Both compression and shear-wave seismic sources were used and both were located 2 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a metallic U-shaped object that was horizontally struck with a 9.9 kilogram sledge hammer on alternate ends to induce polarized shear waves. Test measurements started at 1 m from ground surface and were recorded in the borehole with a 3-component receiver spaced at 1 metre intervals below the ground surface to a maximum depth of the casing (24 m).

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

- 1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
- 2) Low Pass Filtering of data to remove spurious high frequency noise;
- 3) First break picking of the compression and shear-wave arrivals; and,
- 4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following four plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths for Borehole BH16-724. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

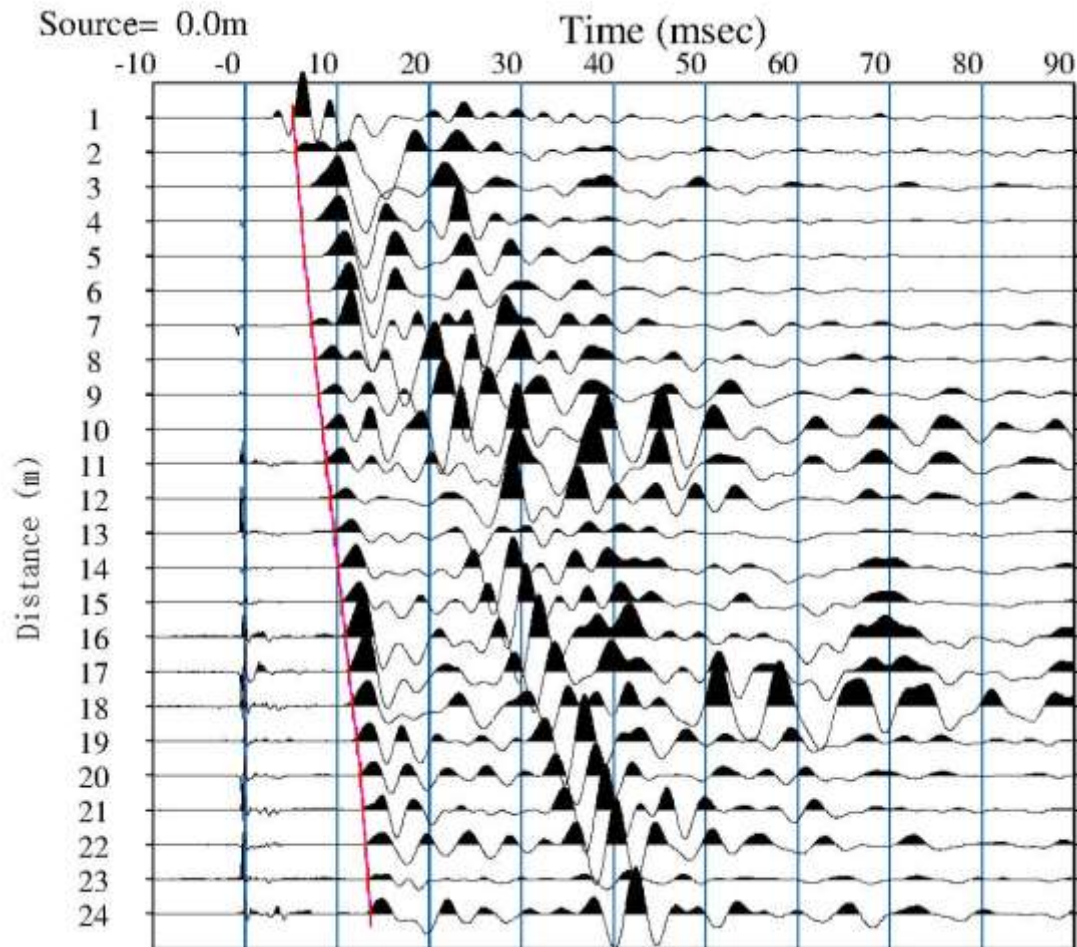


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-724.

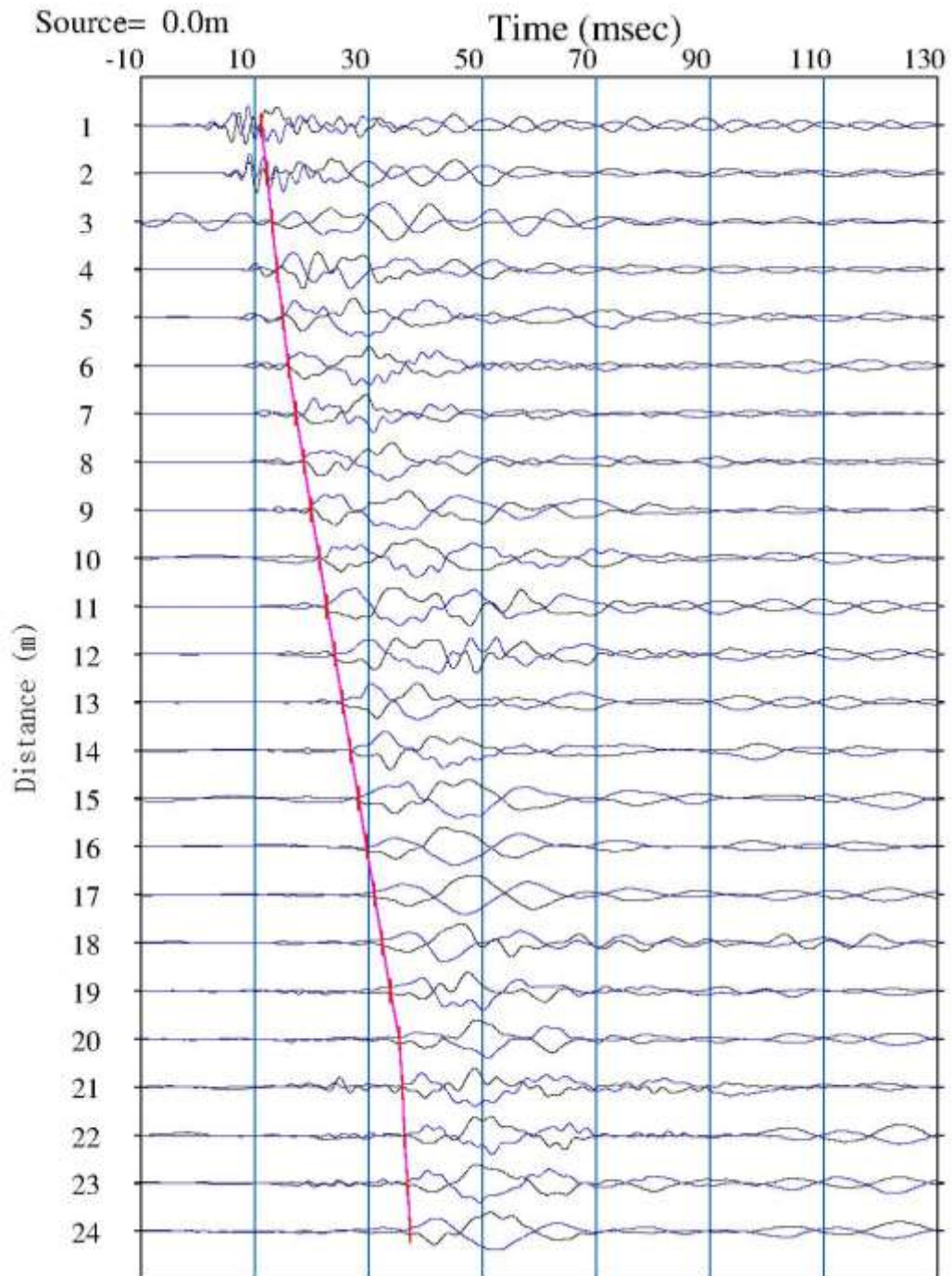


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-724.

Results

The VSP results are summarized in Table 1. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. For the top 19 m of overburden, the bulk density of 1,600 kg/m³ was used. For the bedrock down to the bottom of the borehole at 24 metres, a bulk density of 2,600 kilogram per cubic metre was used.

The average shear wave velocity from ground surface to a depth of 30 metres was measured to be 741 metres per second. The average velocity was calculated assuming that the velocity from 24 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 1,900 m/s which is equal to the velocity of the limestone bedrock at the bottom of the borehole.

Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.



Stephane Sol, Ph.D., P.Geo
Senior Geophysicist



Christopher Phillips, M.Sc., P.Geo
Principal, Senior Geophysicist

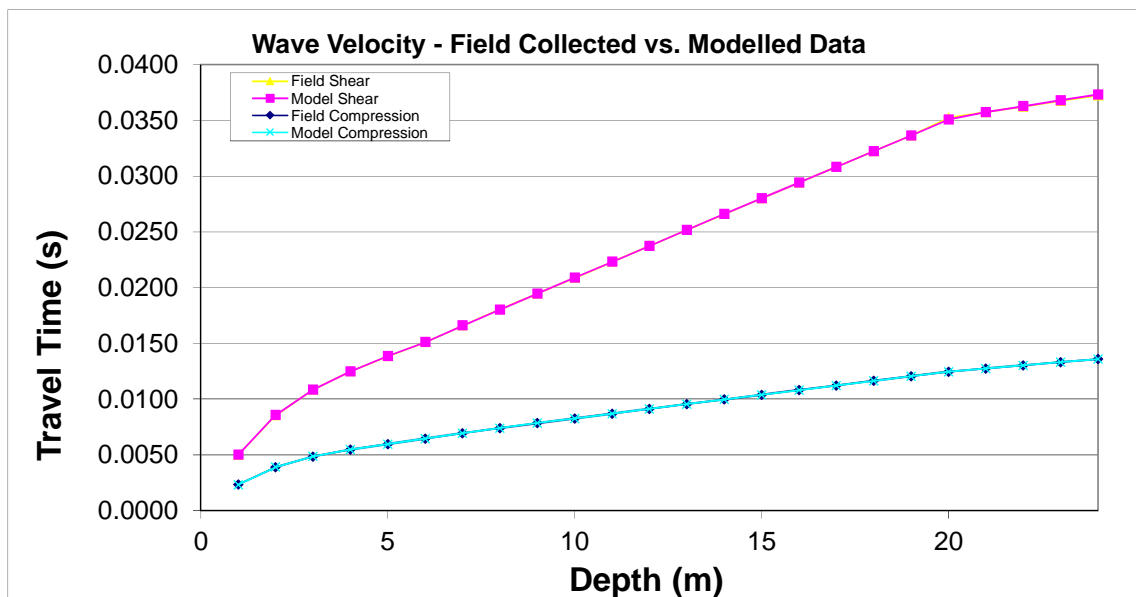
SS/CRP/jl

\\golder.gds\gal\ottawa\active\2012\1121 - geotechnical\12-1121-0099 mrc 22 structures eastern region\survey data\1720 post rd\geophysics\results\12-1121-0099 1720 tech memo2017jan20 vsp mrc 22 bridge.docx

Attachment: Table 1 – Shear Wave Velocity Profile at BH-16-724

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BH16-724

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m ³)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1	430	200	1600	0.36	64	174	211
1.0	2	630	280	1600	0.38	125	345	468
2.0	3	1060	440	1600	0.40	310	865	1385
3.0	4	1600	610	1600	0.41	595	1685	3302
4.0	5	2050	730	1600	0.43	853	2434	5587
5.0	6	2070	800	1600	0.41	1024	2892	5491
6.0	7	2100	670	1600	0.44	718	2073	6098
7.0	8	2150	700	1600	0.44	784	2259	6351
8.0	9	2200	700	1600	0.44	784	2264	6699
9.0	10	2400	700	1600	0.45	784	2279	8171
10.0	11	2300	700	1600	0.45	784	2272	7419
11.0	12	2400	700	1600	0.45	784	2279	8171
12.0	13	2300	700	1600	0.45	784	2272	7419
13.0	14	2400	700	1600	0.45	784	2279	8171
14.0	15	2400	710	1600	0.45	807	2342	8141
15.0	16	2400	710	1600	0.45	807	2342	8141
16.0	17	2400	710	1600	0.45	807	2342	8141
17.0	18	2400	710	1600	0.45	807	2342	8141
18.0	19	2400	710	1600	0.45	807	2342	8141
19.0	20	2500	700	2600	0.46	1274	3714	14551
20.0	21	3500	1500	2600	0.39	5850	16234	24050
21.0	22	3550	1900	2600	0.30	9386	24390	20252
22.0	23	3550	1900	2600	0.30	9386	24390	20252
23.0	24	3550	1900	2600	0.30	9386	24390	20252

**Notes**

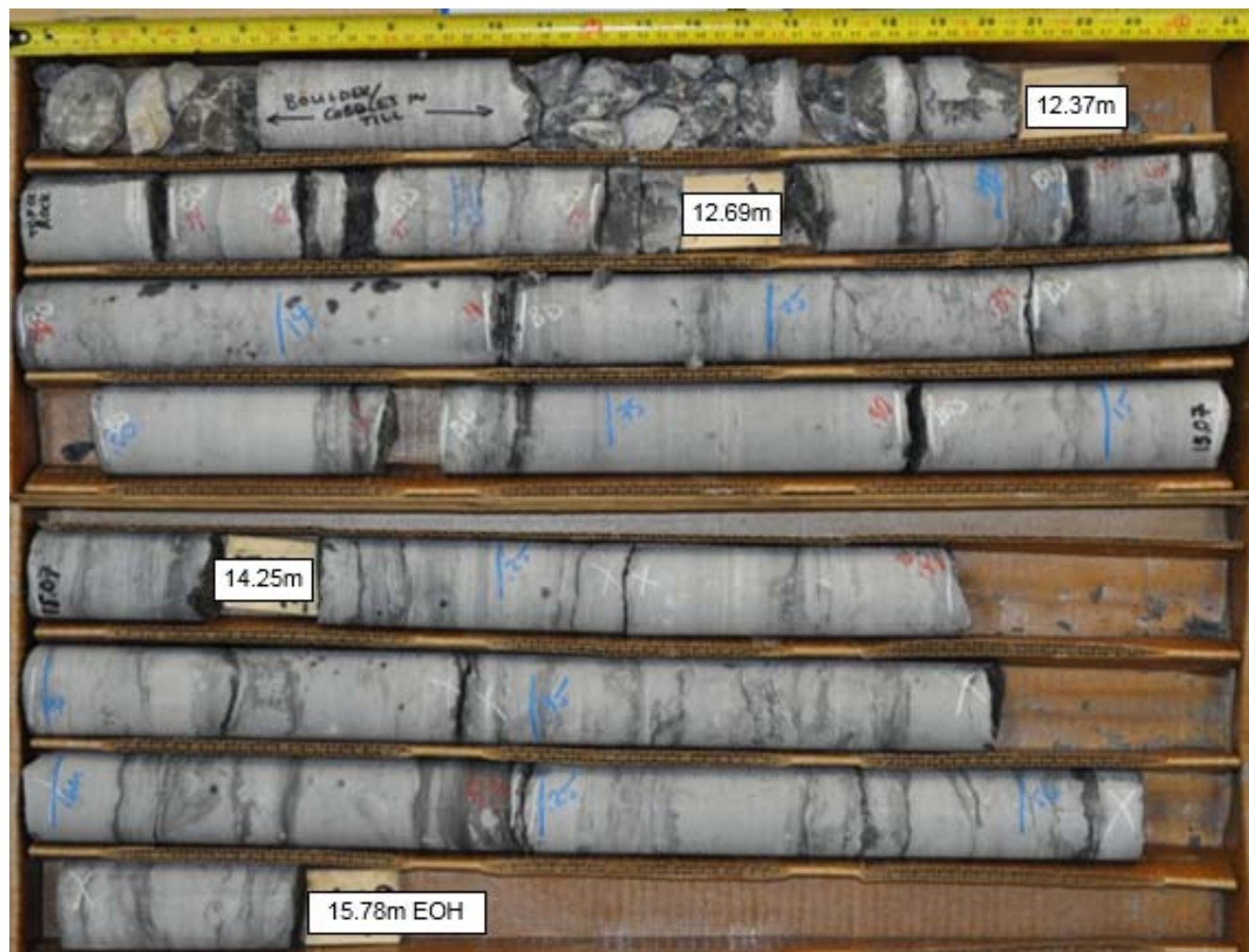
1. Depth presented relative to ground surface.
2. This table to be analyzed in conjunction with the accompanying report.

APPENDIX D

Bedrock Core Photographs

Bedrock Core Photographs
Borehole 14-721 (12.4 m to 15.8 m)

Figure D1



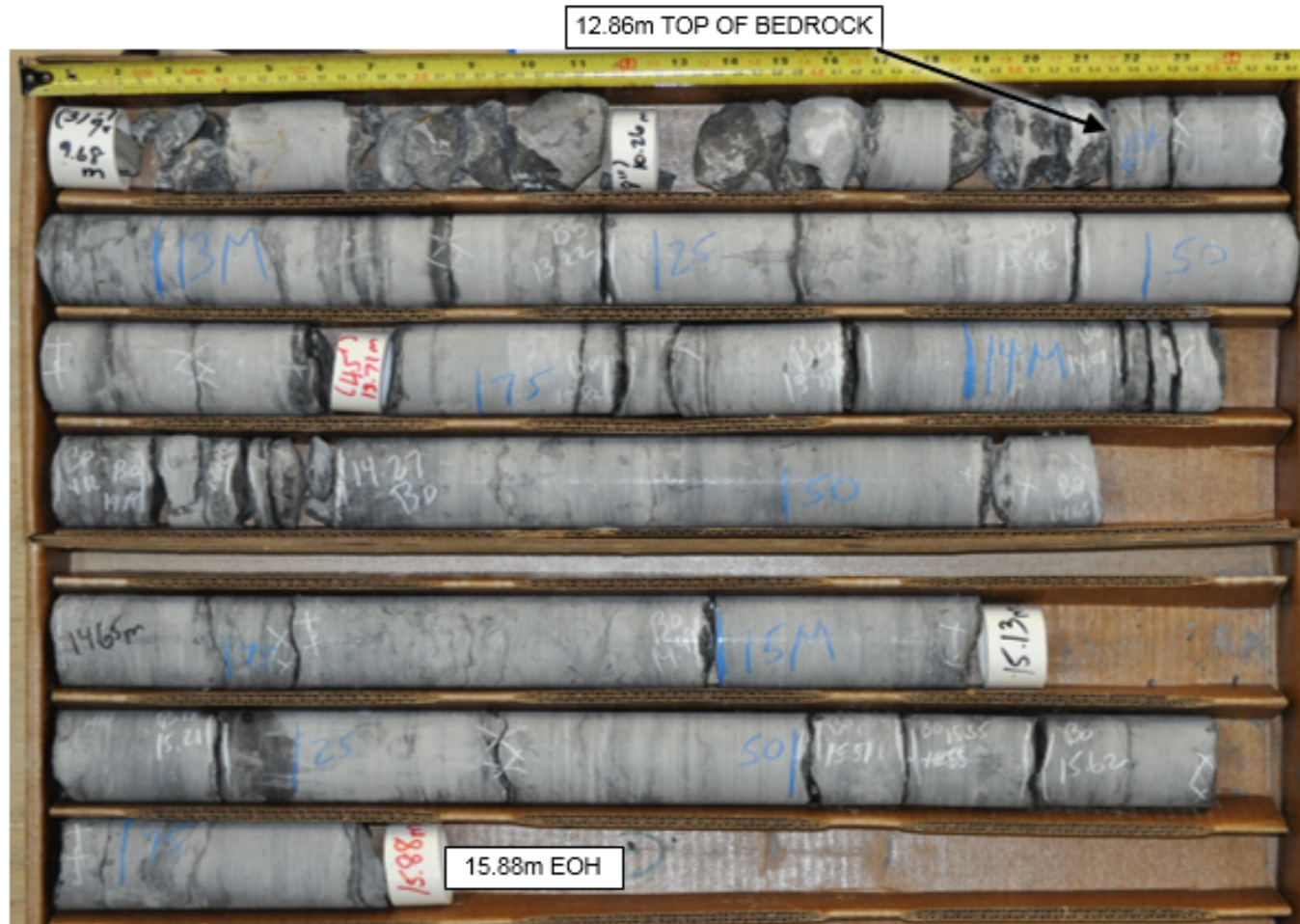
Note: Borehole 14-721 was advanced and logged in the field from the top of a 0.91m thick temporary granular pad placed above the existing ground surface. The depths provided in the report and shown above are relative to the existing ground surface.

Figure D2



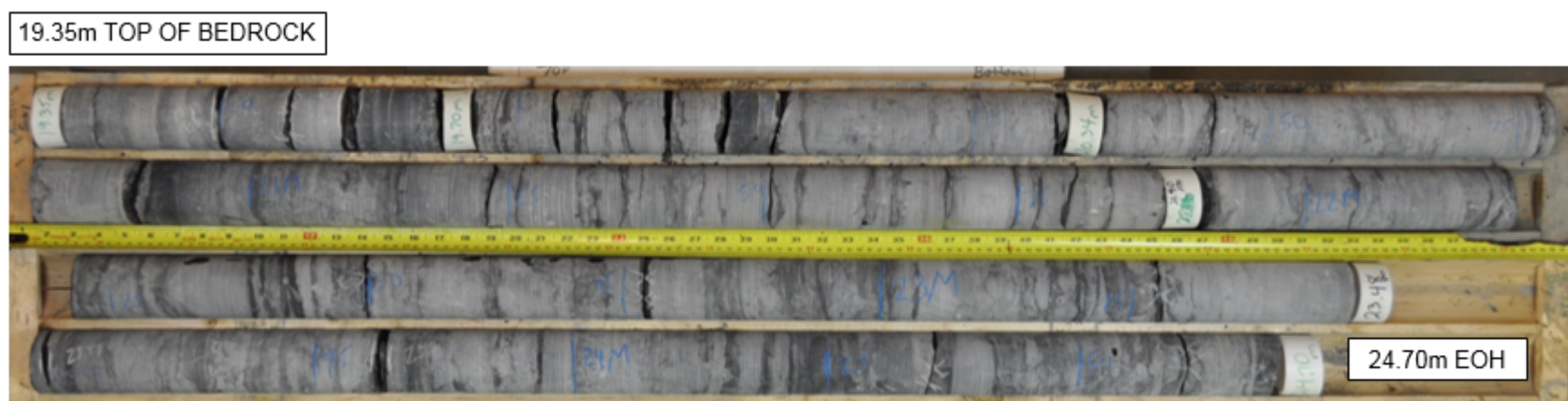
Bedrock Core Photographs
Borehole 14-723 (12.9 m to 15.9 m)

Figure D3



Bedrock Core Photographs
Borehole 14-724 (19.4 m to 24.7 m)

Figure D4



APPENDIX E

Non-Standard Special Provisions

DEWATERING STRUCTURE EXCAVATIONS – Item No.

Special Provision

Amendment to OPSS 902

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.02 Submission Requirements

Section 902.04.02 is amended by the addition of the following Subsection:

902.4.02.03 Dewatering

At least two weeks prior to commencing dewatering operations, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of working drawings.

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Section 902.07.04 is amended by the addition of the following:

The Contractor is advised that construction of the new centre pier foundation will require excavation below the groundwater level or may encounter artesian conditions in the cohesionless till deposit. Cohesionless soils below the groundwater table will be subjected to conditions of unbalanced hydrostatic head and can slough, boil and cave in during temporary excavation work. The Contractor shall reference borehole records as shown elsewhere in the Contract Documents as a guide in determining dewatering requirements.

A continuous dewatering operation shall be provided to facilitate the foundation construction operations at all times. The dewatering system shall be adequate to lower the groundwater level to at least 0.5 m below the founding level for the new centre pier, to allow excavation, subgrade preparation and foundation construction in dry conditions. All components of the dewatering system shall be maintained in an effective, functioning and stable condition during the construction.

The dewatering requirements will be influenced by the excavation depths and construction methodologies selected by the Contractor. If the Contractor's staging and construction methodology for removals and/or new foundation construction will result in high water inflows, their dewatering plan should include retention of a dewatering specialist.

The work for dewatering shall be completed in accordance with the environmental and operational constraints specified elsewhere in the Contract Documents.

WORKING SLAB – Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab on top of approved subgrade under structure foundations.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling – Structures

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

Concrete for working slabs shall have a minimum 28-day strength of 20 MPa. The concrete curing requirements of OPSS 904 shall not apply.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Within four hours following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Dewatering

Dewatering shall be carried out in accordance with OPSS 902.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab – Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

VIBRATION MONITORING – Item No.

Special Provision

1.0 SCOPE

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

2.0 DEFINITIONS

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

3.0 SUBMISSION REQUIREMENTS

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

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

4.0 MONITORING

The vibration monitoring equipment shall be placed on the existing Post Road (County Road 36) underpass. The Contractor shall take readings on the existing structure throughout pile driving operations, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

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

5.0 BASIS OF PAYMENT

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

END OF SECTION

DEEP FOUNDATIONS – Item No.

Special Provision

1.0 SCOPE

The predominant soil deposit at this site is a water-bearing cohesionless till, which contains cobbles and boulders. The Contractor is advised that cohesionless soils are susceptible to disturbance under conditions of unbalanced hydrostatic head, and that appropriate equipment and construction procedures will be required for pre-augering into the till for steel piles, or for caisson construction through the till deposit. The Contractor is also advised that appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundation elements and shoring elements.

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

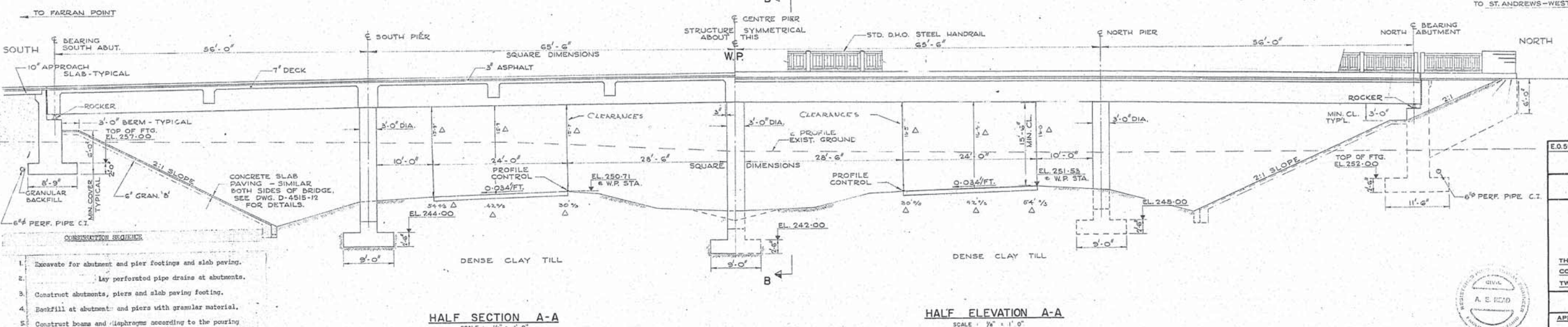
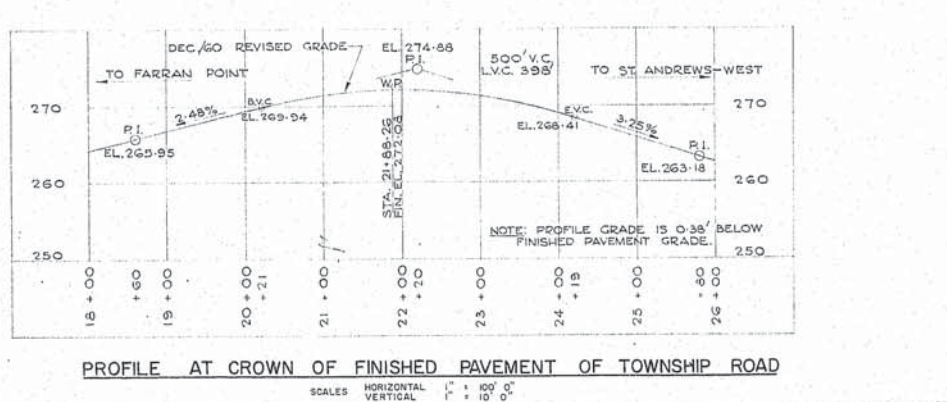
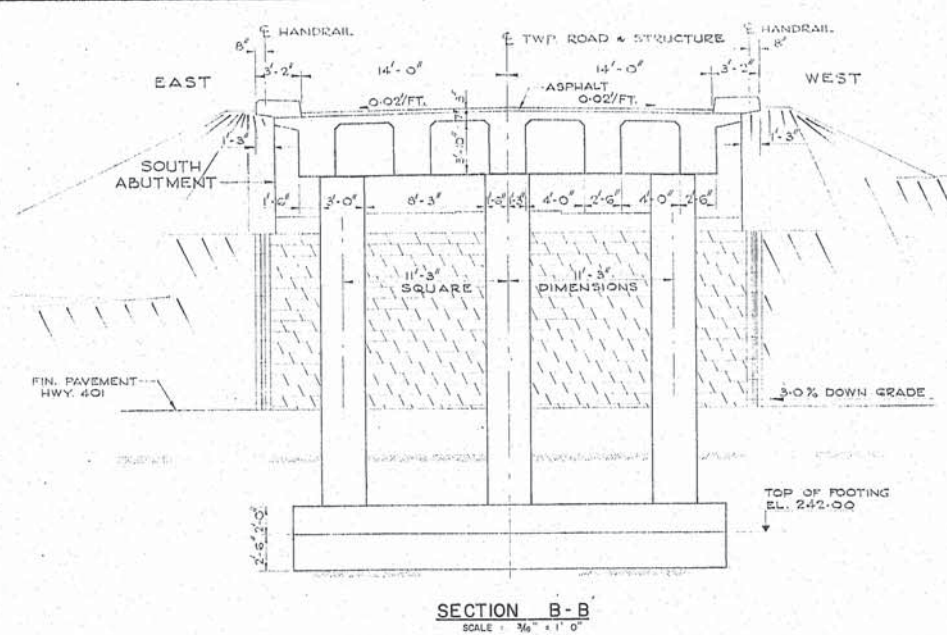
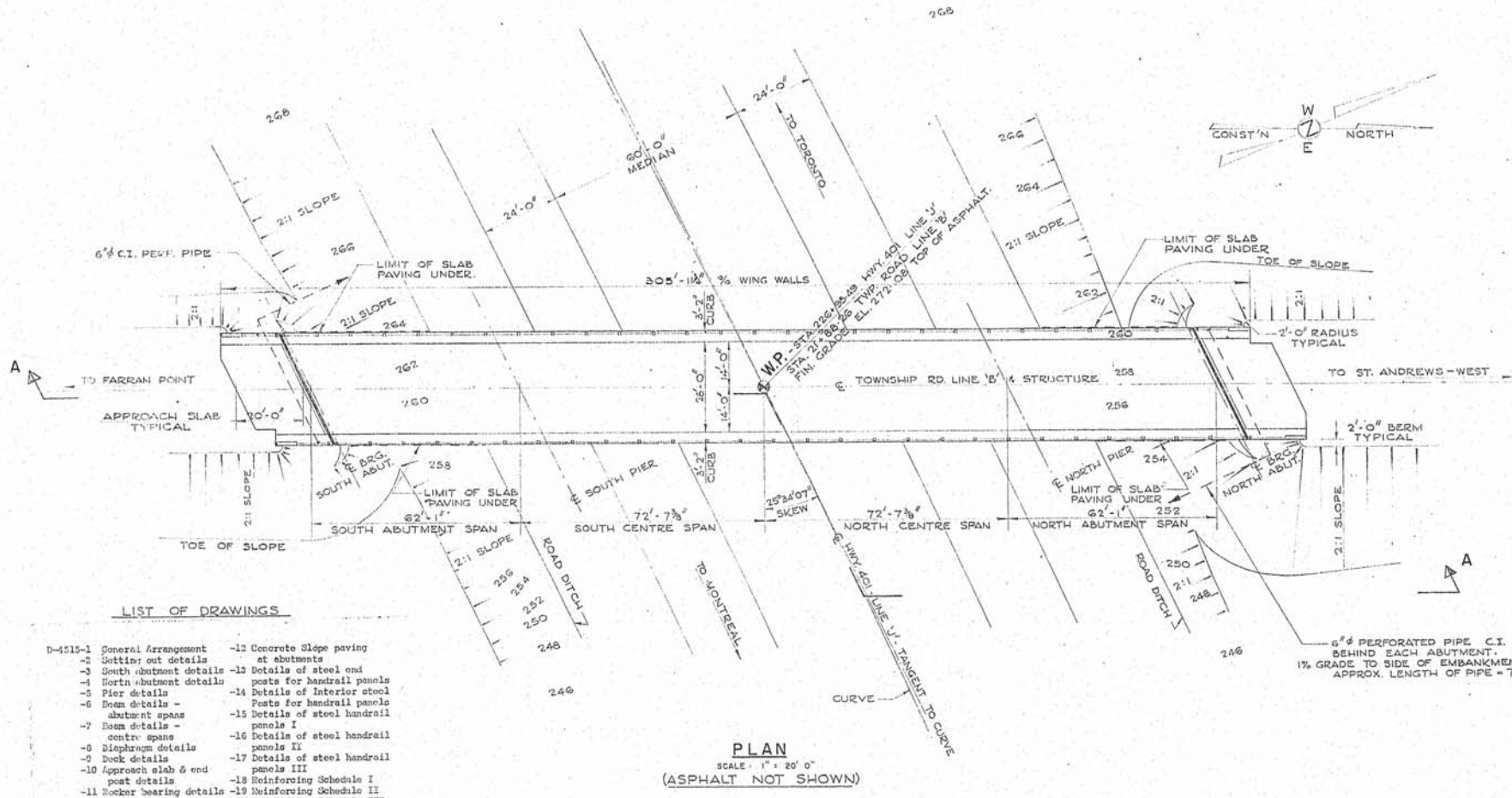
2.0 BASIS OF PAYMENT

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

END OF SECTION

APPENDIX F

Selected 1961 Design Drawings



NOTES

1. Structure to be built in accordance with Form No. 9, latest revision, and the special provisions, extra copies of which may be obtained from the District Engineer.

2. All concrete shall be 3000 p.s.i. at 28 days. Approved admixtures will be added by the Contractor to all concrete as specified by the Materials & Research Section of the D.H.O.

3. Profile data
The complete soil investigation report BA 1016A may be examined at the bridge office, Downsview. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.

4. Clear cover on reinforcing steel:

Footings	- 3"
Abutments	- 3"
Beams & Deck	- 1 1/2"
Handrails	- 1 1/2"
Columns	- 2"

5. All exposed edges to be chamfered 1" x 1" except as noted. All construction joints must be approved by the Bridge Engineer.

W.P. 78-59

E.O. 59257

PROCTOR & REDFERN
CONSULTING ENGINEERS
TORONTO

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

CORNWALL TOWNSHIP
BRIDGE No. 5

THE KING'S HIGHWAY No. 401
CO. STORMONT
TWP. CORNWALL
LOT 21
CON. IV

GENERAL ARRANGEMENT

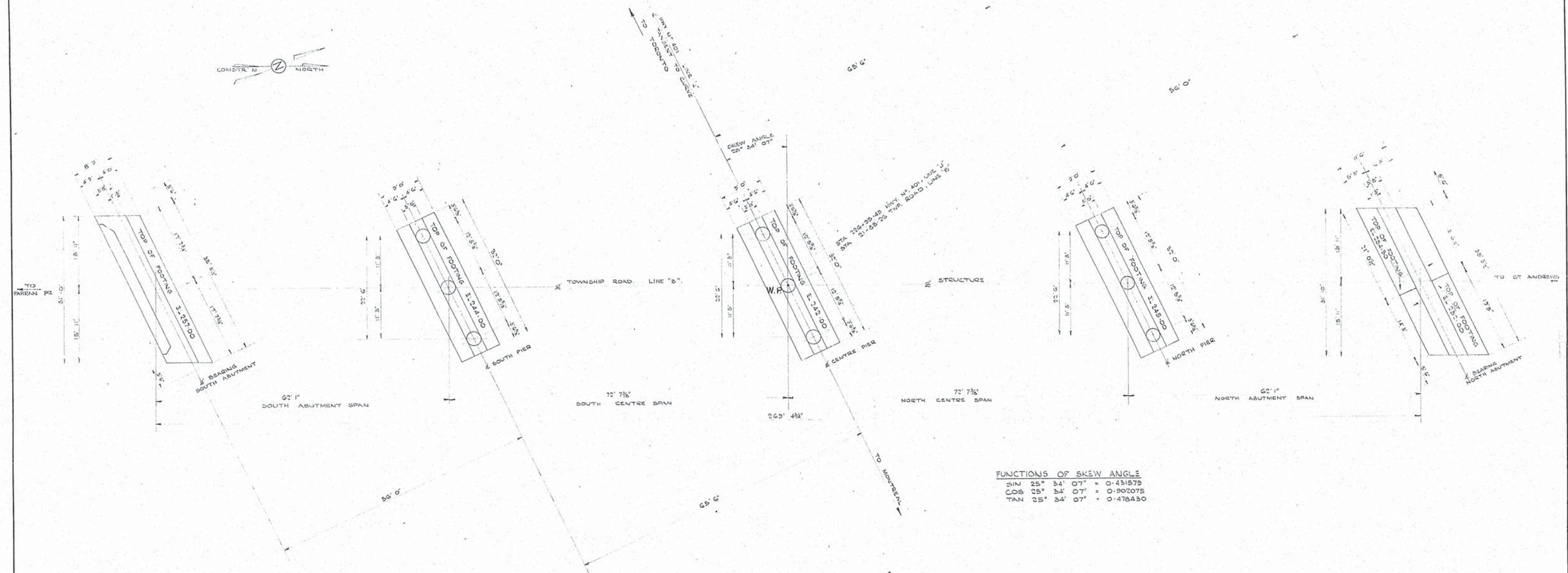
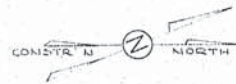
APPROVED: *[Signature]*
DESIGN ENGINEER

DATE: FEB. 1961

REVISIONS:

NO.	DATE	DESCRIPTION
1	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
2	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
3	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
4	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
5	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
6	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
7	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
8	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
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12	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
13	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
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17	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
18	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
19	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.
20	10/1/60	Revised as Constructed. A. Clearances added. D. and L. added.

TWP# 31-179-1-A



FUNCTIONS OF SKEW ANGLE			
SIN	25° 34' 07"	=	0.431579
COS	25° 34' 07"	=	0.902075
TAN	25° 34' 07"	=	0.478430

SETTING OUT PLAN
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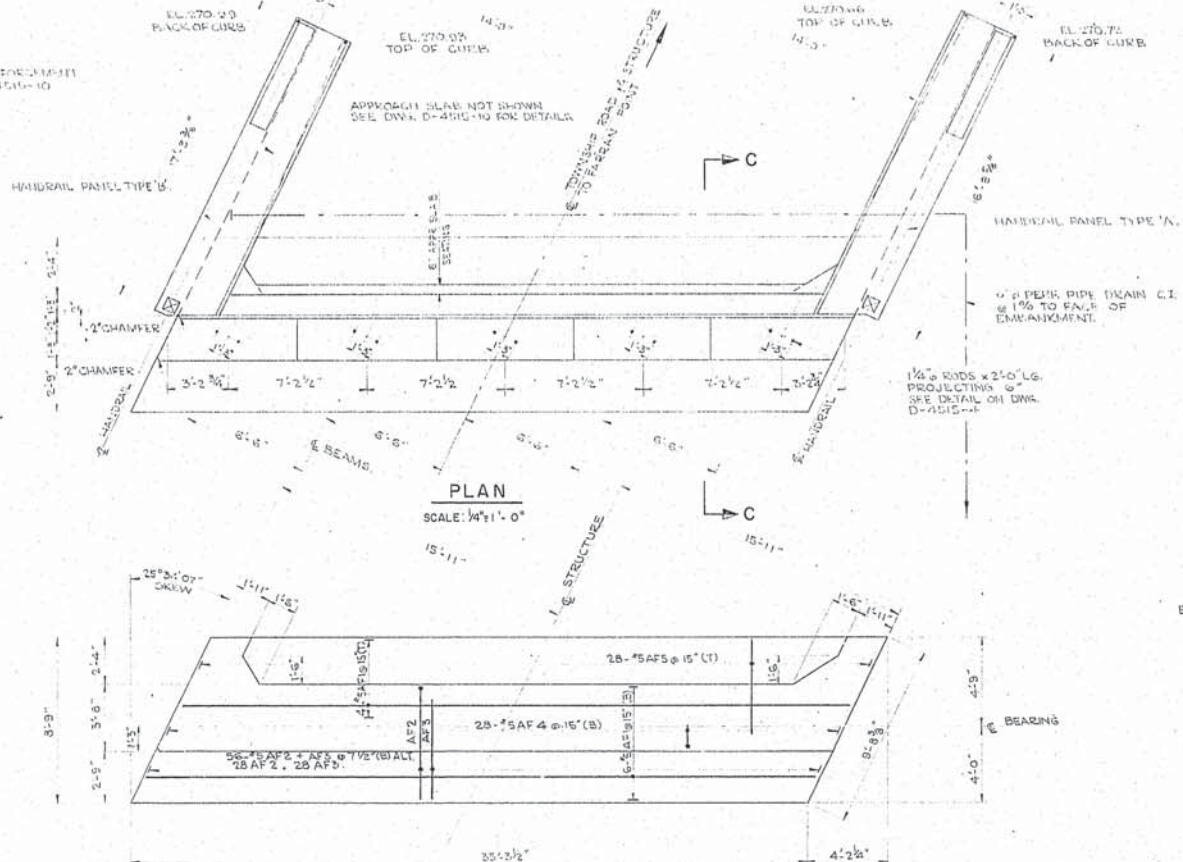


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NO.	DATE	BY	DESCRIPTION	NO.	DATE	BY	DESCRIPTION	NO.	DATE	NO.	DATE
1	1961	W.T.	Revised as Contracted	1	1961	R.E.M.	Check	1	1961	1	1961
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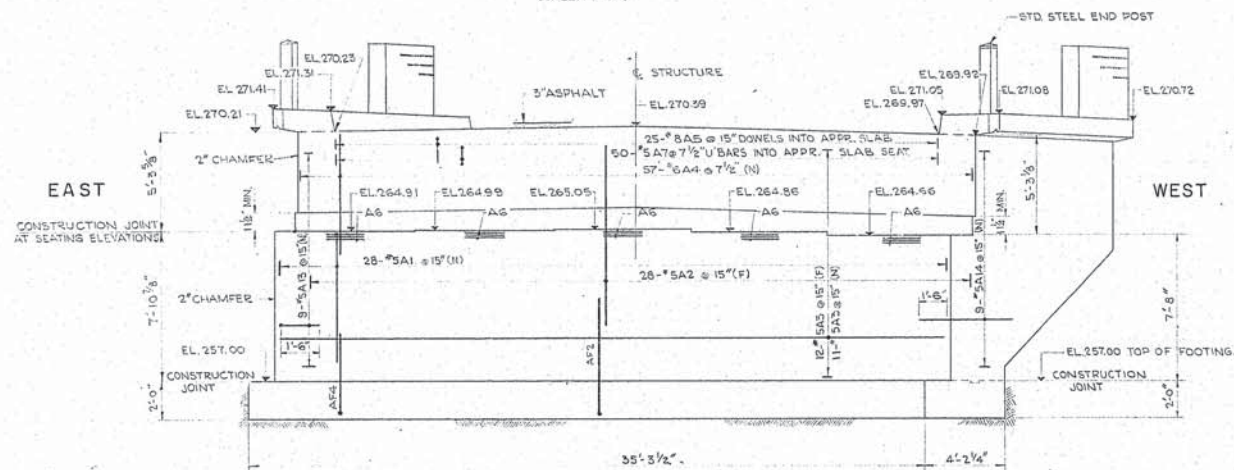
E.O. 59257		WP 78-59	
PROCTOR & REDFERN CONSULTING ENGINEERS TORONTO			
DEPARTMENT OF HIGHWAYS-ONTARIO BRIDGE OFFICE-TORONTO			
CORNWALL TOWNSHIP BRIDGE No 5			
THE KING'S HIGHWAY No. 401		DIST. No. 9	
CO. STORMONT		CON. IV	
TWP. CORNWALL		LOT 21	
SETTING-OUT DETAILS			
APPROVED		DESIGN ENGINEER	
A. E. READ		A. E. READ	
BRIDGE ENGINEER		DESIGN ENGINEER	
DATE FEB. 1961		DATE FEB. 1961	
H20-S16		H20-S16	
B-4515-2		B-4515-2	

TWP#31-179-2-A

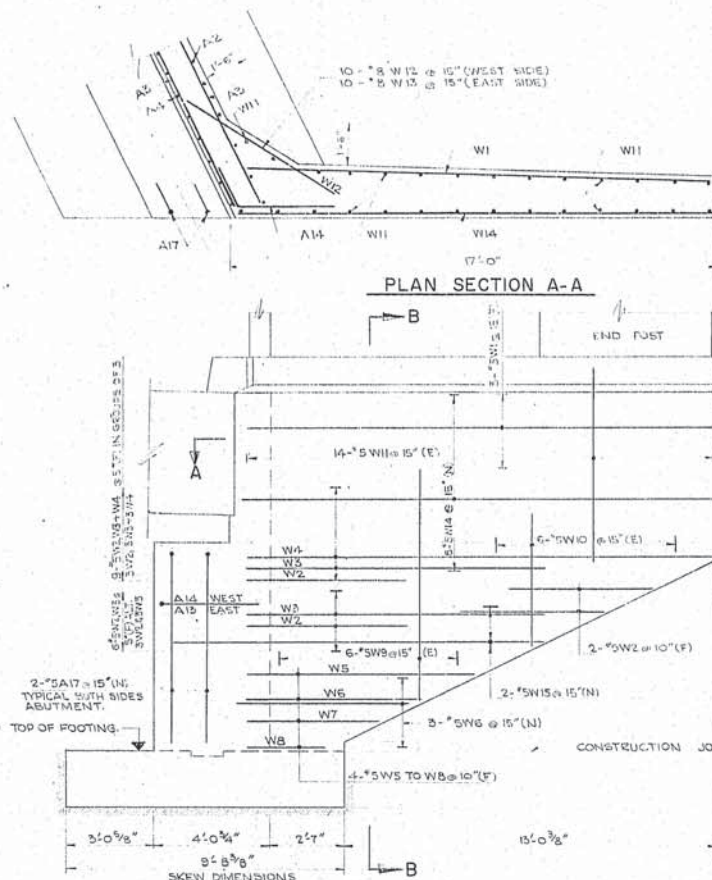
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SEE DWG. D-4515-10



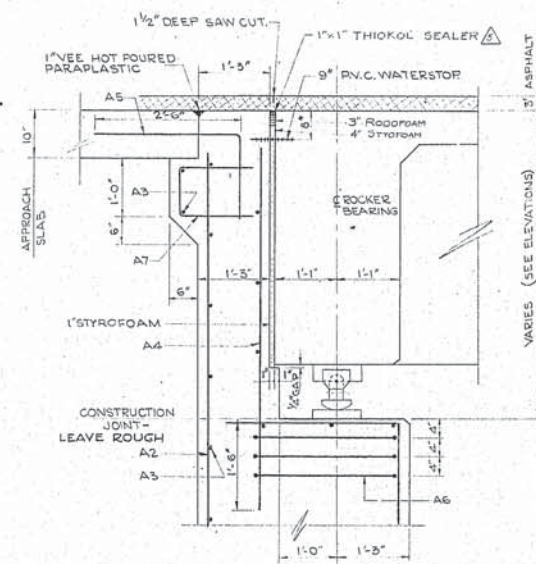
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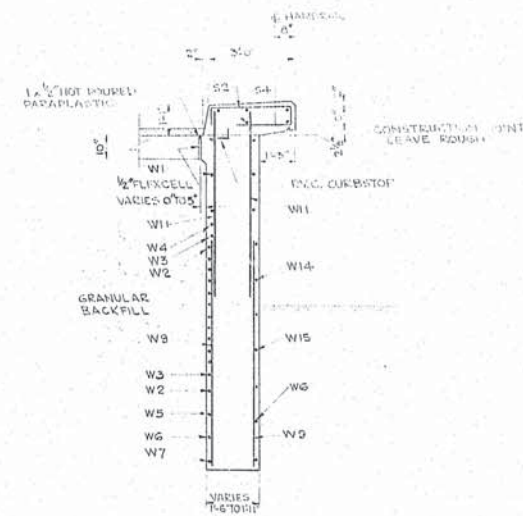
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WING-WALL ELEVATION
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REINFORCEMENT SIMILAR FOR ALL WING WALLS

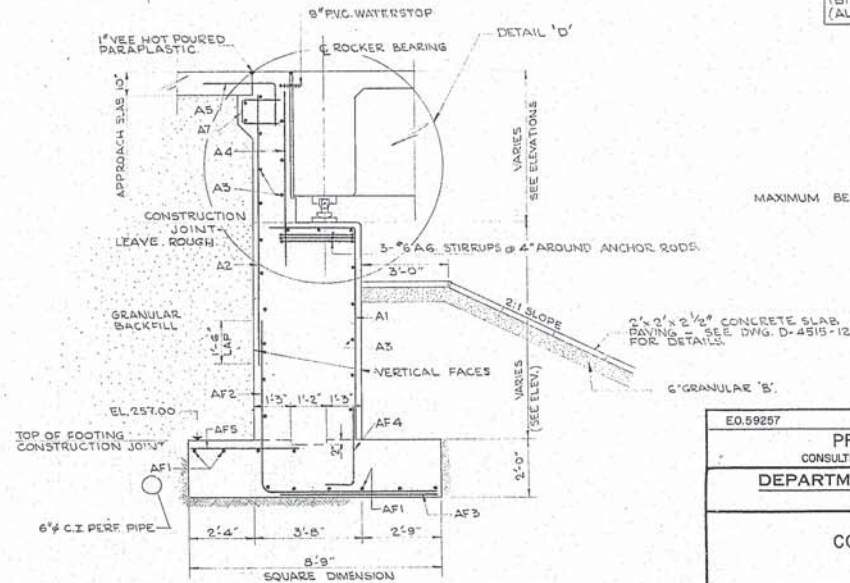


DETAIL 'D'
SCALE: 3/4" = 1'-0"



SECTION B-B
SCALE: 3/8" = 1'-0"

(N) DENOTES NEAR FACE
(F) = FAR "
(E) = EACH "
(T) = TOP "
(B) = BOTTOM "
(ALT) = ALTERNATING



SECTION C-C
SCALE: 3/8" = 1'-0"

MAXIMUM BEARING PRESSURE ON SOIL = 2 TONS/SQ. FT.

W.P. 78-39

E.O. 59257

PROCTOR & REDFERN
CONSULTING ENGINEERS
TORONTO
DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

CORNWALL TOWNSHIP
BRIDGE No 5

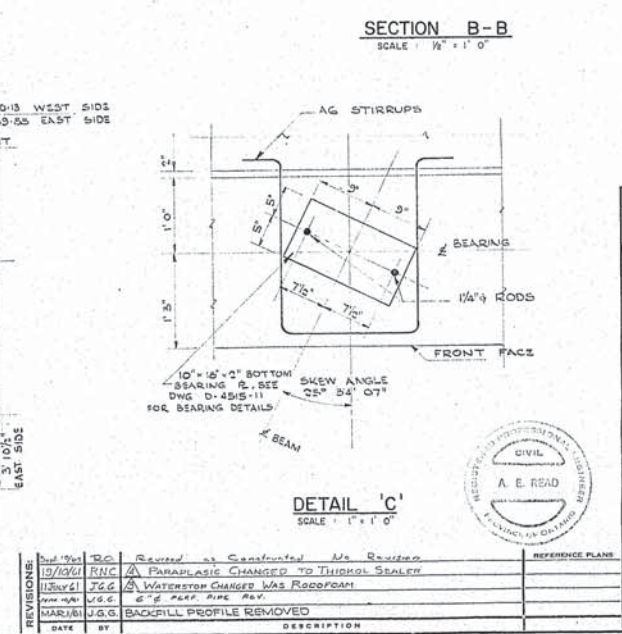
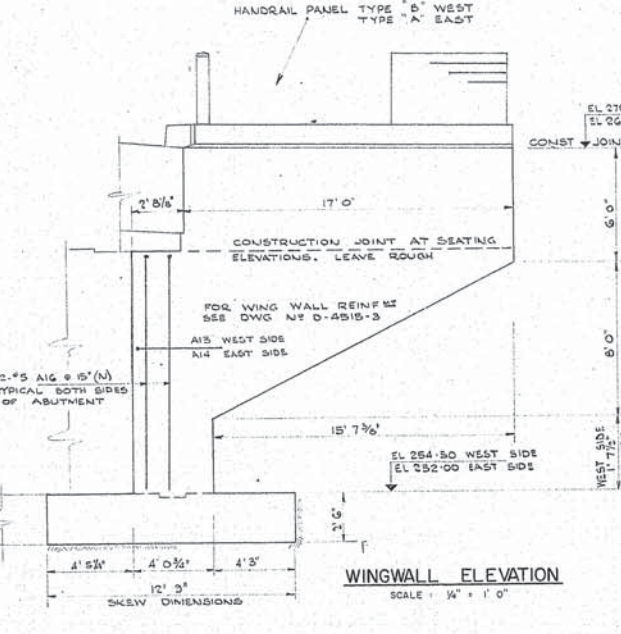
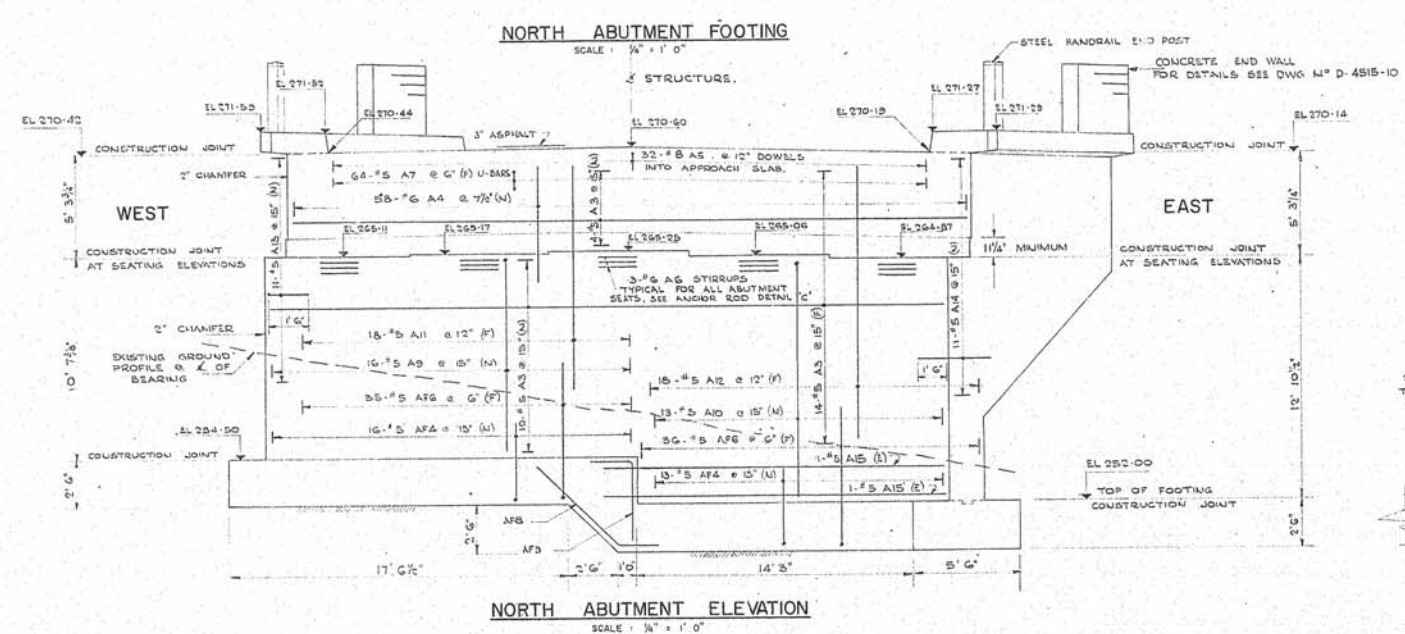
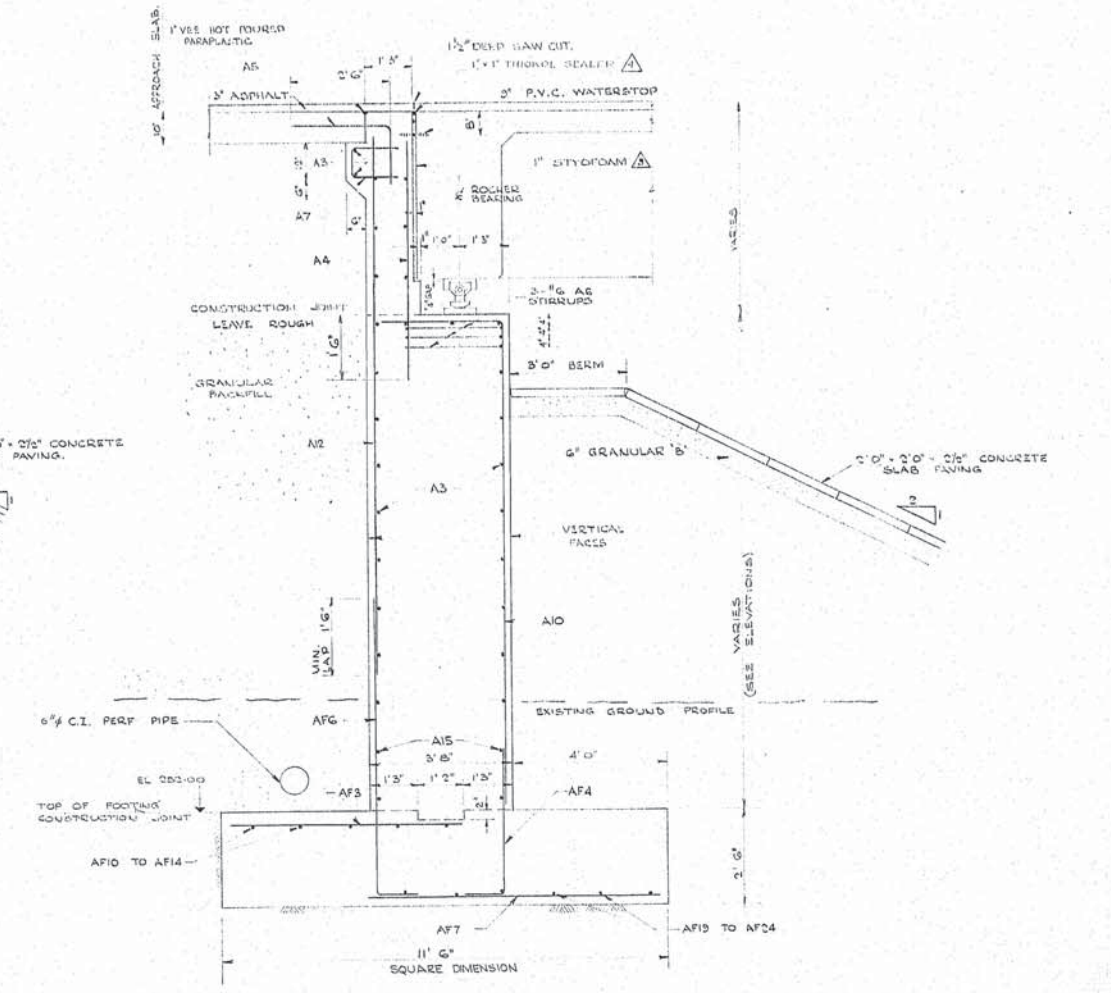
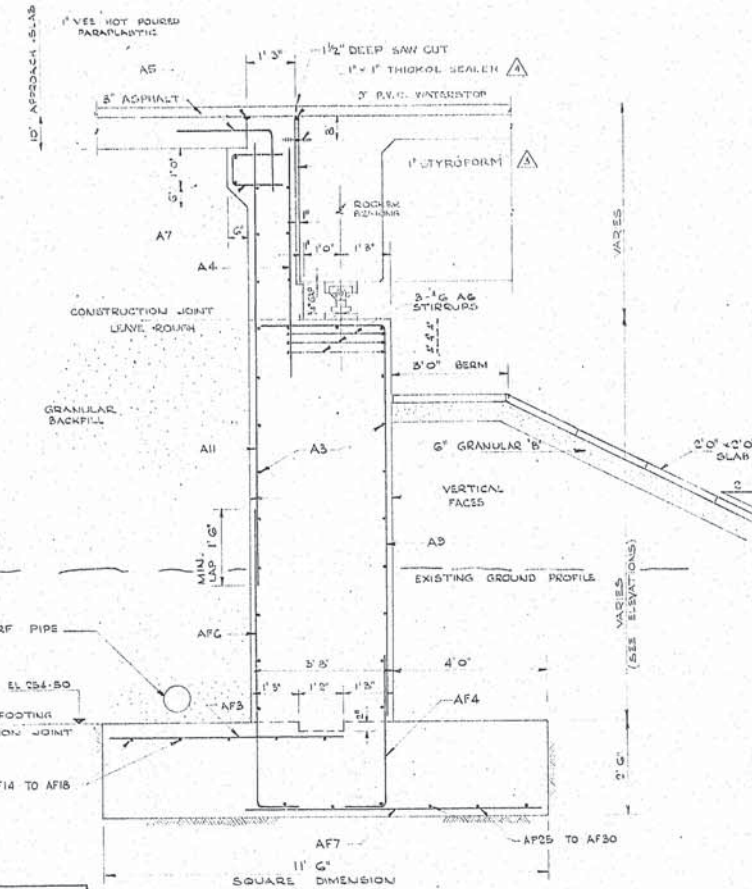
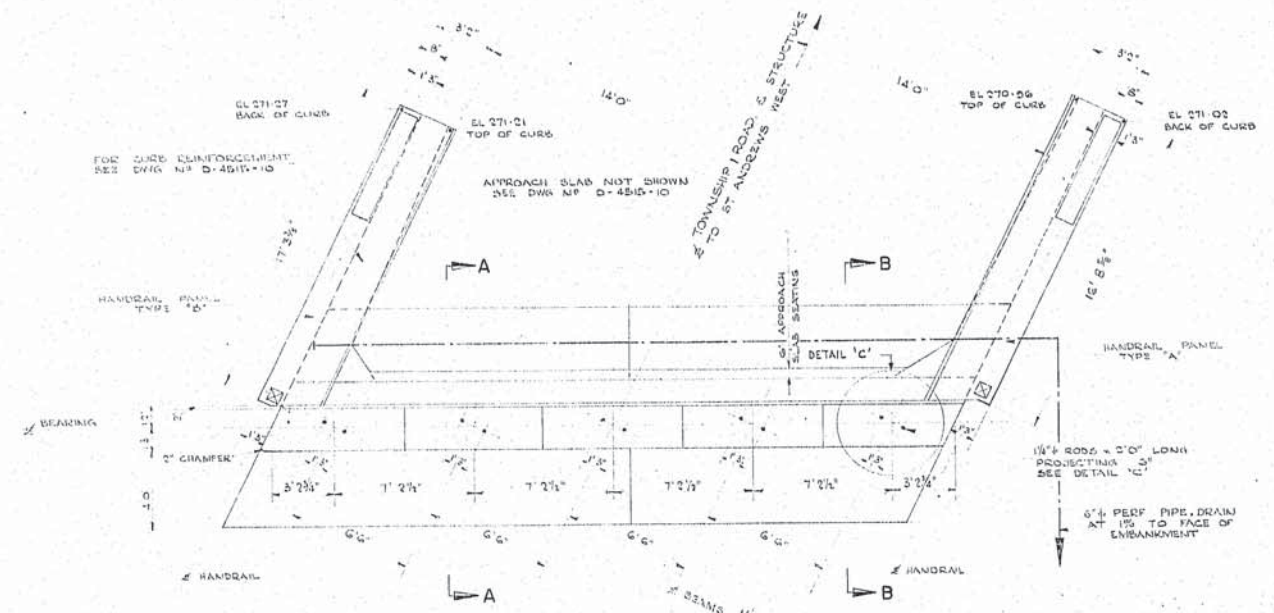
THE KING'S HIGHWAY No. 401
CO. STORMONT
TWP. CORNWALL
LOT 21
CON. IV
DIST. No. 9

SOUTH ABUTMENT DETAILS

APPROVED
DESIGN ENGINEER

REVISIONS	DATE	BY	DESCRIPTION	REFERENCE PLANS	DESIGN	W.T.	CHECK	O.S.Z.	CONTRACT	NUMBERS	LOADING	DRINKS	NUMBER
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2	11/16/61	JLB	ROCKERS CHANGED TO STYROFOAM										
3	11/16/61	JLB	FACT C-C 6" PARAPLASTIC REMOVED										
4	11/16/61	JLB	FACT C-C 6" PARAPLASTIC REMOVED										
5	11/16/61	JLB	FACT C-C 6" PARAPLASTIC REMOVED										
6	11/16/61	JLB	FACT C-C 6" PARAPLASTIC REMOVED										
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9	11/16/61	JLB	FACT C-C 6" PARAPLASTIC REMOVED										
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Twp# 31-179-3-A



WP78-59
E.O. 59257
DWG. NPA-59257-25

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CONSULTING ENGINEERS
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DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

CORNWALL TOWNSHIP
BRIDGE No. 5

THE KING'S HIGHWAY No. 401
CO. STORMONT
TWP. CORNWALL
LOT 21
CON. IV

NORTH ABUTMENT DETAILS

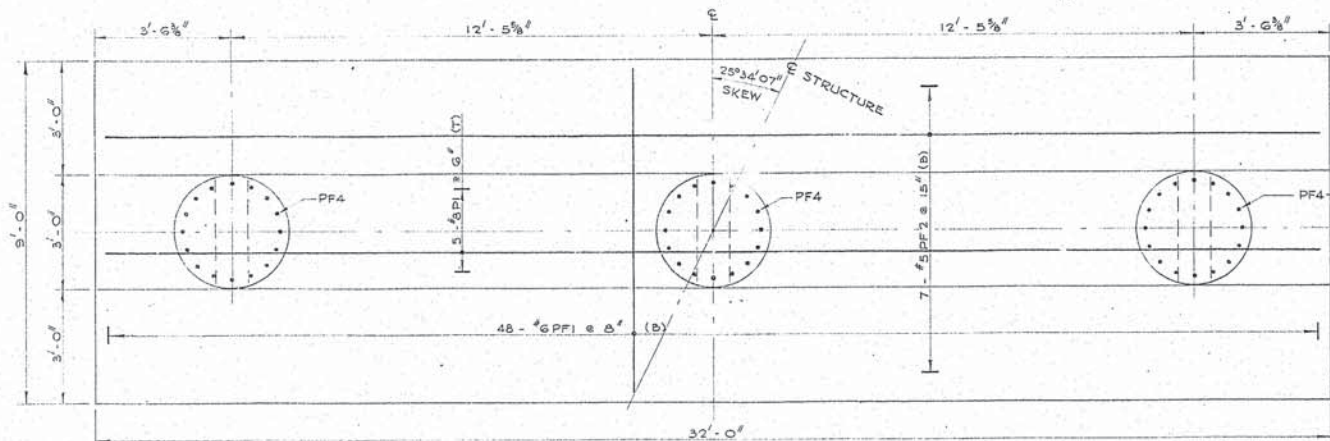
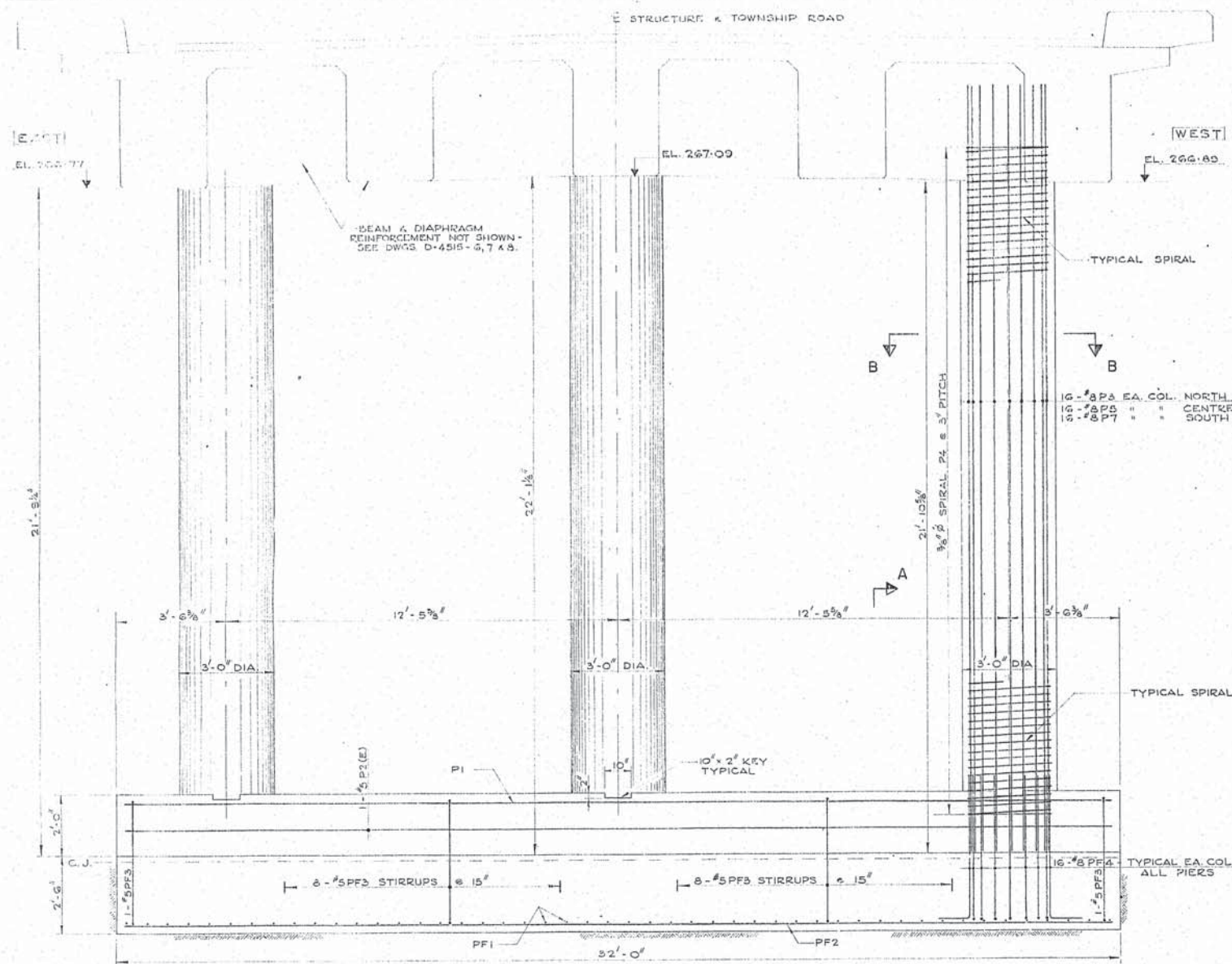
APPROVED
A. E. READ
BRIDGE ENGINEER

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DRAWING R.E.M. CHECK W.T. LOADING
TRACING J.G.G. CHECK H20-S16
DATE FEB. 1961

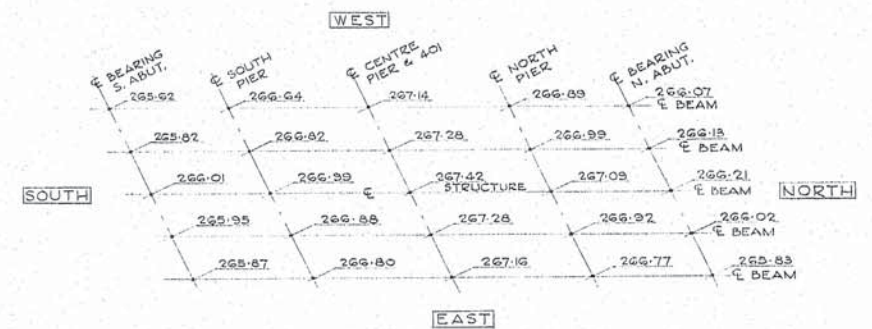
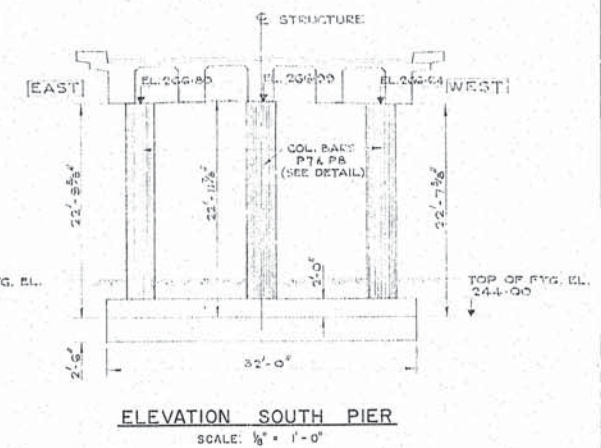
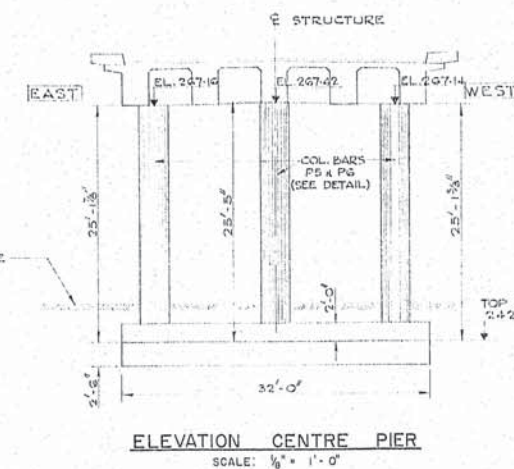
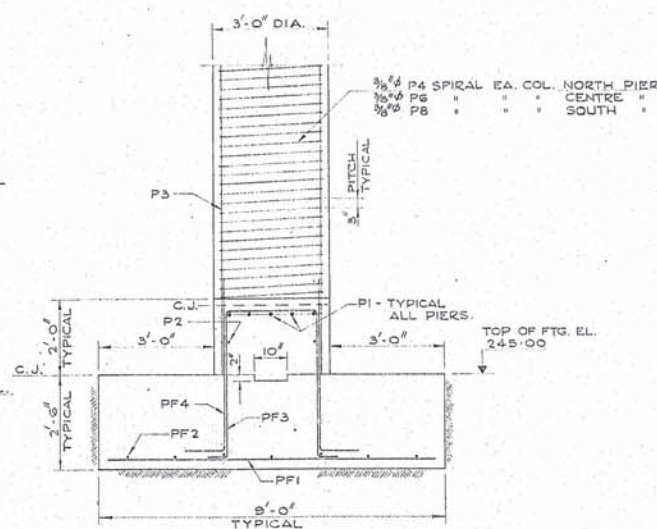
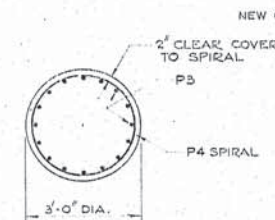
REVISIONS
1. 10/10/61 RNC A PARAPETIS CHANGED TO THURLOUGH SEALER
2. 11/10/61 J.G.G. A WATERSTOP CHANGED WAS RODFOAM
3. 11/10/61 J.G.G. 6" PERP PIPE REMOVED
4. 11/10/61 J.G.G. BACKFILL PROFILE REMOVED

DATE FEB. 1961

BRIDGE NUMBER D-4515-4



NOTE: - REINFORCEMENT FOR ALL
PIER FOOTINGS SIMILAR.



MAXIMUM BEARING PRESSURE ON SOIL = 3 TONS/SQ. FT.

NOTE:

(T)	DENOTES	TOP FACE
(B)	"	BOTTOM "
(E)	"	EACH "
EA. COL.	"	EACH COLUMN
C. J.	"	CONSTRUCTION JOINT.

SCALE: ALL $\frac{1}{2}" = 1' - 0"$
UNLESS OTHERWISE NOTED.

E.O. 59257	PROCTOR & REDFERN CONSULTING ENGINEERS TORONTO	W.P. 78-59 DWG. N° A-59257-26
DEPARTMENT OF HIGHWAYS-ONTARIO- BRIDGE OFFICE-TORONTO		
CORNWALL TOWNSHIP BRIDGE N° 5		
THE KING'S HIGHWAY No. 401 CO. STORMONT TWP. CORNWALL		DIST. N CON. IV
		LOT 21

REVISIONS: DATE BY DESCRIPTION					REFERENCE PLANS	BRIDGE ENGINEER DESIGN W.T. CHECK O.S.Z. CONTRACT NUMBER DRAWING J.N.G. CHECK W.T. TRACING CHECK DATE FEB, 1961				HIGHWAY ENGINEER CHECKED 6-1-67 LOADING H20-516 DRAWING NUMBER D 4515-5			

APPENDIX G

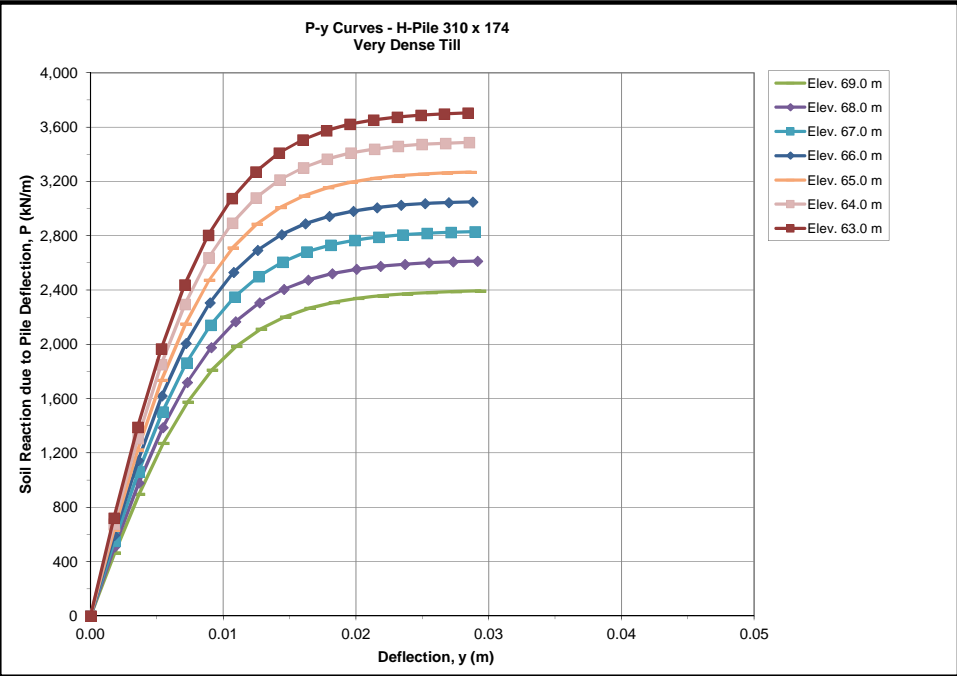
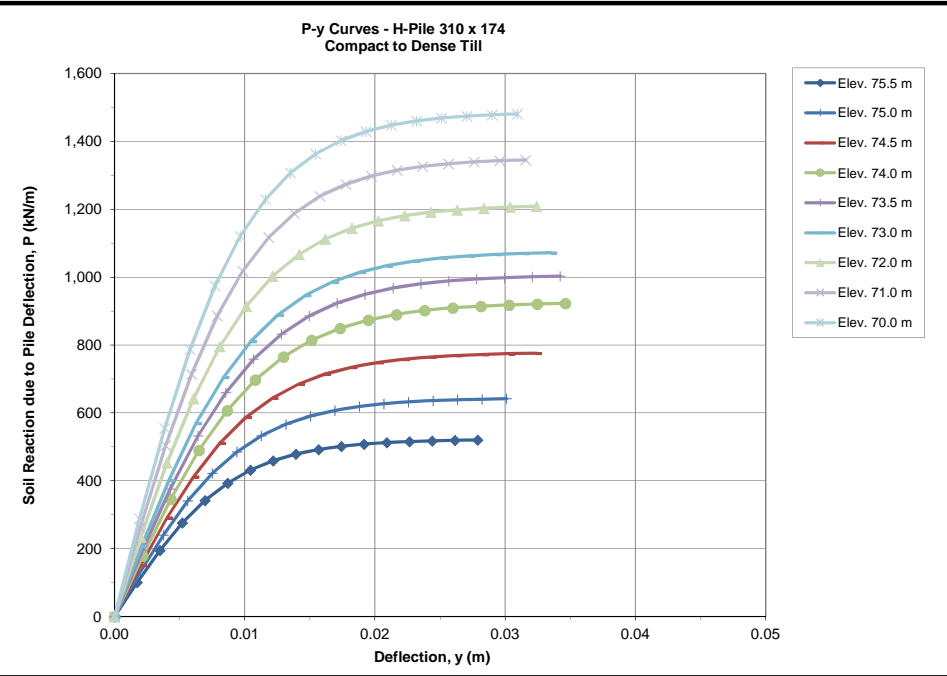
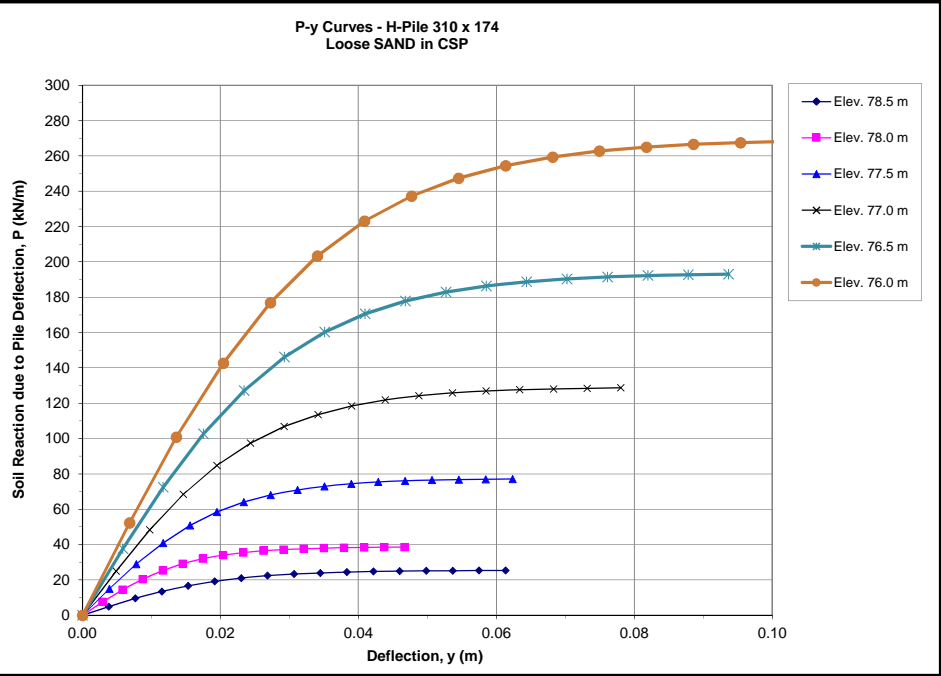
P-y Curves for Deep Foundations

SUMMARY OF P-y CURVES FOR A H-Pile 310x174 - Abutments

Description Depth (z) * Elevation P-y Curves	Loose Sand (CSP)												Compact to Dense Silt and Sand Till																		
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		
	Elev. 78.5 m		Elev. 78.0 m		Elev. 77.5 m		Elev. 77.0 m		Elev. 76.5 m		Elev. 76.0 m		Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.5 m		Elev. 73.0 m		Elev. 72.0 m		Elev. 71.0 m		Elev. 70.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.004	4.921	0.003	7.497	0.004	15.013	0.005	25.041	0.006	37.580	0.007	52.305	0.002	101.207	0.002	124.887	0.002	151.001	0.002	179.550	0.002	195.211	0.002	208.542	0.002	235.205	0.002	261.868	0.002	288.265		
0.008	9.486	0.006	14.451	0.008	28.939	0.010	48.267	0.012	72.438	0.014	100.822	0.003	195.082	0.004	240.726	0.004	291.063	0.004	346.092	0.004	376.280	0.004	401.977	0.004	453.372	0.004	504.766	0.004	555.646		
0.012	13.433	0.009	20.465	0.012	40.983	0.015	68.356	0.018	102.586	0.020	142.783	0.005	276.274	0.006	340.916	0.006	412.202	0.006	490.134	0.006	532.886	0.006	569.279	0.006	642.063	0.006	714.848	0.006	786.904		
0.015	16.647	0.012	25.361	0.016	50.785	0.020	84.706	0.023	127.124	0.027	176.936	0.007	342.357	0.008	422.460	0.008	510.798	0.009	607.370	0.009	660.349	0.008	705.446	0.008	795.640	0.008	885.834	0.008	975.126		
0.019	19.135	0.015	29.152	0.019	58.377	0.024	97.368	0.029	146.126	0.034	203.384	0.009	393.532	0.009	485.609	0.010	587.151	0.011	698.159	0.011	759.057	0.010	810.895	0.010	914.571	0.010	1018.247	0.010	1120.886		
0.023	20.989	0.018	31.976	0.023	64.032	0.029	106.801	0.035	160.282	0.041	223.087	0.010	431.657	0.011	532.654	0.012	644.034	0.013	765.796	0.013	832.593	0.013	889.453	0.012	1003.173	0.012	1116.893	0.012	1229.476		
0.027	22.330	0.020	34.020	0.027	68.125	0.034	113.628	0.041	170.527	0.048	237.347	0.012	459.248	0.013	566.701	0.014	685.199	0.015	814.744	0.015	885.811	0.015	946.306	0.014	1067.295	0.014	1188.283	0.014	1308.062		
0.031	23.281	0.023	35.468	0.031	71.025	0.039	118.465	0.047	177.787	0.055	247.451	0.014	478.799	0.015	590.826	0.016	714.370	0.017	849.429	0.017	923.522	0.017	986.592	0.016	1112.731	0.016	1238.871	0.015	1363.749		
0.035	23.945	0.026	36.479	0.035	73.050	0.044	121.842	0.053	182.855	0.061	254.505	0.016	492.447	0.017	607.668	0.018	734.733	0.019	873.642	0.019	949.847	0.019	1014.715	0.018	1144.450	0.018	1274.185	0.017	1402.623		
0.038	24.403	0.029	37.177	0.039	74.448	0.049	124.175	0.059	186.356	0.068	259.377	0.017	501.875	0.019	619.302	0.020	748.800	0.022	890.369	0.021	968.032	0.021	1034.142	0.020	1166.361	0.020	1298.580	0.019	1429.477		
0.042	24.717	0.032	37.656	0.043	75.408	0.054	125.774	0.064	188.757	0.075	262.719	0.019	508.341	0.021	627.281	0.022	758.447	0.024	901.840	0.024	980.504	0.023	1047.465	0.022	1181.388	0.022	1315.310	0.021	1447.894		
0.046	24.932	0.035	37.983	0.047	76.062	0.059	126.866	0.070	190.395	0.082	264.999	0.021	512.753	0.023	632.725	0.024	765.030	0.026	909.668	0.026	989.015	0.025	1056.557	0.024	1191.642	0.024	1326.727	0.023	1460.461		
0.050	25.078	0.038	38.205	0.051	76.507	0.063	127.609	0.076	191.509	0.089	266.550	0.023	515.754	0.024	636.428	0.026	769.507	0.028	914.991	0.028	994.803	0.027	1062.741	0.026	1198.616	0.026	1334.492	0.025	1469.008		
0.054	25.177	0.041	38.356	0.055	76.809	0.068	128.112	0.082	192.265	0.095	267.602	0.024	517.790	0.026	638.941	0.028	772.545	0.030	918.604	0.030	998.730	0.029	1066.936	0.028	1203.348	0.028	1339.760	0.027	1474.808		
0.058	25.244	0.044	38.458	0.058	77.014	0.073	128.454	0.088	192.778	0.102	268.315	0.026	519.170	0.028	640.643	0.030	774.603	0.032	921.051	0.032	1001.391	0.031	1069.778	0.030	1206.554	0.030	1343.329	0.029	1478.737		
0.061	25.289	0.047	38.528	0.062	77.152	0.078	128.685	0.094	193.124	0.109	268.798	0.028	520.103	0.030	641.795	0.032	775.996	0.035	922.707	0.034	1003.191	0.034	1071.702	0.032	1208.723	0.032	1345.744	0.031	1481.395		

Description Depth (z) * Elevation P-y Curves	Very Dense Sand and Silt Till													
	z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m	
	Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	465.801	0.002	508.401	0.002	551.000	0.002	593.600	0.002	636.199	0.002	678.799	0.002	720.972	
0.004	897.858	0.004	979.971	0.004	1062.084	0.004	1144.197	0.004	1226.310	0.004	1308.422	0.004	1389.714	
0.006	1271.543	0.005	1387.831	0.005	1504.119	0.005	1620.407	0.005	1736.695	0.005	1852.983	0.005	1968.108	
0.007	1575.686	0.007	1719.790	0.007	1863.893	0.007	2007.996	0.007	2152.099	0.007	2296.203	0.007	2438.865	
0.009	1811.218	0.009	1976.861	0.009	2142.505	0.009	2308.149	0.009	2473.792	0.009	2639.436	0.009	2803.423	
0.011	1986.686	0.011	2168.377	0.011	2350.068	0.011	2531.759	0.011	2713.450	0.011	2895.141	0.011	3075.015	
0.013	2113.672	0.013	2306.976	0.013	2500.281	0.013	2693.585	0.013	2886.889	0.012	3080.194	0.012	3271.565	
0.015	2203.655	0.015	2405.189	0.014	2606.723	0.014	2808.256	0.014	3009.790	0.014	3211.324	0.014	3410.842	
0.017	2266.470	0.016	2473.749	0.016	2681.027	0.016	2888.306	0.016	3095.584	0.016	3302.863	0.016	3508.068	
0.018	2309.863	0.018	2521.110	0.018	2732.357	0.018	2943.604	0.018	3154.851	0.018	3366.098	0.018	3575.232	
0.020	2339.622	0.020	2553.591	0.020	2767.559	0.020	2981.528	0.020	3195.496	0.020	3409.465	0.020	3621.294	
0.022	2359.930	0.022	2575.756	0.022	2791.582	0.022	3007.407	0.021	3223.233	0.021	3439.059	0.021	3652.726	
0.024	2373.742	0.024	2590.830	0.024	2807.919	0.023	3025.008	0.023	3242.097	0.023	3459.186	0.023	3674.103	
0.026	2383.113	0.026	2601.059	0.025	2819.004	0.025	3036.950	0.025	3254.896	0.025	3472.842	0.025	3688.608	
0.028	2389.461	0.027	2607.988	0.027	2826.514	0.027	3045.040	0.027	3263.567	0.027	3482.093	0.027	3698.435	
0.029	2393.757	0.029	2612.677	0.029	2831.596	0.029	3050.515	0.029	3269.435	0.029	3488.354	0.028	3705.084	

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap at abutments (Elevation 79.0 m).
Please note the following assumptions:
1. P-y curves have been generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for H-pile 310x174.

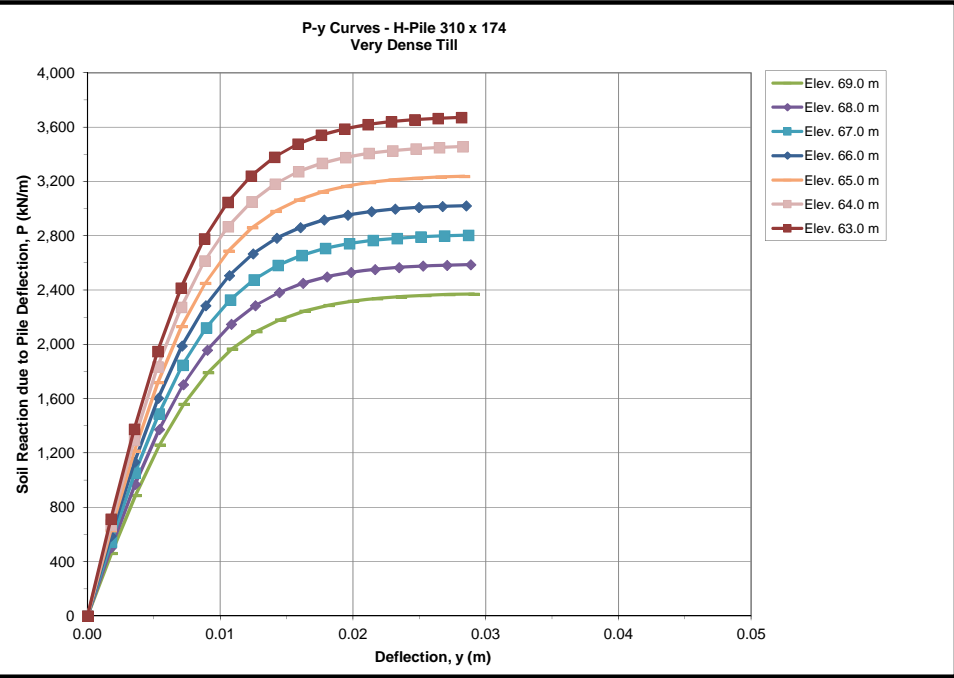
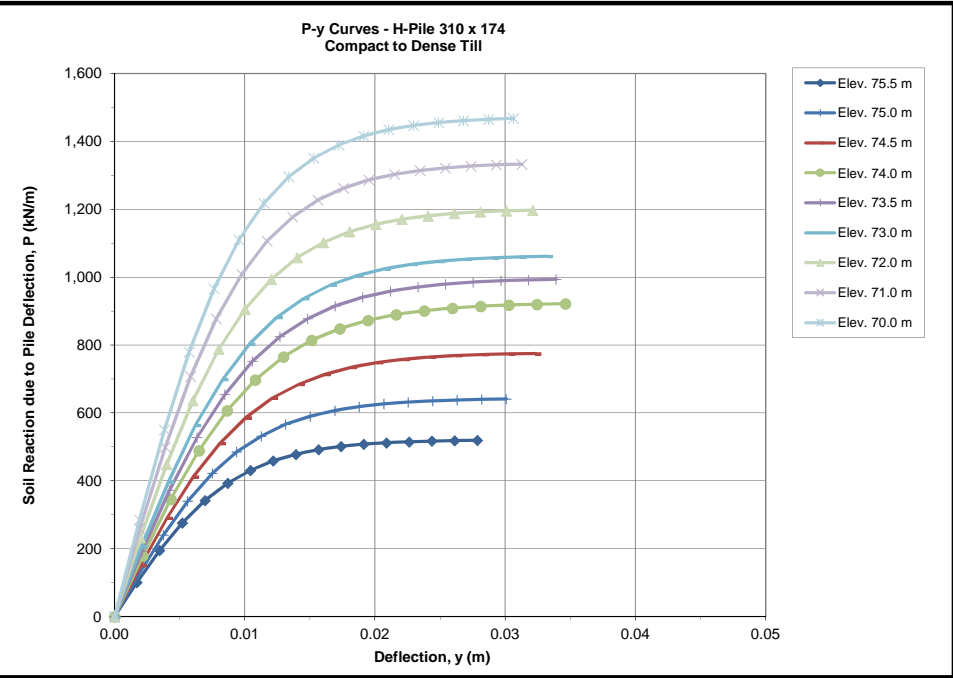
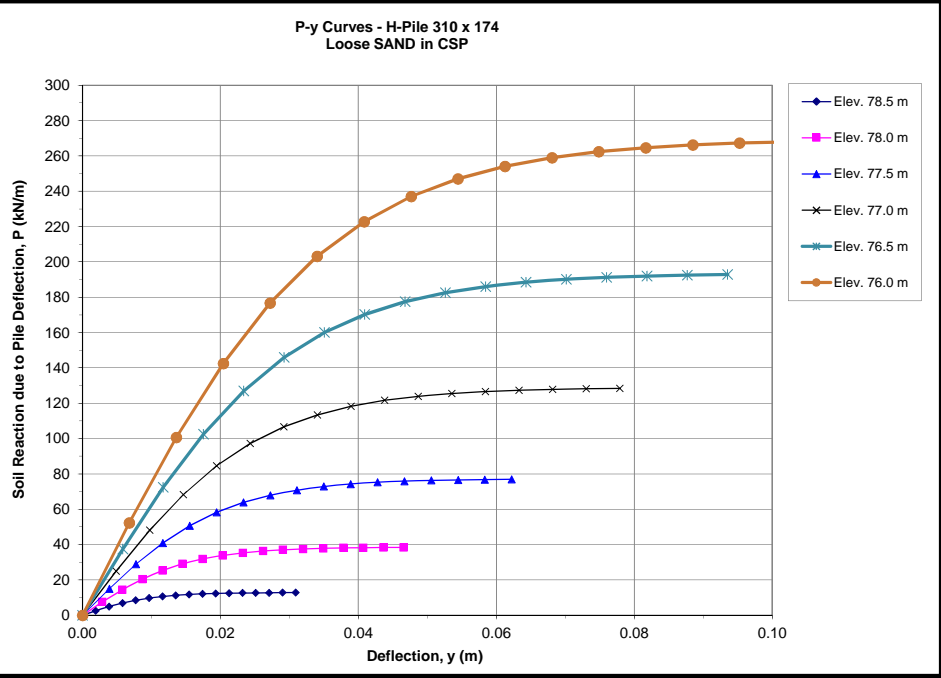


SUMMARY OF P-y CURVES FOR A H-Pile 310x174 - Abutments - Cyclic Loading

Description Depth (z) * Elevation P-y Curves	Loose Sand (CSP)												Compact to Dense Silt and Sand Till																		
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		
	Elev. 78.5 m		Elev. 78.0 m		Elev. 77.5 m		Elev. 77.0 m		Elev. 76.5 m		Elev. 76.0 m		Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.5 m		Elev. 73.0 m		Elev. 72.0 m		Elev. 71.0 m		Elev. 70.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	2.481	0.003	7.474	0.004	14.979	0.005	24.995	0.006	37.523	0.007	52.237	0.002	101.110	0.002	124.781	0.002	150.886	0.002	179.426	0.002	193.420	0.002	206.629	0.002	233.048	0.002	259.466	0.002	285.620	0.002	285.620
0.004	4.783	0.006	14.407	0.008	28.873	0.010	48.180	0.012	72.328	0.014	100.691	0.003	194.896	0.004	240.523	0.004	290.842	0.004	345.853	0.004	372.828	0.004	398.289	0.004	449.212	0.004	500.135	0.004	550.549	0.004	550.549
0.006	6.774	0.009	20.404	0.012	40.890	0.015	68.232	0.018	102.431	0.020	142.598	0.005	276.011	0.006	340.627	0.006	411.889	0.006	489.796	0.006	527.998	0.006	564.056	0.006	636.173	0.006	708.289	0.006	779.685	0.006	779.685
0.008	8.394	0.012	25.284	0.016	50.670	0.019	84.553	0.023	126.932	0.027	176.706	0.007	342.030	0.008	422.103	0.008	510.410	0.009	606.951	0.008	654.291	0.008	698.974	0.008	788.340	0.008	877.707	0.008	966.180	0.008	966.180
0.010	9.649	0.015	29.063	0.019	58.244	0.024	97.192	0.029	145.905	0.034	203.120	0.009	393.156	0.009	485.198	0.010	586.705	0.011	697.677	0.011	752.093	0.010	803.455	0.010	906.180	0.010	1008.905	0.010	1110.603	0.010	1110.603
0.012	10.583	0.017	31.879	0.023	63.887	0.029	106.608	0.035	160.040	0.041	222.798	0.010	431.245	0.011	532.203	0.012	643.544	0.013	765.267	0.013	824.955	0.012	881.293	0.012	993.970	0.012	1106.647	0.012	1218.197	0.012	1218.197
0.014	11.260	0.020	33.917	0.027	67.971	0.034	113.422	0.041	170.270	0.048	237.039	0.012	458.809	0.013	566.221	0.014	684.679	0.015	814.182	0.015	877.685	0.015	937.624	0.014	1057.503	0.014	1177.382	0.013	1296.062	0.013	1296.062
0.015	11.739	0.023	35.360	0.031	70.864	0.039	118.250	0.047	177.519	0.054	247.130	0.014	478.342	0.015	590.326	0.016	713.827	0.017	848.843	0.017	915.049	0.017	977.541	0.016	1102.523	0.016	1227.505	0.015	1351.238	0.015	1351.238
0.017	12.074	0.026	36.368	0.035	72.884	0.044	121.621	0.053	182.579	0.061	254.174	0.016	491.977	0.017	607.153	0.018	734.174	0.019	873.040	0.019	941.133	0.019	1005.405	0.018	1133.950	0.018	1262.495	0.017	1389.755	0.017	1389.755
0.019	12.305	0.029	37.065	0.039	74.280	0.049	123.950	0.058	186.074	0.068	259.041	0.017	501.396	0.019	618.778	0.020	748.231	0.022	889.754	0.021	959.151	0.021	1024.654	0.020	1155.660	0.020	1286.666	0.019	1416.362	0.019	1416.362
0.021	12.463	0.032	37.542	0.043	75.237	0.054	125.546	0.064	188.472	0.075	262.378	0.019	507.856	0.021	626.750	0.022	757.870	0.024	901.218	0.023	971.509	0.023	1037.856	0.022	1170.549	0.022	1303.243	0.021	1434.610	0.021	1434.610
0.023	12.572	0.035	37.868	0.047	75.890	0.058	126.636	0.070	190.108	0.082	264.655	0.021	512.264	0.023	632.190	0.024	764.449	0.026	909.040	0.025	979.941	0.025	1046.864	0.024	1180.710	0.023	1314.555	0.023	1447.063	0.023	1447.063
0.025	12.645	0.038	38.090	0.051	76.334	0.063	127.377	0.076	191.220	0.088	266.204	0.023	515.262	0.024	635.890	0.026	768.923	0.028	914.360	0.028	985.676	0.027	1052.991	0.026	1187.620	0.025	1322.249	0.025	1455.531	0.025	1455.531
0.027	12.695	0.041	38.240	0.054	76.635	0.068	127.880	0.082	191.975	0.095	267.255	0.024	517.296	0.026	638.400	0.028	771.958	0.030	917.970	0.030	989.568	0.029	1057.148	0.028	1192.308	0.027	1327.469	0.027	1461.278	0.027	1461.278
0.029	12.729	0.044	38.342	0.058	76.839	0.073	128.221	0.088	192.487	0.102	267.967	0.026	518.674	0.028	640.101	0.030	774.015	0.032	920.415	0.032	992.204	0.031	1059.964	0.030	1195.484	0.029	1331.005	0.029	1465.170	0.029	1465.170
0.031	12.752	0.047	38.411	0.062	76.977	0.078	128.451	0.094	192.833	0.109	268.449	0.028	519.607	0.030	641.252	0.032	775.406	0.035	922.070	0.034	993.988	0.033	1061.870	0.032	1197.634	0.031	1333.398	0.031	1467.805	0.031	1467.805

Description Depth (z) * Elevation P-y Curves	Very Dense Sand and Silt Till													
	z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m	
	Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	461.528	0.002	503.737	0.002	545.945	0.002	588.154	0.002	630.363	0.002	672.571	0.002	714.358	
0.004	889.621	0.004	970.980	0.004	1052.340	0.004	1133.699	0.004	1215.059	0.004	1296.419	0.004	1376.965	
0.005	1259.878	0.005	1375.099	0.005	1490.320	0.005	1605.541	0.005	1720.762	0.005	1835.983	0.005	1950.052	
0.007	1561.230	0.007	1704.012	0.007	1846.793	0.007	1989.574	0.007	2132.355	0.007	2275.136	0.007	2416.490	
0.009	1794.601	0.009	1958.725	0.009	2122.849	0.009	2286.973	0.009	2451.097	0.009	2615.221	0.009	2777.704	
0.011	1968.460	0.011	2148.484	0.011	2328.508	0.011	2508.532	0.011	2688.556	0.011	2868.580	0.011	3046.804	
0.013	2094.281	0.013	2285.811	0.013	2477.342	0.012	2668.873	0.012	2860.404	0.012	3051.935	0.012	3241.551	
0.015	2183.438	0.014	2383.123	0.014	2582.808	0.014	2782.493	0.014	2982.177	0.014	3181.862	0.014	3379.550	
0.016	2245.677	0.016	2451.054	0.016	2656.431	0.016	2861.807	0.016	3067.184	0.016	3272.561	0.016	3475.884	
0.018	2288.672	0.018	2497.981	0.018	2707.289	0.018	2916.598	0.018	3125.907	0.018	3335.216	0.018	3542.432	
0.020	2318.158	0.020	2530.164	0.020	2742.169	0.020	2954.174	0.020	3166.180	0.019	3378.185	0.019	3588.071	
0.022	2338.280	0.022	2552.125	0.022	2765.971	0.021	2979.817	0.021	3193.662	0.021	3407.508	0.021	3619.215	
0.024	2351.964	0.023	2567.061	0.023	2782.158	0.023	2997.256	0.023	3212.353	0.023	3427.450	0.023	3640.396	
0.025	2361.249	0.025	2577.196	0.025	2793.142	0.025	3009.088	0.025	3225.035	0.025	3440.981	0.025	3654.768	
0.027	2367.540	0.027	2584.061	0.027	2800.583	0.027	3017.104	0.027	3233.626	0.027	3450.148	0.026	3664.504	
0.029	2371.796	0.029	2588.707	0.029	2805.618	0.029	3022.529	0.028	3239.440	0.028	3456.351	0.028	3671.093	

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap at abutments (Elevation 79.0 m).
Please note the following assumptions:
1. P-y curves have been generated for vertical piles (i.e. no inclination)
2. Cyclic loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for H-pile 310x174.

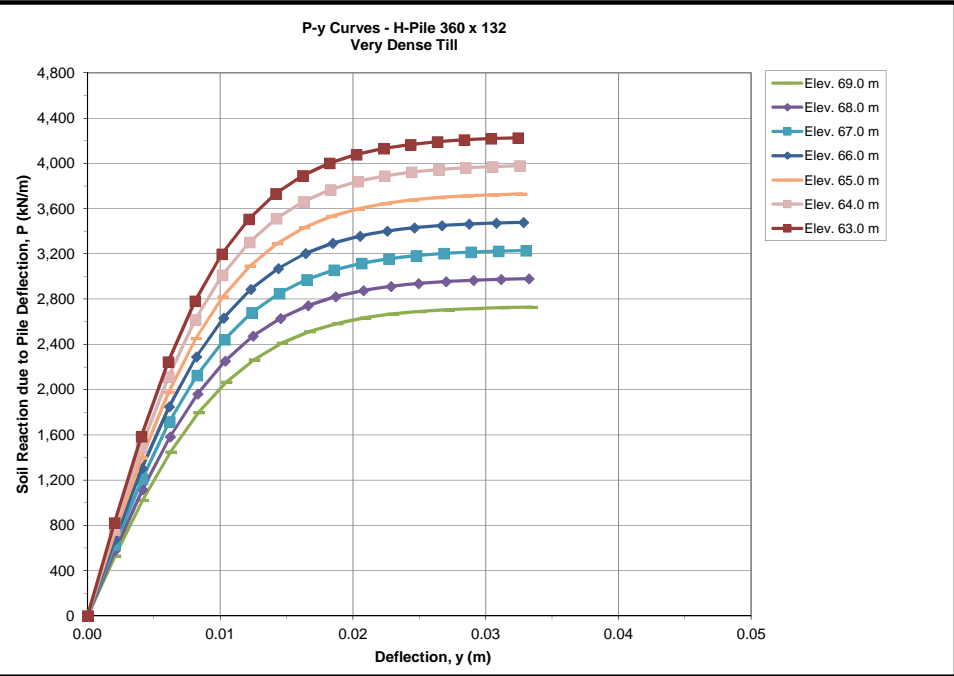
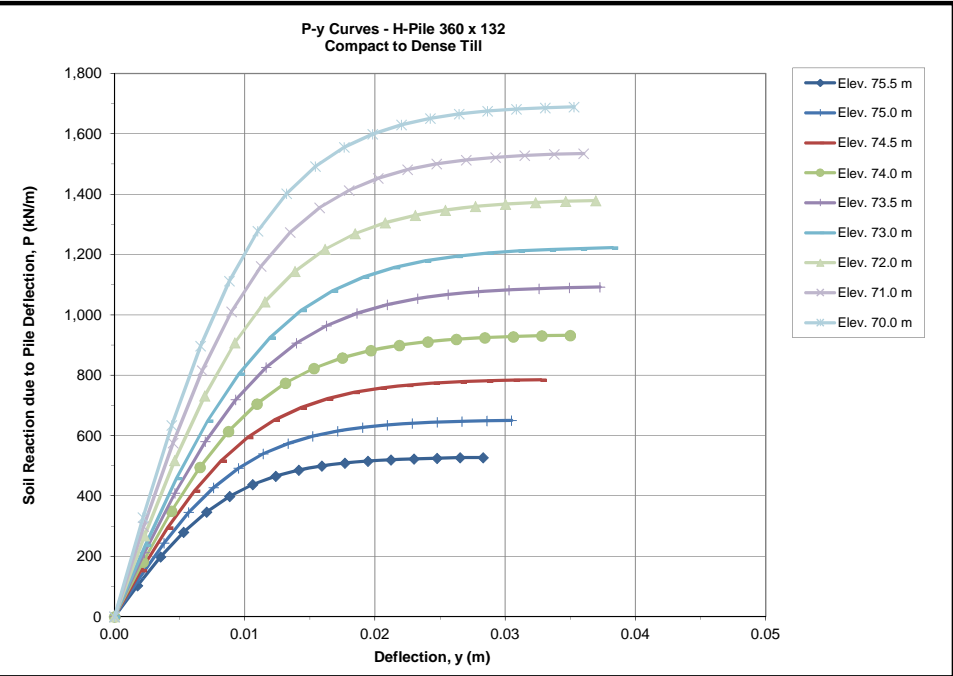
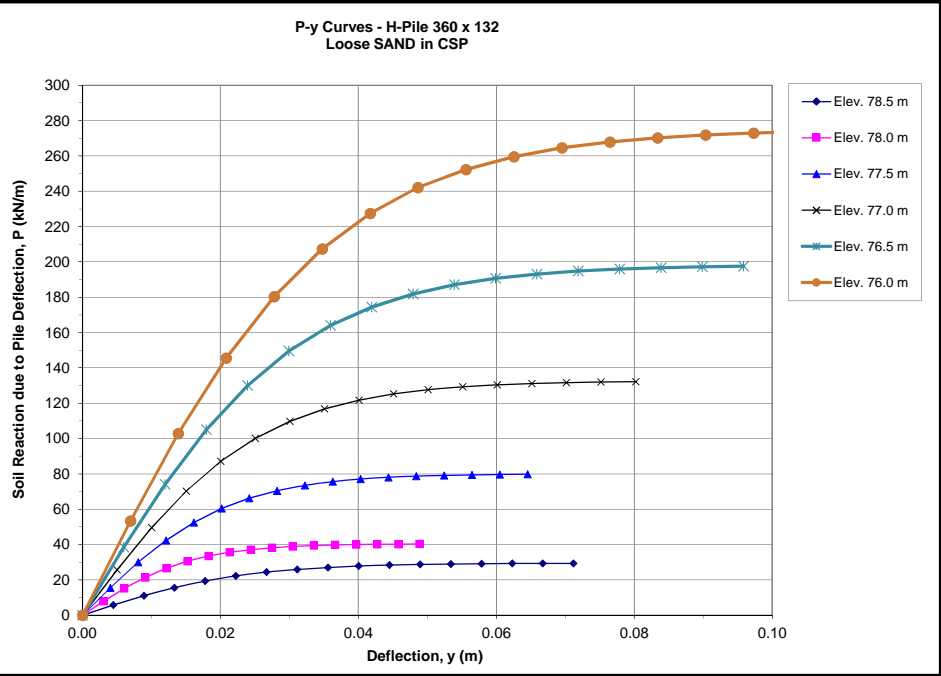


SUMMARY OF P-y CURVES FOR A H-Pile 360x132 - Abutments

Description Depth (z) * Elevation P-y Curves	Loose Sand (CSP)												Compact to Dense Silt and Sand Till																		
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		
	Elev. 78.5 m		Elev. 78.0 m		Elev. 77.5 m		Elev. 77.0 m		Elev. 76.5 m		Elev. 76.0 m		Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.5 m		Elev. 73.0 m		Elev. 72.0 m		Elev. 71.0 m		Elev. 70.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.004	5.712	0.003	7.845	0.004	15.535	0.005	25.737	0.006	38.450	0.007	53.346	0.002	102.689	0.002	126.508	0.002	152.761	0.002	181.449	0.002	212.571	0.002	237.879	0.002	268.292	0.002	298.706	0.002	328.816	0.002	328.816
0.009	11.009	0.006	15.122	0.008	29.945	0.010	49.609	0.012	74.115	0.014	102.827	0.004	197.938	0.004	243.851	0.004	294.456	0.004	349.753	0.005	409.742	0.005	458.524	0.005	517.149	0.005	575.773	0.004	633.811	0.004	633.811
0.013	15.591	0.009	21.415	0.012	42.408	0.015	70.256	0.018	104.961	0.021	145.623	0.005	280.319	0.006	345.340	0.006	417.007	0.007	495.318	0.007	580.275	0.007	649.361	0.007	732.384	0.007	815.407	0.007	897.600	0.007	897.600
0.018	19.321	0.012	26.538	0.016	52.551	0.020	87.061	0.024	130.067	0.028	180.455	0.007	347.369	0.008	427.943	0.008	516.752	0.009	613.795	0.009	719.072	0.010	804.683	0.009	907.565	0.009	1010.446	0.009	1112.299	0.009	1112.299
0.022	22.209	0.015	30.505	0.020	60.406	0.025	100.074	0.030	149.509	0.035	207.430	0.009	399.293	0.010	491.911	0.010	593.995	0.011	705.544	0.012	826.558	0.012	924.966	0.012	1043.226	0.011	1161.487	0.011	1278.564	0.011	1278.564
0.027	24.360	0.018	33.460	0.024	66.259	0.030	109.770	0.036	163.993	0.042	227.525	0.011	437.976	0.011	539.567	0.012	651.540	0.013	773.896	0.014	906.634	0.014	1014.575	0.014	1144.293	0.014	1274.010	0.013	1402.430	0.013	1402.430
0.031	25.917	0.021	35.599	0.028	70.494	0.035	116.786	0.042	174.475	0.049	242.068	0.012	465.971	0.013	574.055	0.014	693.186	0.015	823.362	0.016	964.584	0.017	1079.425	0.016	1217.434	0.016	1355.443	0.015	1492.071	0.015	1492.071
0.036	27.021	0.024	37.114	0.032	73.495	0.040	121.758	0.048	181.903	0.056	252.373	0.014	485.808	0.015	598.494	0.016	722.696	0.018	858.414	0.019	1005.648	0.019	1125.378	0.018	1269.262	0.018	1413.146	0.018	1555.592	0.018	1555.592
0.040	27.791	0.028	38.172	0.036	75.590	0.045	125.228	0.054	187.088	0.063	259.567	0.016	499.656	0.017	615.554	0.018	743.297	0.020	882.883	0.021	1034.315	0.022	1157.457	0.021	1305.443	0.020	1453.428	0.020	1599.934	0.020	1599.934
0.044	28.323	0.031	38.903	0.040	77.037	0.050	127.626	0.060	190.670	0.070	264.537	0.018	509.222	0.019	627.339	0.020	757.527	0.022	899.787	0.023	1054.117	0.024	1179.618	0.023	1330.436	0.023	1481.255	0.022	1630.565	0.022	1630.565
0.049	28.688	0.034	39.404	0.044	78.029	0.055	129.270	0.066	193.126	0.076	267.945	0.019	515.783	0.021	635.422	0.023	767.287	0.024	911.379	0.026	1067.698	0.026	1194.815	0.025	1347.577	0.025	1500.339	0.024	1651.573	0.024	1651.573
0.053	28.937	0.037	39.746	0.048	78.707	0.060	130.392	0.072	194.803	0.083	270.271	0.021	520.260	0.023	640.937	0.025	773.947	0.026	919.290	0.028	1076.965	0.029	1205.186	0.028	1359.274	0.027	1513.362	0.026	1665.908	0.026	1665.908
0.058	29.106	0.040	39.979	0.052	79.167	0.065	131.155	0.078	195.943	0.090	271.853	0.023	523.304	0.025	644.688	0.027	778.477	0.028	924.670	0.030	1083.268	0.031	1212.239	0.030	1367.229	0.029	1522.218	0.029	1675.658	0.029	1675.658
0.062	29.221	0.043	40.137	0.056	79.480	0.070	131.673	0.084	196.716	0.097	272.926	0.025	525.370	0.027	647.233	0.029	781.550	0.031	928.320	0.033	1087.545	0.033	1217.025	0.032	1372.626	0.032	1528.228	0.031	1682.273	0.031	1682.273
0.067	29.299	0.046	40.244	0.061	79.692	0.075	132.024	0.090	197.240	0.104	273.653	0.027	526.770	0.029	648.957	0.031	783.632	0.033	930.793	0.035	1090.442	0.036	1220.267	0.035	1376.283	0.034	1532.299	0.033	1686.755	0.033	1686.755
0.071	29.352	0.049	40.316	0.065	79.835	0.080	132.261	0.096	197.595	0.111	274.145	0.028	527.717	0.031	650.124	0.033	785.041	0.035	932.467	0.037	1092.403	0.038	1222.461	0.037	1378.758	0.036	1535.054	0.035	1689.787	0.035	1689.787

Description Depth (z) * Elevation P-y Curves	Very Dense Sand and Silt Till													
	z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m	
	Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	531.327	0.002	579.919	0.002	628.511	0.002	677.103	0.002	725.695	0.002	774.287	0.002	822.393	
0.004	1024.162	0.004	1117.826	0.004	1211.490	0.004	1305.154	0.004	1398.818	0.004	1492.482	0.004	1585.209	
0.006	1450.415	0.006	1583.061	0.006	1715.708	0.006	1848.354	0.006	1981.001	0.006	2113.648	0.006	2244.968	
0.008	1797.342	0.008	1961.717	0.008	2126.092	0.008	2290.466	0.008	2454.841	0.008	2619.216	0.008	2781.947	
0.010	2066.007	0.010	2254.952	0.010	2443.897	0.010	2632.842	0.010	2821.788	0.010	3010.733	0.010	3197.788	
0.013	2266.159	0.012	2473.409	0.012	2680.659	0.012	2887.909	0.012	3095.159	0.012	3302.408	0.012	3507.586	
0.015	2411.008	0.015	2631.505	0.014	2852.002	0.014	3072.499	0.014	3292.996	0.014	3513.493	0.014	3731.785	
0.017	2513.650	0.017	2743.534	0.017	2973.418	0.016	3203.302	0.016	3433.186	0.016	3663.070	0.016	3890.655	
0.019	2585.301	0.019	2821.738	0.019	3058.175	0.018	3294.612	0.018	3531.049	0.018	3767.485	0.018	4001.558	
0.021	2634.798	0.021	2875.762	0.021	3116.725	0.021	3357.689	0.020	3598.652	0.020	3839.616	0.020	4078.170	
0.023	2668.744	0.023	2912.812	0.023	3156.880	0.023	3400.948	0.022	3645.016	0.022	3889.084	0.022	4130.711	
0.025	2691.908	0.025	2938.095	0.025	3184.281	0.025	3430.468	0.025	3676.654	0.024	3922.841	0.024	4166.566	
0.027	2707.662	0.027	2955.290	0.027	3202.917	0.027	3450.544	0.027	3698.172	0.026	3945.799	0.026	4190.950	
0.029	2718.352	0.029	2966.957	0.029	3215.562	0.029	3464.166	0.029	3712.771	0.028	3961.376	0.028	4207.495	
0.031	2725.593	0.031	2974.860	0.031	3224.128	0.031	3473.395	0.031	3722.662	0.031	3971.929	0.030	4218.704	
0.034	2730.494	0.033	2980.209	0.033	3229.925	0.033	3479.640	0.033	3729.355	0.033	3979.071	0.032	4226.289	

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap at abutments (Elevation 79.0 m).
Please note the following assumptions:
1. P-y curves have been generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for H-pile 360x132.

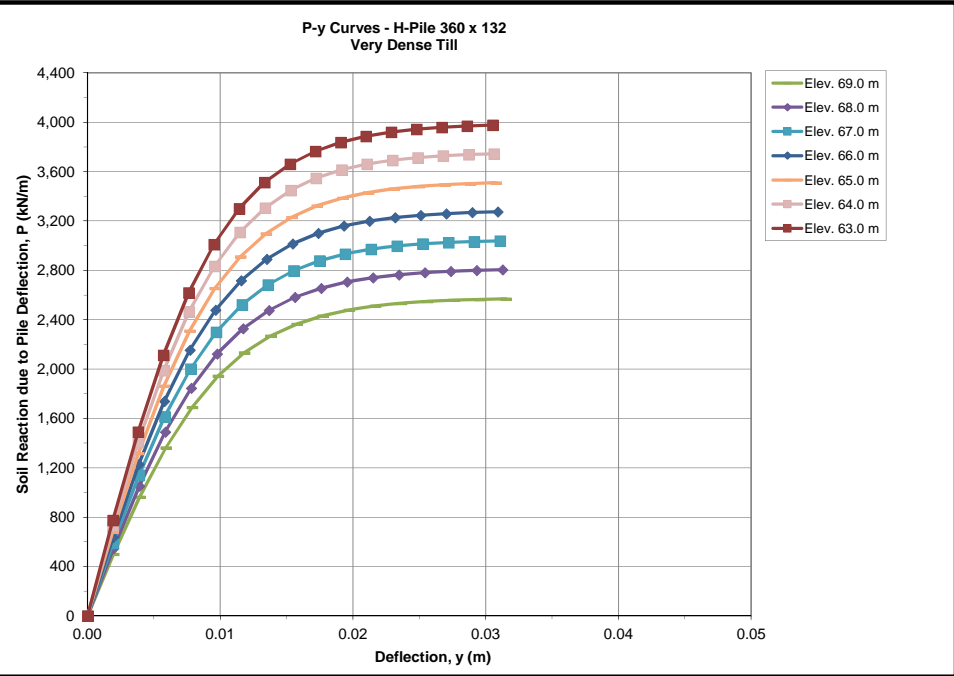
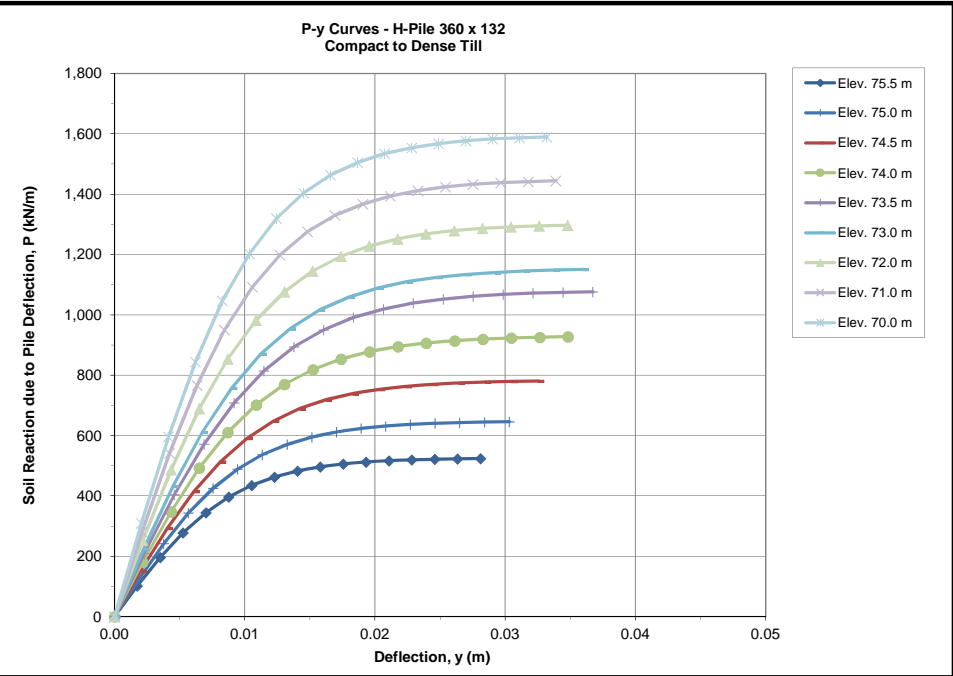
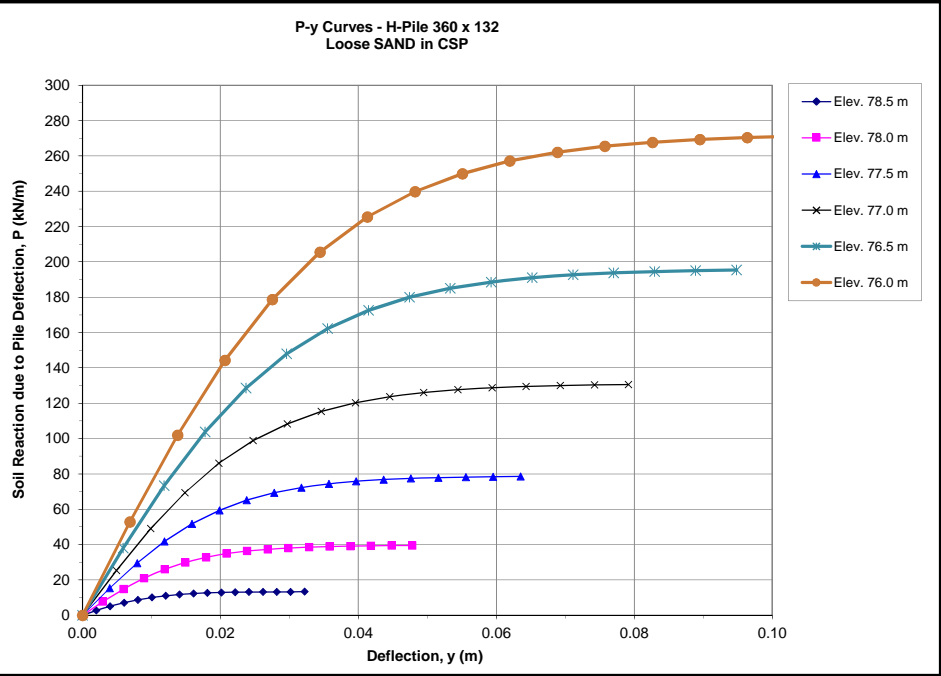


SUMMARY OF P-y CURVES FOR A H-Pile 360x132 - Abutments - Cyclic Loading

Description Depth (z) * Elevation P-y Curves	Loose Sand (CSP)												Compact to Dense Silt and Sand Till																		
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		
	Elev. 78.5 m		Elev. 78.0 m		Elev. 77.5 m		Elev. 77.0 m		Elev. 76.5 m		Elev. 76.0 m		Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.5 m		Elev. 73.0 m		Elev. 72.0 m		Elev. 71.0 m		Elev. 70.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	2.583	0.003	7.679	0.004	15.285	0.005	25.404	0.006	38.034	0.007	52.848	0.002	101.980	0.002	125.732	0.002	151.919	0.002	180.540	0.002	209.538	0.002	223.848	0.002	252.468	0.002	281.088	0.002	309.422	0.002	309.422
0.004	4.980	0.006	14.801	0.008	29.463	0.010	48.967	0.012	73.313	0.014	101.868	0.004	196.572	0.004	242.356	0.004	292.833	0.004	348.002	0.005	403.897	0.004	431.480	0.004	486.647	0.004	541.813	0.004	596.428	0.004	596.428
0.006	7.052	0.009	20.961	0.012	41.726	0.015	69.347	0.018	103.825	0.021	144.265	0.005	278.384	0.006	343.224	0.006	414.709	0.007	492.839	0.007	571.997	0.007	611.061	0.007	689.187	0.006	767.314	0.006	844.659	0.006	844.659
0.008	8.739	0.012	25.975	0.016	51.707	0.020	85.935	0.024	128.659	0.028	178.772	0.007	344.972	0.008	425.321	0.008	513.904	0.009	610.722	0.009	708.815	0.009	757.222	0.009	854.035	0.008	950.849	0.008	1046.695	0.008	1046.695
0.010	10.046	0.015	29.858	0.020	59.436	0.025	98.780	0.030	147.891	0.034	205.495	0.009	396.538	0.009	488.897	0.010	590.722	0.011	702.012	0.011	814.767	0.011	870.410	0.011	981.695	0.011	1092.981	0.010	1203.153	0.010	1203.153
0.012	11.019	0.018	32.750	0.024	65.194	0.030	108.350	0.036	162.218	0.041	225.403	0.011	434.954	0.011	536.261	0.012	647.950	0.013	770.022	0.014	893.701	0.013	954.734	0.013	1076.801	0.013	1198.867	0.012	1319.713	0.012	1319.713
0.014	11.723	0.021	34.843	0.028	69.361	0.035	115.275	0.041	172.587	0.048	239.810	0.012	462.755	0.013	570.538	0.014	689.366	0.015	819.240	0.016	950.825	0.016	1015.759	0.015	1145.628	0.015	1275.497	0.015	1404.067	0.015	1404.067
0.016	12.222	0.024	36.327	0.032	72.314	0.040	120.183	0.047	179.934	0.055	250.019	0.014	482.456	0.015	594.827	0.016	718.714	0.017	854.117	0.018	991.303	0.018	1059.002	0.017	1194.400	0.017	1329.797	0.017	1463.841	0.017	1463.841
0.018	12.571	0.027	37.362	0.036	74.375	0.045	123.609	0.053	185.063	0.062	257.146	0.016	496.208	0.017	611.782	0.018	739.201	0.020	878.464	0.021	1019.561	0.020	1089.189	0.020	1228.446	0.019	1367.703	0.019	1505.568	0.019	1505.568
0.020	12.811	0.030	38.078	0.040	75.799	0.049	125.975	0.059	188.607	0.069	262.069	0.018	505.708	0.019	623.495	0.020	753.353	0.022	895.282	0.023	1039.081	0.022	1110.042	0.022	1251.965	0.021	1393.889	0.021	1534.393	0.021	1534.393
0.022	12.976	0.033	38.568	0.044	76.775	0.054	127.598	0.065	191.037	0.076	265.446	0.019	512.224	0.021	631.528	0.022	763.059	0.024	906.817	0.025	1052.468	0.025	1124.344	0.024	1268.095	0.023	1411.847	0.023	1554.161	0.023	1554.161
0.024	13.089	0.036	38.903	0.048	77.442	0.059	128.706	0.071	192.695	0.083	267.750	0.021	516.670	0.023	637.010	0.024	769.682	0.026	914.688	0.028	1061.603	0.027	1134.103	0.026	1279.102	0.025	1424.102	0.025	1567.651	0.025	1567.651
0.026	13.166	0.039	39.131	0.052	77.895	0.064	129.459	0.077	193.822	0.090	269.317	0.023	519.693	0.025	640.738	0.026	774.187	0.028	920.041	0.030	1067.816	0.029	1140.740	0.028	1286.588	0.028	1432.436	0.027	1576.826	0.027	1576.826
0.028	13.218	0.042	39.285	0.056	78.203	0.069	129.970	0.083	194.588	0.096	270.380	0.025	521.745	0.027	643.267	0.028	777.243	0.030	923.673	0.032	1072.032	0.031	1145.243	0.030	1291.667	0.030	1438.091	0.029	1583.051	0.029	1583.051
0.030	13.253	0.045	39.390	0.060	78.411	0.074	130.316	0.089	195.106	0.103	271.100	0.026	523.135	0.028	644.981	0.031	779.314	0.033	926.134	0.034	1074.887	0.034	1148.294	0.033	1295.108	0.032	1441.922	0.031	1587.268	0.031	1587.268
0.032	13.277	0.048	39.461	0.064	78.552	0.079	130.551	0.095	195.457	0.110	271.587	0.028	524.076	0.030	646.141	0.033	780.715	0.035	927.799	0.037	1076.820	0.036	1150.359	0.035	1297.437	0.034	1444.515	0.033	1590.122	0.033	1590.122

Description Depth (z) * Elevation P-y Curves	Very Dense Sand and Silt Till													
	z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m	
	Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	499.988	0.002	545.715	0.002	591.441	0.002	637.167	0.002	682.893	0.002	728.619	0.002	773.888	
0.004	963.756	0.004	1051.895	0.004	1140.035	0.004	1228.174	0.004	1316.314	0.004	1404.453	0.004	1491.712	
0.006	1364.867	0.006	1489.690	0.006	1614.513	0.006	1739.336	0.006	1864.159	0.006	1988.982	0.006	2112.557	
0.008	1691.333	0.008	1846.013	0.008	2000.692	0.008	2155.372	0.008	2310.052	0.008	2464.731	0.008	2617.864	
0.010	1944.151	0.010	2121.952	0.010	2299.753	0.010	2477.554	0.010	2655.355	0.010	2833.156	0.010	3009.179	
0.012	2132.498	0.012	2327.524	0.012	2522.550	0.012	2717.576	0.012	2912.602	0.011	3107.628	0.011	3300.704	
0.014	2268.804	0.014	2476.296	0.014	2683.788	0.014	2891.279	0.013	3098.771	0.013	3306.263	0.013	3511.680	
0.016	2365.391	0.016	2581.717	0.016	2798.042	0.015	3014.367	0.015	3230.692	0.015	3447.017	0.015	3661.179	
0.018	2432.817	0.018	2655.308	0.017	2877.800	0.017	3100.291	0.017	3322.783	0.017	3545.274	0.017	3765.541	
0.020	2479.395	0.020	2706.146	0.019	2932.897	0.019	3159.648	0.019	3386.399	0.019	3613.151	0.019	3837.634	
0.022	2511.338	0.022	2741.010	0.021	2970.683	0.021	3200.356	0.021	3430.028	0.021	3659.701	0.021	3887.077	
0.024	2533.136	0.023	2764.802	0.023	2996.469	0.023	3228.135	0.023	3459.801	0.023	3691.467	0.023	3920.816	
0.026	2547.961	0.025	2780.983	0.025	3014.005	0.025	3247.027	0.025	3480.049	0.025	3713.071	0.025	3943.762	
0.028	2558.020	0.027	2791.962	0.027	3025.904	0.027	3259.846	0.027	3493.787	0.027	3727.729	0.027	3959.332	
0.030	2564.834	0.029	2799.400	0.029	3033.965	0.029	3268.530	0.029	3503.095	0.029	3737.660	0.029	3969.879	
0.032	2569.446	0.031	2804.433	0.031	3039.420	0.031	3274.407	0.031	3509.393	0.031	3744.380	0.031	3977.017	

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap at abutments (Elevation 79.0 m).
Please note the following assumptions:
1. P-y curves have been generated for vertical piles (i.e. no inclination)
2. Cyclic loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for H-pile 360x132.

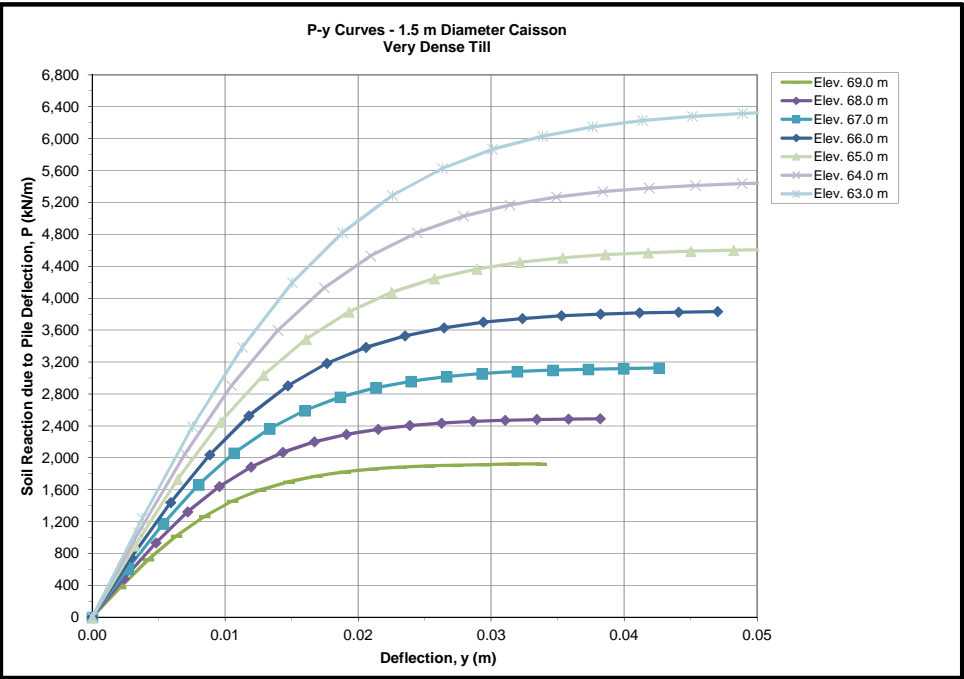
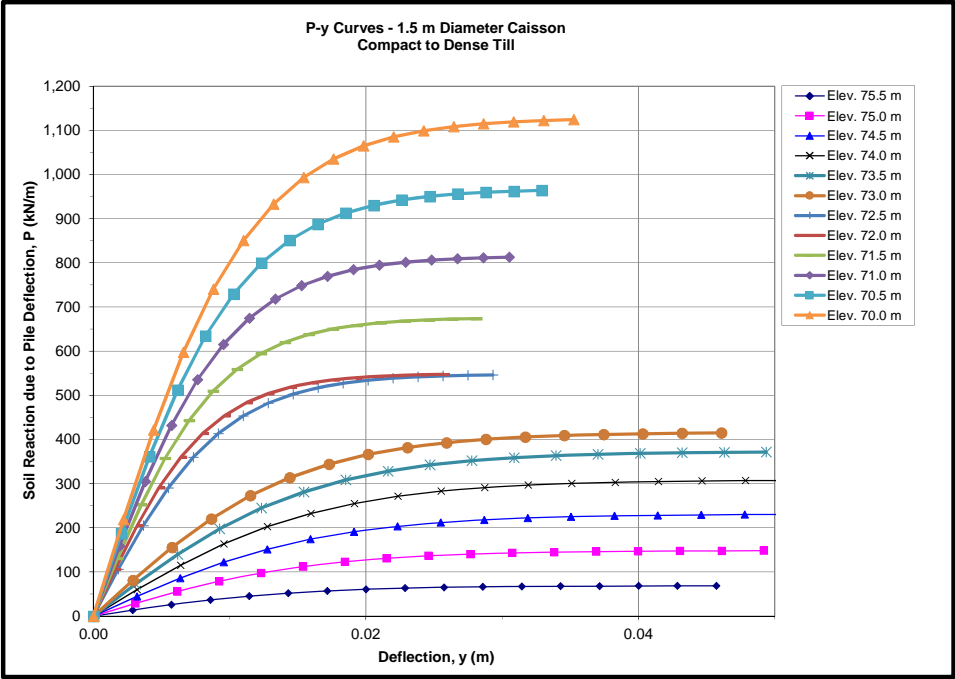


SUMMARY OF P-y CURVES FOR A 1.5 m Diameter Caisson - Pier

Description Depth (z) * Elevation P-y Curves	Compact to Dense Sand and Silt Till																							
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m	
	Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.5 m		Elev. 73.0 m		Elev. 72.5 m		Elev. 72.0 m		Elev. 71.5 m		Elev. 71.0 m		Elev. 70.5 m		Elev. 70.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.003	13.393	0.003	28.835	0.003	44.814	0.003	59.818	0.003	72.334	0.003	80.730	0.002	106.305	0.002	106.563	0.002	131.141	0.002	158.154	0.002	187.600	0.002	218.820	0.002
0.006	25.816	0.006	55.581	0.006	86.382	0.006	115.302	0.006	139.429	0.006	155.612	0.004	204.909	0.003	205.406	0.004	252.782	0.004	304.850	0.004	361.610	0.004	421.787	0.004
0.009	36.560	0.009	78.714	0.010	122.334	0.010	163.291	0.009	197.459	0.009	220.377	0.005	290.191	0.005	290.895	0.005	357.989	0.006	431.727	0.006	512.111	0.007	597.334	0.007
0.011	45.305	0.012	97.542	0.013	151.595	0.013	202.349	0.012	244.689	0.012	273.090	0.007	359.603	0.006	360.475	0.007	443.617	0.008	534.993	0.008	634.604	0.009	740.212	0.009
0.014	52.077	0.015	112.122	0.016	174.255	0.016	232.596	0.015	281.265	0.014	313.911	0.009	413.356	0.008	414.358	0.009	509.928	0.010	614.963	0.010	729.464	0.011	850.857	0.011
0.017	57.123	0.018	122.985	0.019	191.136	0.019	255.129	0.019	308.513	0.017	344.322	0.011	453.401	0.010	454.500	0.011	559.329	0.011	674.540	0.012	800.133	0.013	933.287	0.013
0.020	60.774	0.022	130.846	0.022	203.354	0.022	271.437	0.022	328.233	0.020	366.331	0.013	482.382	0.011	483.551	0.012	595.081	0.013	717.656	0.014	851.276	0.015	992.942	0.015
0.023	63.361	0.025	136.416	0.025	212.011	0.026	282.992	0.025	342.207	0.023	381.926	0.015	502.918	0.013	504.137	0.014	620.414	0.015	748.208	0.016	887.517	0.018	1035.213	0.018
0.026	65.167	0.028	140.304	0.029	218.054	0.029	291.059	0.028	351.961	0.026	392.813	0.016	517.253	0.014	518.507	0.016	638.099	0.017	769.535	0.019	912.816	0.020	1064.722	0.020
0.029	66.415	0.031	142.991	0.032	222.229	0.032	296.631	0.031	358.700	0.029	400.334	0.018	527.156	0.016	528.435	0.018	650.316	0.019	784.268	0.021	930.292	0.022	1085.106	0.022
0.031	67.271	0.034	144.833	0.035	225.092	0.035	300.453	0.034	363.321	0.032	405.491	0.020	533.948	0.018	535.243	0.019	658.694	0.021	794.373	0.023	942.278	0.024	1099.086	0.024
0.034	67.854	0.037	146.090	0.038	227.046	0.038	303.061	0.037	366.475	0.035	409.011	0.022	538.583	0.019	539.889	0.021	664.412	0.023	801.268	0.025	950.456	0.026	1108.627	0.026
0.037	68.252	0.040	146.945	0.041	228.375	0.041	304.835	0.040	368.619	0.037	411.405	0.024	541.735	0.021	543.048	0.023	668.300	0.025	805.957	0.027	956.019	0.029	1115.115	0.029
0.040	68.521	0.043	147.525	0.045	229.276	0.045	306.038	0.043	370.075	0.040	413.029	0.026	543.873	0.022	545.192	0.025	670.939	0.027	809.139	0.029	959.793	0.031	1119.517	0.031
0.043	68.704	0.046	147.918	0.048	229.887	0.048	306.853	0.046	371.061	0.043	414.129	0.027	545.322	0.024	546.644	0.026	672.726	0.029	811.294	0.031	962.350	0.033	1122.499	0.033
0.046	68.827	0.049	148.184	0.051	230.300	0.051	307.405	0.049	371.728	0.046	414.874	0.029	546.303	0.026	547.627	0.028	673.935	0.031	812.753	0.033	964.080	0.035	1124.518	0.035

Description Depth (z) * Elevation P-y Curves	Very Dense Sand and Silt Till											
	z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m	
	Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 64.0 m		Elev. 63.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	374.506	0.002	484.453	0.003	608.211	0.003	745.778	0.003	897.154	0.003	1062.340	0.004
0.004	721.881	0.005	933.811	0.005	1172.360	0.006	1437.528	0.006	1729.315	0.007	2047.720	0.008
0.006	1022.325	0.007	1322.460	0.008	1660.292	0.009	2035.822	0.010	2449.050	0.010	2899.974	0.011
0.008	1266.857	0.010	1638.782	0.011	2057.422	0.012	2522.775	0.013	3034.843	0.014	3593.625	0.015
0.011	1456.225	0.012	1883.745	0.013	2364.962	0.015	2899.876	0.016	3488.488	0.017	4130.796	0.019
0.013	1597.302	0.014	2066.240	0.016	2594.077	0.018	3180.813	0.019	3826.448	0.021	4530.982	0.023
0.015	1699.399	0.017	2198.310	0.019	2759.886	0.021	3384.125	0.022	4071.028	0.024	4820.595	0.026
0.017	1771.746	0.019	2291.897	0.021	2877.380	0.024	3528.194	0.026	4244.340	0.028	5025.817	0.030
0.019	1822.249	0.021	2357.228	0.024	2959.399	0.026	3628.765	0.029	4365.325	0.031	5169.078	0.034
0.021	1857.137	0.024	2402.358	0.027	3016.059	0.029	3698.240	0.032	4448.901	0.035	5268.043	0.038
0.023	1881.064	0.026	2433.309	0.029	3054.916	0.032	3745.886	0.035	4506.219	0.038	5335.914	0.041
0.025	1897.391	0.029	2454.430	0.032	3081.433	0.035	3778.401	0.039	4545.333	0.042	5382.230	0.045
0.027	1908.496	0.031	2468.794	0.035	3099.467	0.038	3800.513	0.042	4571.934	0.045	5413.728	0.049
0.030	1916.030	0.033	2478.540	0.037	3111.703	0.041	3815.517	0.045	4589.983	0.049	5435.101	0.053
0.032	1921.134	0.036	2485.143	0.040	3119.992	0.044	3825.681	0.048	4602.211	0.052	5449.580	0.056
0.034	1924.588	0.038	2489.611	0.043	3125.602	0.047	3832.560	0.051	4610.485	0.056	5459.378	0.060

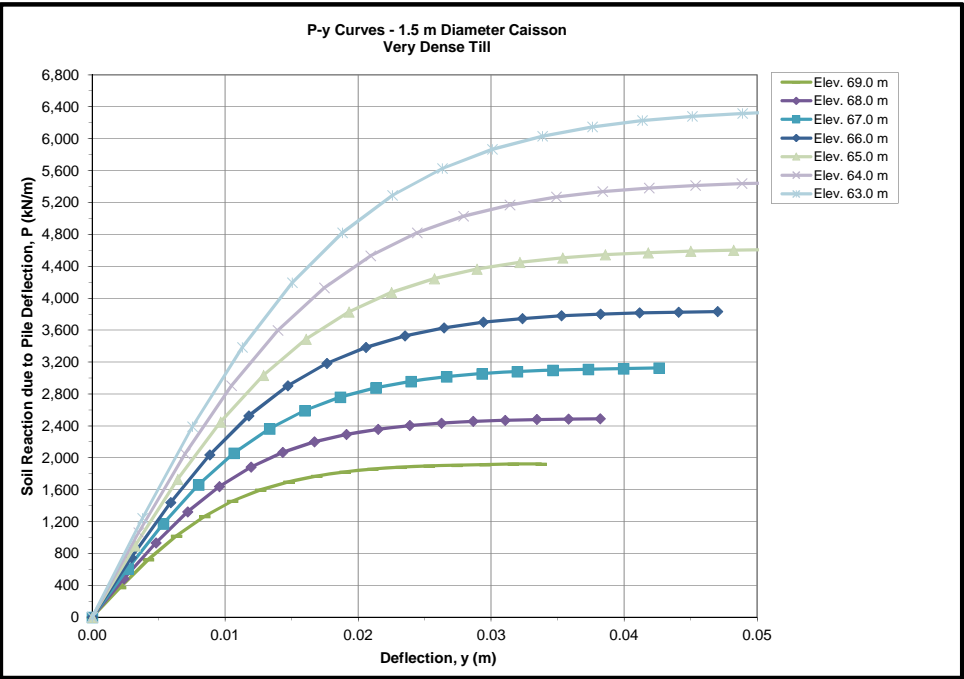
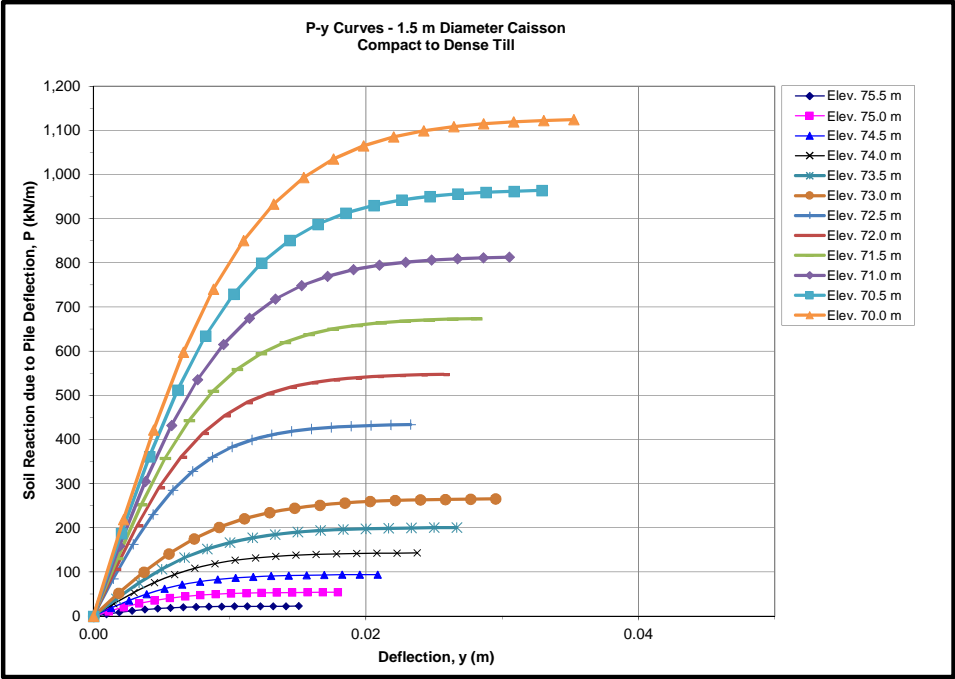
NOTES: * Depth (z) is measured to be positive below the underside of the caisson cap at center pier (Elevation 76.0 m).
Please note the following assumptions:
1. P-y curves have been generated for vertical caissons (i.e. no inclination)
2. Static loading condition is considered.
3. There are no group effects (i.e. analysis is based on a single caisson).
4. P-y curves have been generated for a single 1.5 m diameter caisson.



SUMMARY OF P-y CURVES FOR A 1.5 m Diameter Caisson - Pier - Cylic Loading

Description Depth (z) * Elevation P-y Curves	Compact to Dense Sand and Silt Till																							
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m	
	Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.5 m		Elev. 73.0 m		Elev. 72.5 m		Elev. 72.0 m		Elev. 71.5 m		Elev. 71.0 m		Elev. 70.5 m		Elev. 70.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.001	4.410	0.001	10.521	0.001	18.333	0.001	27.846	0.002	39.061	0.002	51.701	0.001	84.419	0.002	106.563	0.002	131.141	0.002	158.154	0.002	187.600	0.002	218.820
	0.002	8.500	0.002	20.280	0.003	35.338	0.003	53.675	0.003	75.292	0.004	99.657	0.003	162.722	0.003	205.406	0.004	252.782	0.004	304.850	0.004	361.610	0.004	421.787
	0.003	12.038	0.003	28.720	0.004	50.046	0.004	76.015	0.005	106.628	0.006	141.134	0.004	230.446	0.005	290.895	0.005	357.989	0.006	431.727	0.006	512.111	0.007	597.334
	0.004	14.918	0.004	35.590	0.005	62.016	0.006	94.197	0.007	132.132	0.007	174.892	0.006	285.567	0.006	360.475	0.007	443.617	0.008	534.993	0.008	634.604	0.009	740.211
	0.005	17.147	0.006	40.909	0.007	71.286	0.007	108.277	0.008	151.883	0.009	201.034	0.007	328.253	0.008	414.358	0.009	509.928	0.010	614.963	0.010	729.464	0.011	850.857
0.006	18.809	0.007	44.873	0.008	78.192	0.009	118.767	0.010	166.597	0.011	220.510	0.009	360.054	0.010	454.500	0.011	559.329	0.011	674.540	0.012	800.133	0.013	933.287	
0.007	20.011	0.008	47.741	0.009	83.190	0.010	126.358	0.012	177.246	0.013	234.605	0.010	383.068	0.011	483.551	0.012	595.081	0.013	717.656	0.014	851.276	0.015	992.942	
0.008	20.863	0.009	49.773	0.010	86.732	0.012	131.738	0.013	184.792	0.015	244.592	0.012	399.376	0.013	504.137	0.014	620.414	0.015	748.208	0.016	887.517	0.018	1035.213	
0.008	21.457	0.010	51.192	0.012	89.204	0.013	135.493	0.015	190.059	0.017	251.564	0.013	410.760	0.014	518.507	0.016	638.099	0.017	769.535	0.019	912.816	0.020	1064.722	
0.009	21.868	0.011	52.172	0.013	90.912	0.015	138.087	0.017	193.698	0.018	256.381	0.015	418.624	0.016	528.434	0.018	650.316	0.019	784.268	0.021	930.292	0.022	1085.106	
0.010	22.150	0.012	52.844	0.014	92.083	0.016	139.866	0.018	196.193	0.020	259.684	0.016	424.017	0.018	535.243	0.019	658.694	0.021	794.373	0.023	942.277	0.024	1099.086	
0.011	22.342	0.013	53.303	0.016	92.882	0.018	141.080	0.020	197.896	0.022	261.938	0.017	427.698	0.019	539.888	0.021	664.412	0.023	801.268	0.025	950.456	0.026	1108.627	
0.012	22.473	0.015	53.615	0.017	93.426	0.019	141.906	0.022	199.054	0.024	263.471	0.019	430.201	0.021	543.048	0.023	668.300	0.025	805.957	0.027	956.019	0.029	1115.115	
0.013	22.562	0.016	53.827	0.018	93.795	0.021	142.466	0.023	199.840	0.026	264.511	0.020	431.899	0.022	545.192	0.025	670.938	0.027	809.139	0.029	959.793	0.031	1119.517	
0.014	22.622	0.017	53.970	0.020	94.045	0.022	142.845	0.025	200.373	0.028	265.216	0.022	433.050	0.024	546.644	0.026	672.726	0.029	811.294	0.031	962.350	0.033	1122.499	
0.015	22.663	0.018	54.067	0.021	94.214	0.024	143.102	0.027	200.733	0.029	265.692	0.023	433.828	0.026	547.627	0.028	673.935	0.031	812.753	0.033	964.080	0.035	1124.518	
Description Depth (z) * Elevation P-y Curves	Very Dense Sand and Silt Till																							
	z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m											
	Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m											
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)										
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000										
	0.002	374.506	0.002	484.453	0.003	608.211	0.003	745.778	0.003	897.154	0.003	1062.340	0.004	1239.478										
	0.004	721.880	0.005	933.811	0.005	1172.360	0.006	1437.528	0.006	1729.315	0.007	2047.720	0.008	2389.162										
	0.006	1022.325	0.007	1322.460	0.008	1660.292	0.009	2035.822	0.010	2449.050	0.010	2899.974	0.011	3383.523										
	0.008	1266.857	0.010	1638.782	0.011	2057.422	0.012	2522.775	0.013	3034.843	0.014	3593.625	0.015	4192.836										
	0.011	1456.225	0.012	1883.745	0.013	2364.962	0.015	2899.876	0.016	3488.488	0.017	4130.796	0.019	4819.575										
	0.013	1597.302	0.014	2066.240	0.016	2594.077	0.018	3180.813	0.019	3826.448	0.021	4530.982	0.023	5286.489										
	0.015	1699.399	0.017	2198.310	0.019	2759.886	0.021	3384.125	0.022	4071.028	0.024	4820.595	0.026	5624.393										
	0.017	1771.746	0.019	2291.897	0.021	2877.380	0.024	3528.194	0.026	4244.340	0.028	5025.817	0.030	5863.835										
	0.019	1822.249	0.021	2357.228	0.024	2959.399	0.026	3628.765	0.029	4365.325	0.031	5169.078	0.034	6030.983										
	0.021	1857.137	0.024	2402.358	0.027	3016.059	0.029	3698.240	0.032	4448.901	0.035	5268.043	0.038	6146.450										
	0.023	1881.064	0.026	2433.309	0.029	3054.916	0.032	3745.886	0.035	4506.219	0.038	5335.914	0.041	6225.638										
	0.025	1897.391	0.029	2454.430	0.032	3081.433	0.035	3778.401	0.039	4545.333	0.042	5382.230	0.045	6279.676										
	0.027	1908.496	0.031	2468.794	0.035	3099.467	0.038	3800.513	0.042	4571.934	0.045	5413.728	0.049	6316.427										
	0.030	1916.030	0.033	2478.540	0.037	3111.703	0.041	3815.517	0.045	4589.983	0.049	5435.101	0.053	6341.363										
0.032	1921.134	0.036	2485.143	0.040	3119.992	0.044	3825.681	0.048	4602.211	0.052	5449.580	0.056	6358.257											
0.034	1924.588	0.038	2489.611	0.043	3125.602	0.047	3832.560	0.051	4610.485	0.056	5459.378	0.060	6369.689											

NOTES: * Depth (z) is measured to be positive below the underside of the caisson cap at center pier (Elevation 76.0 m).
Please note the following assumptions:
1. P-y curves have been generated for vertical caissons (i.e. no inclination)
2. Cyclic loading condition is considered.
3. There are no group effects (i.e. analysis is based on a single caisson).
4. P-y curves have been generated for a single 1.5 m diameter caisson.

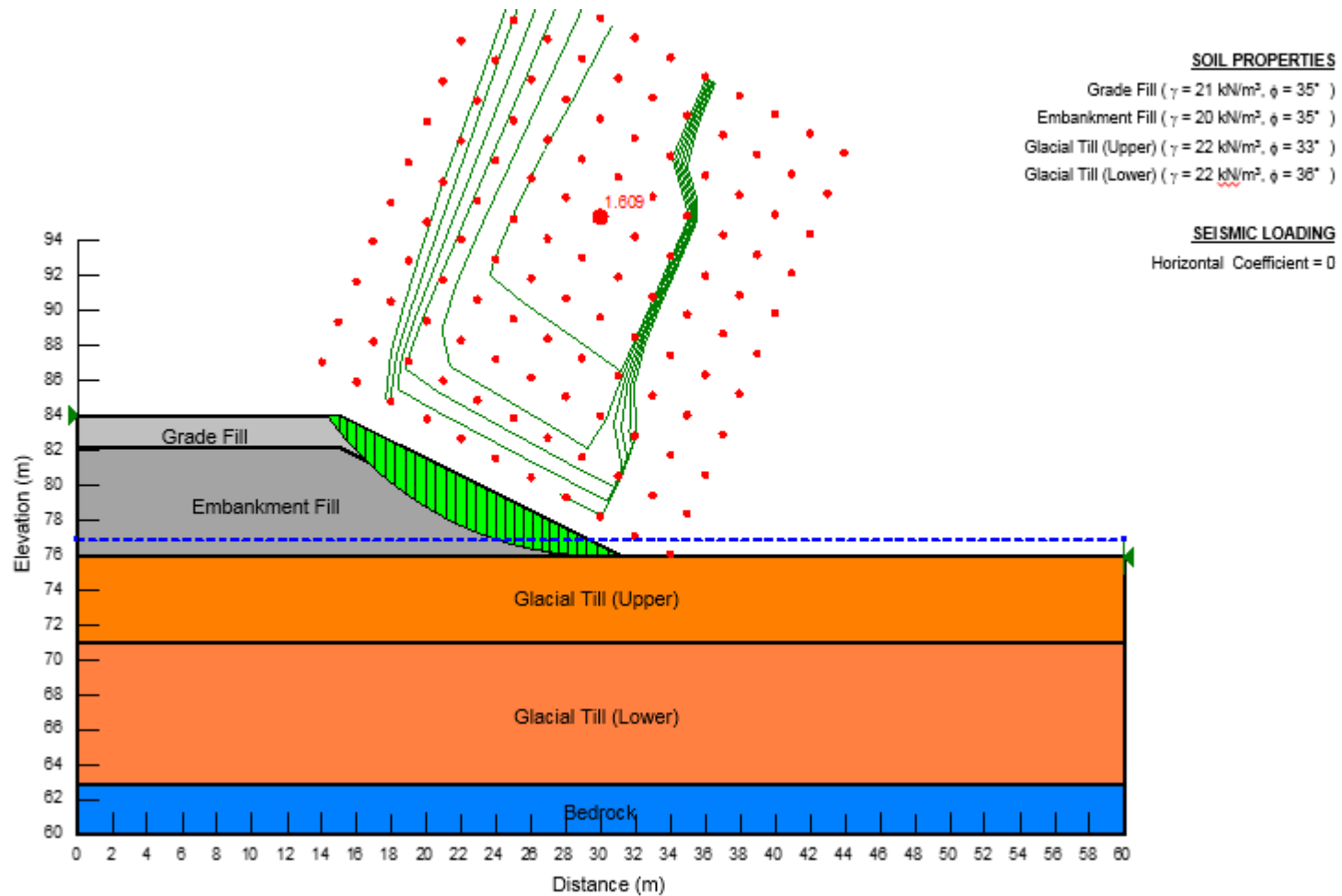


APPENDIX H

Slope Stability Analysis Results

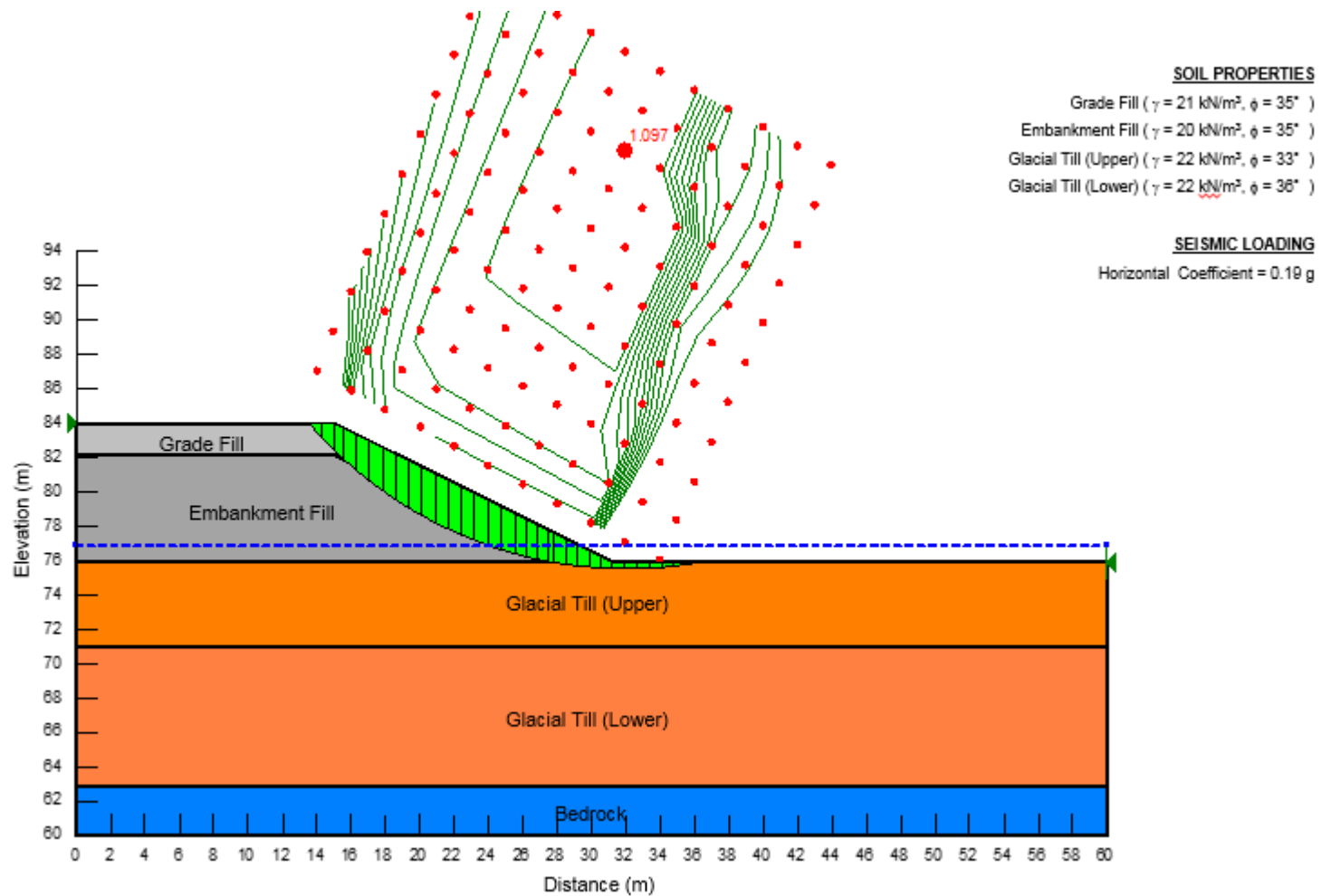
Stability Analysis - Embankments Static Loading Conditions

Figure H1



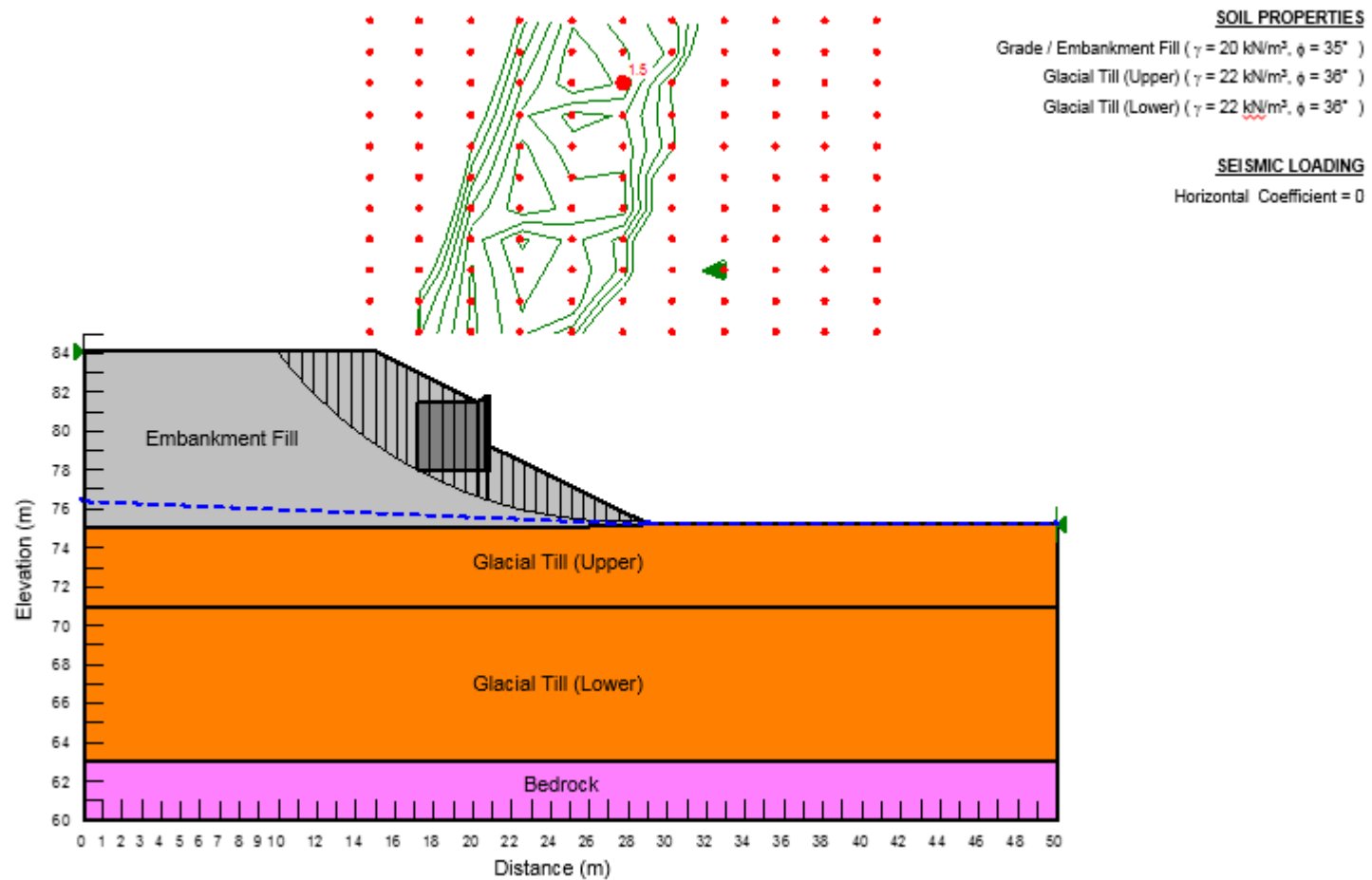
Stability Analysis - Embankments Static Loading Conditions

Figure H2



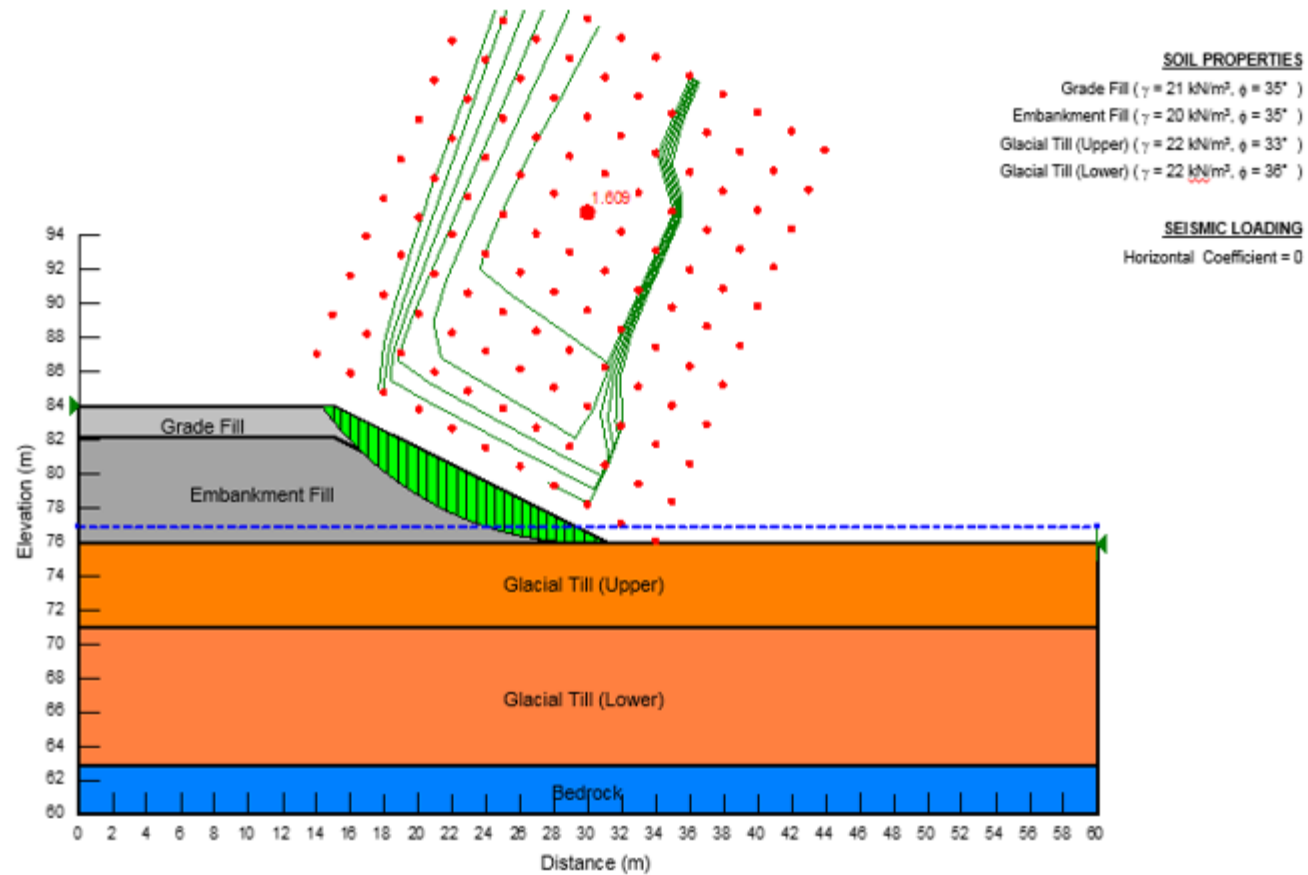
Stability Analysis – Northeast RSS Wall Static Loading Conditions

Figure H3



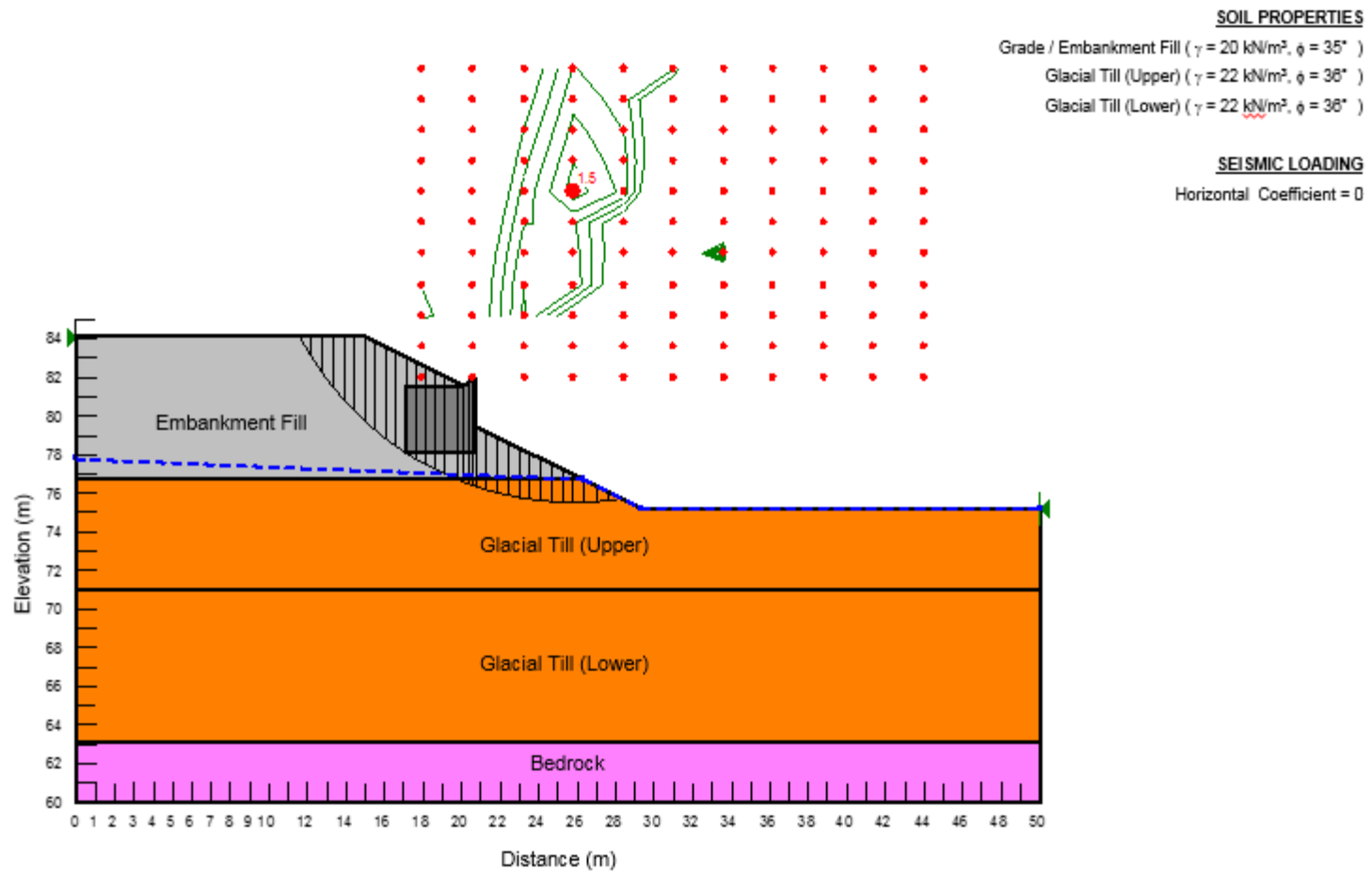
Stability Analysis – Northeast RSS Wall Static Loading Conditions

Figure H4



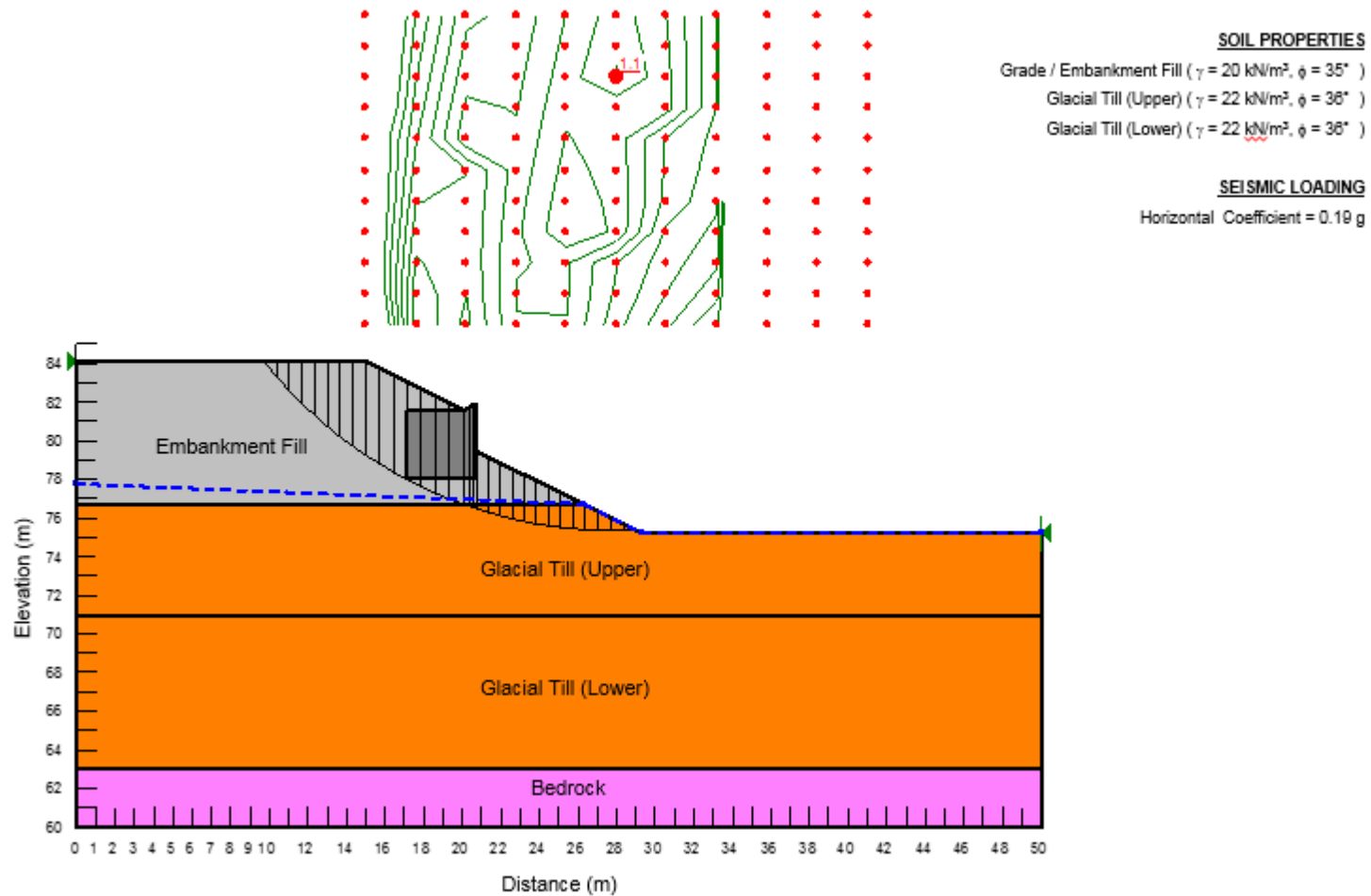
Stability Analysis – Southwest RSS Wall Static Loading Conditions

Figure H5



Stability Analysis – Southwest RSS Wall Static Loading Conditions

Figure H6





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