



**DETAIL FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PEDESTRIAN UNDERPASS - SITE NO. 43-370
OVER HIGHWAY 11/17, FROM MEIGHEN AVENUE/ FROST STREET
TO CHIPPEWA STREET
CITY OF NORTH BAY, ONTARIO
AGREEMENT NUMBER 5005-E-0069
GWP NO. 5159-01-00**

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PML Ref.: 06TF060
Index No.: 045FIR and 046FDR
Geocres No.: 31L-110
July 31, 2007



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TABLE OF CONTENTS

1. INTRODUCTION.....	1
2. SITE DESCRIPTION.....	1
3. INVESTIGATION PROCEDURES.....	2
4. SUMMARIZED SUBSURFACE CONDITIONS	4
4.1 South Abutment (Boreholes 1 and 2)	4
4.1.1 Topsoil	4
4.1.2 Sand.....	4
4.1.3 Silty Clay	5
4.1.4 Clayey Silt	5
4.1.5 Bedrock.....	5
4.1.6 Groundwater	6
4.2 Bridge Pier (Boreholes 3 and 4).....	6
4.2.1 Fill	6
4.2.2 Sand.....	6
4.2.3 Clayey Silt	7
4.2.4 Bedrock.....	7
4.2.5 Groundwater	7
4.3 North Abutment (Boreholes 5 and 6).....	8
4.3.1 Fill	8
4.3.2 Topsoil	8
4.3.3 Silty sand.....	8
4.3.4 Silty Clay	8
4.3.5 Clayey Silt	9
4.3.6 Bedrock.....	9
4.3.7 Groundwater	9
4.4 South Embankment (Boreholes 1, 2, 7, 8 and 9).....	10
4.4.1 Topsoil	10
4.4.2 Sand.....	10
4.4.3 Silty Clay	10
4.4.4 Clayey Silt	11
4.4.5 Bedrock.....	11
4.4.6 Groundwater	11



4.5 North Embankment (Boreholes 5, 6, 10 to 19 and Cone Penetration Test 20).....	12
4.5.1 Fill	12
4.5.2 Topsoil/Peat	12
4.5.3 Silty sand.....	12
4.5.4 Silty Clay	13
4.5.5 Clayey Silt	13
4.5.6 Sand/ Sandy Silt/ Silt	14
4.5.7 Bedrock.....	14
4.5.8 Groundwater	14
5. CLOSURE.....	15

Table 1 - List of Atterberg Plasticity Test Results

Table 2 - Rock Core Description (Borehole 1)

Table 3 - Rock Core Description (Borehole 4)

Table 4 - Rock Core Description (Borehole 6)

Figures PC-1 and PC-2 - Plasticity Charts

Figures 1 to 4 - Particle Size Distribution Charts

Figure 5 - Results of Consolidation Test

Explanation of Terms Used in Report

Record of Borehole Sheets

Drawings 1 and 2 - Borehole Locations and Soil Strata

Drawings 3 to 4 - Soil Strata

Appendix A - Site Photographs

Appendix B - Rock Core Photographs



FOUNDATION INVESTIGATION REPORT

for

Pedestrian Underpass - Site No. 43-370
Over Highway 11/17, from Meighen Avenue/ Frost Street
To Chippewa Street
City of North Bay, Ontario
GWP No. 5159-01-00

1. INTRODUCTION

This report presents the results of the foundation investigation carried out at the site of a new pedestrian underpass over the Highway 11/17 and associated embankments in the City of North Bay, Ontario. The foundation study was carried out for Stantec Consulting Limited (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The structure is to be located at the approximate Sta. 14+450 of Highway 11/17 chainage. The proposed 66 m long two-span pedestrian underpass will extend from a location north of the highway immediately east of the highway exit ramp at Meighen Avenue/Frost Street to the north end of Chippewa Street on the south side of the highway. The south embankment will support a combination of stairway/ramps leading to the south abutment. The north embankment will be about 175 m long and extend from the north bridge abutment to the intersection of Gardiner and Harrison Streets.

This report summarizes the results of the foundation investigation carried out at the sites of the new pedestrian underpass foundations and associated embankments.

2. SITE DESCRIPTION

The existing Highway 11/17 through the investigated section is presently a four-lane highway with an exit/entry ramp to/from Meighen Avenue/Frost Street from the Highway 11/17 westbound lanes.

The pedestrian bridge will span the highway and the north end of Chippewa Street from the end of Meighen Avenue/Frost Street to the south side of Chippewa Street. The pier location is off the south shoulder of the highway, about 26 m from the south abutment.

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The ground surface is relatively level at the proposed south abutment and embankment locations. The proposed north abutment is about 8 m north of the north edge of the highway pavement and at the edge of a relatively flat and low wet/treed area that extends northerly about 130 m and then rises about 2 m to the level of the adjacent street, as inferred from a site inspection. A localized fill area and a shallow water filled pond were noted on the northeast quadrant of the proposed embankment and the existing highway. Site photographs are enclosed in Appendix A.

The investigated bridge alignment and embankments are located across a variable geological area comprising deposits of sands and clays with bedrock outcrops readily visible north and east of the north abutment and approach embankment, but undulating below grade, as inferred from the investigation.

The pedestrian underpass location is within the City of North Bay. The northern work areas are close to residences and to industrial/commercial areas while the southern work areas are located near schools and a retirement residence.

Overhead power lines cross the highway in the vicinity of the north end of Chippewa Street.

The bedrock underlying the pedestrian bridge structure site is comprised of granite or granitic gneisses.

The assessed frost penetration depth for the area of the pedestrian bridge is 2.0 m.

3. INVESTIGATION PROCEDURES

The subsurface investigation was carried out during the periods from December 2 to 6, 2006 and from January 15 to 17, 2007. Nineteen boreholes and 2 dynamic cone penetration tests (cone tests) were drilled at the site for foundation investigation purposes. The boreholes and cone tests for the bridge foundations were numbered from 1 to 6 and those drilled for the embankments were designated from 7 to 20.



The boreholes were advanced using continuous flight hollow stem augers through the soil cover with a track-mounted CME-75 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. Cores of the bedrock were taken in one borehole at each of the three bridge foundation locations using rotary drilling and NQ coring bits. The boreholes and cone tests extended to depths ranging from 1.2 to 10.1 m, including the 3.3 m long rock core obtained in the deeper borehole.

The borehole layout was established in accordance with the requirements noted in the Request for Proposal and as allowed by existing underground and overhead services and utilities present at the site. PML selected the location of the boreholes and cone tests that are shown on the Borehole Location and Soil Strata Drawings 1 and 2. The ground surface elevations at the borehole and cone test locations were determined by PML and referred to geodetic benchmarks provided by DelBosco Surveying Limited. All elevations in this report are expressed in meters.

Soils were identified in accordance with the MTO Soil Classification Manual procedures. The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, where encountered, by measuring the groundwater level in the open boreholes. All of the boreholes were backfilled with a bentonite/cement mixture in accordance with the MTO guideline for borehole abandonment.

The recovered soil samples were returned to our laboratory for detailed visual examination and classification. The laboratory testing program consisted of 86 natural moisture content determinations, grain size distribution analyses of 20 selected soil samples and determination of Atterberg plasticity limits on 13 samples. A consolidation test on a cohesive sample and a moisture-density relationship test on a cohesionless sample were also carried out. The plasticity limits tests are shown on the plasticity charts PC-1 and PC-2 and summarized on Table 1. The laboratory grain size determinations are reported on Figures 1 to 4. The results of the consolidation test are presented on Figure 5. All of the test results are summarized on the Record of Borehole sheets.



4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole and Penetration Test sheets for details of the subsurface conditions including soil classifications, inferred soil stratigraphy, natural moisture content determinations, standard penetration and dynamic cone penetration test results, grain size analyses, plasticity limits, consolidation and moisture-density relationship test results and groundwater observations. Stratigraphic profiles prepared from the borehole data are presented on Drawings 1 to 4.

The typical subsurface soil stratigraphy at the site comprises discontinuous layers of fill, topsoil or peat at the ground surface overlying upper deposits of sand/sandy silt underlain by discontinuous silty clay/clayey silt layers that overlay discontinuous lower sand/and sandy silt deposits. These soils mantle granite/granitic gneiss bedrock at variable depths. The stratigraphy and groundwater observations that were revealed in the boreholes are summarized separately for each of the bridge foundations and for the north and south embankments in the following paragraphs.

4.1 South Abutment (Boreholes 1 and 2)

4.1.1 Topsoil

Layers of topsoil 100 mm thick were found at the surface of the two boreholes 1 and 2 drilled for the south abutment.

4.1.2 Sand

Cohesionless sand with silt was found below the topsoil and extended to about 1.2 and 1.4 m depths, common elevation 217.2. The relative density of the sand was very loose to compact. Penetration resistance N-values ranged from 2 to 14.

The particle size distribution chart of one sample of the sand from borehole 1 is shown on Figure 2. Water content determinations ranged from 18 to 21%.



4.1.3 Silty Clay

Underlying the sand at 1.2 and 1.4 m depths, cohesive very soft to soft silty clay trace sand with numerous silt layers was encountered to about 3.0 to 3.8 m depths, elevations 214.6 and 215.6 where bedrock was inferred by refusal to augering. N-values ranged from 0 (penetration under weight of rods and hammer) to 4 in these cohesive layers. A penetrometer test of 18 kPa was obtained in the stratum.

The particle size distribution chart of one sample of the silty clay is included in the envelope shown on Figure 3 and the Atterberg plasticity chart on Figure PC-1. Water content determinations ranged from 32 to 40%. The plasticity liquid limits on the sample from borehole 1 was 35 and the plastic limit was 20, indicating plasticity index 15.

4.1.4 Clayey Silt

A 0.80 m thick layer of clayey silt trace sand was encountered in borehole 1 below the silty clay and extended to the bedrock surface level at about 3.8 m depth, elevation 214.8. The clayey silt had a soft consistency with an N-value of 0 (penetration due to weight of hammer and rods). A penetrometer test of about 18 kPa was obtained on the clayey silt sample.

The particle size distribution charts of one sample of the clayey silt from borehole 1 is included in the envelope shown on Figure 4 and the plasticity limits are shown on the Figure PC-2 chart. The water content determination on the clayey silt sample was 31%. The Atterberg plasticity liquid limit on the clayey silt sample was 31 and the plastic limit was 19, indicating a plasticity index of 12.

4.1.5 Bedrock

Granitic gneiss bedrock was contacted at 3.8 m depth, elevation 214.8 in borehole 1 and inferred at the same depth, elevation 214.6 in borehole 2. The bedrock was cored for 3.6 m to a depth of 7.4 m, elevation 211.2. The rock was a very strong unweathered to slightly weathered rock formation with good to fair quality. The RQD values measured on the NQ rock cores from



borehole 1 ranged from 72 to 77%. Details of the rock core descriptions are provided on Table 2 and photographs were included in Appendix B.

4.1.6 Groundwater

Groundwater was present in the two boreholes at depths of 1.1 to 1.4 m, elevations 217.2 and 217.3. The water level is expected to vary seasonally.

4.2 Bridge Pier (Boreholes 3 and 4)

4.2.1 Fill

Fill comprising mixed sand, silt and gravel was encountered at the surface in the two boreholes drilled at the pier location located to the south of the existing highway platform. The fill extended to 1.4 to 1.8 m depth, elevations 218.0 and 218.5 in the boreholes 3 and 4. The fill was in a compact condition with N-values ranging from 11 to 22.

The particle chart distribution chart of one sample of the fill is enclosed in Figure 1. Water content determinations in the fill ranged from 4 to 18%.

4.2.2 Sand

Two layers of cohesionless sand with silt locally with gravel were contacted below the fill in the boreholes and extended to the bedrock surface at 6.1 to 6.7 m depths, elevations 213.1 and 213.8. The sand was interbedded with 2.0 and 3.2 m thick layers of clayey silt between 2.5 and 4.5 m (borehole 3) and between 2.4 and 5.6 m depths (borehole 4). The upper zone of the sand was in loose/compact condition and the lower zone was loose to dense. The N-values ranged from 5 and 10 in the upper layers to between 8 and over 50 blows in the lower layers.

Water content determinations were 20 and 23% in the upper layers and 12 to 18% in the lower layers.



4.2.3 Clayey Silt

The interbedding clayey silt layers in the boreholes contained occasional silty clay layers and were in a very soft to firm condition. The clayey silt extended from 2.5 to 4.5 m depths (elevations 215.4 to 217.4) in borehole 3 and from 2.4 to 5.6 m depths (elevations 214.2 to 217.4) in borehole 4. N-values were variable from 0 (penetration due to weight of hammer and rods) to 6.

The particle size distribution charts of two samples of the clayey silt from borehole 4 are included in the envelope shown on Figure 4 and the plasticity limits are shown on the Figure PC-2 chart. Water content determinations ranged from 32 to 41%. The Atterberg liquid limits on the clayey silt samples were 26 and 32 and the plastic limits were 16 and 19, indicating plasticity index values of 10 and 13.

4.2.4 Bedrock

Bedrock was contacted at depths of 6.1 and 6.7 m, elevations 213.1 and 213.8 and was cored 3.4 m in borehole 4 to 10.1 m depth, elevation 209.7. The contacted bedrock consisted of very strong, unweathered to slightly weathered granitic gneiss of good quality. The RQD values of the NQ rock cores ranged from 82 to 100%. Details of the rock core descriptions are provided on Table 3 and photographs were included in Appendix B.

4.2.5 Groundwater

The soils were typically wet in these boreholes and water was encountered at about 2.6 and 3.7 m depths, elevations 216.1 and 217.3 at the termination of the drilling.



4.3 North Abutment (Boreholes 5 and 6)

4.3.1 Fill

A 600 mm thick layer of fill was found at the surface of borehole 5 drilled at the north abutment location. The fill comprised a mixture of cohesionless sand, silt and peat inclusions in a loose condition. The fill extended to about elevation 218.9.

The water content determined in the fill was about 33%.

4.3.2 Topsoil

A 100 mm thick topsoil unit was encountered at the surface in borehole 6.

4.3.3 Silty sand

A layer of cohesionless silty sand trace clay was found below the surficial soil units and extended to about 2.2 m depth, elevations 217.2 to 217.3. This cohesionless unit exhibited a loose to compact condition with penetration resistance N-values ranging from 5 to 18 blows per 300 mm penetration of the sampler. The material is judged to be typically compact with a single N-value of 5.

The particle size distribution chart of one sample of the silty sand is shown on Figure 2. Water content determinations ranged from 19 to 30%.

4.3.4 Silty Clay

Underlying the sand, a layer of cohesive firm to soft silty clay trace sand with silt layers was encountered to about 4.2 to 4.4 m depths, elevations 215.0 to 215.3. N-values in this layer ranged from 2 to 5. The field vane test value of 94 kPa was likely affected by the silt layers present in the deposit.



The particle size distribution charts of two samples of the silty clay in boreholes 5 and 6 are included in the envelope shown in Figure 3 and the plasticity chart on Figure PC-1. Water content determinations ranged from 37 to 40%. The Atterberg liquid limits of the two samples were 37 and 38 and both plastic limits were 20, indicating plasticity index values of 17 and 18.

4.3.5 Clayey Silt

A layer of cohesive clayey silt trace sand was encountered below the silty clay and extending to 6.6 to 7.5 m depths, elevations 212.0 and 212.8, where the boreholes encountered refusal on bedrock. The clayey silt was very soft to firm with N-values between 1 and 4.

The particle size distribution charts of three samples of the clayey silt are shown on Figure 4 and the plasticity chart on Figure PC-2. Water content determinations ranged from 22 to 31%. The Atterberg liquid limits on the samples ranged from 24 to 27 and the plastic limits from 17 to 18 indicating plasticity index values from 7 to 10.

4.3.6 Bedrock

Bedrock was contacted at depths ranging from 6.6 to 7.5 m, elevations 212.0 and 212.8 in the boreholes and was cored for 3.2 m to 9.8 m depth, elevation 209.6 in borehole 6. The contacted bedrock consisted of relatively strong unweathered granite and granitic gneiss of excellent quality. The rock quality designation (RQD) values measured on the recovered NQ-size rock cores ranged from 93 to 97%. Details of the rock core descriptions are provided on Table 4 and photographs were included in Appendix B.

4.3.7 Groundwater

Groundwater was not detected in the two boreholes drilled at the site. The water level is inferred to be at about 0.5 m depth, elevation 219.0, based on the wet condition of the subsoil. The water level is expected to vary seasonally.



4.4 South Embankment (Boreholes 1, 2, 7, 8 and 9)

4.4.1 Topsoil

Discontinuous layers of topsoil 100 mm thick were found at the surface of four out of the five boreholes drilled in the south abutment area.

4.4.2 Sand

Cohesionless sand with silt/sand and silt deposits were found below the topsoil or at the surface and extended to about 1.2 to 1.5 m depths, elevations 216.9 to 217.5. The sand deposits had a very loose to compact relative density, typically increasing with depth. The penetration resistance N-values ranged from 2 to 21.

The particle size distribution charts of three samples of the sandy soils under the future south embankment are included in the envelope of particle size charts shown on Figure 2. Water content determinations ranged from 9 to 24%. The moisture-density relationship test carried out according to ASTM D-689 on a sample of the sand from borehole 7 indicated a maximum dry density of 17.8 kN/m³ and optimum moisture content of 12%.

4.4.3 Silty Clay

Underlying the sand at 1.2 to 1.5 m depths, a continuous layer of cohesive very soft to firm silty clay trace sand with numerous silt layers was encountered. The silty clay was penetrated at a depth of 3.0 m, elevation 215.6 in borehole 1, and the remaining four boreholes terminated by refusal on probable bedrock at 3.8 to 7.3 m, elevations 211.6 to 214.6. N-values ranged from 0 (penetration under weight of rods and hammer) to 6 in these cohesive layers. Penetrometer test results of 15 kPa and field vane tests results of 20 to 100 kPa were obtained in the stratum. Some of the higher values were likely inflated by the numerous silt layers in the deposit.

The results of a consolidation test carried out on a silty clay sample from borehole 9 are illustrated on Figure 5. The unconfined compression test on the sample provided a result of 59 kPa at a strain of about 8%. The particle size distribution charts of two samples of the silty



clay from the south embankment area are included in the envelope shown on Figure 3 and the Atterberg limits are shown on the plasticity chart on Figure PC-1. Water content determinations ranged from 32 to 48%. The plasticity liquid limits ranged from 35 to 36 and the plastic limits were 19 and 20, indicating plasticity index values of 15 and 17.

4.4.4 Clayey Silt

A 0.80 m thick layer of clayey silt trace sand was encountered in borehole 1 below the silty clay and extended to the bedrock surface level at about 3.8 m depth, elevation 214.8. The clayey silt had a soft consistency with an N-value of 0 (penetration due to weight of hammer and rods). A penetrometer test of about 18 kPa was obtained on the clayey silt sample.

The particle size distribution charts of one sample of the clayey silt from borehole 1 is included in the envelope shown on Figure 4 and the plasticity limits are shown on the Figure PC-2 chart. The water content determination on the clayey silt sample was 31%. The Atterberg plasticity liquid limit on the clayey silt sample was 31 and the plastic limit was 19, indicating a plasticity index of 12.

4.4.5 Bedrock

Granitic gneiss bedrock was contacted at 3.8 m depth, elevation 214.8 in borehole 1 and proved by a 3.6 m long core. Probable bedrock and bedrock were inferred at depths ranging from 3.8 to 7.3 m, elevations 211.6 to 214.8 in the boreholes drilled in the south embankment area. The bedrock was found at the deeper levels to the west of the embankment area (borehole 7). We refer to the bedrock description previously provided for the south abutment for a complete description of the bedrock.

4.4.6 Groundwater

Groundwater was present in four of the five boreholes at depths of 1.0 to 1.4 m, elevations 217.0 to 217.4. The water level is expected to vary seasonally.



4.5 North Embankment (Boreholes 5, 6, 10 to 19 and Cone Penetration Test 20)

4.5.1 Fill

Discontinuous fill units were encountered at the southern section of the north embankment immediately north of the Highway 11/17 (boreholes 5, 10 and 13) and under the northern section of the proposed embankment alignment (in borehole 19 and estimated in cone test 20). The fill comprised of mixtures of sand, gravel, silt, cobbles and boulders. The fill in the southern section of embankment locally contains peat inclusions and also large debris, such as concrete pipes and wood, based on visual observations, and extended to 0.6 to 2.1 m depths, elevations 218.8 to 219.1. At the northern end of the future embankment, the fill extended to about 2.7 m depth, elevation 220.6. The fill is generally in loose condition with compact zones. N-values ranged from 5 to over 50 with other values in the 8 to 16 range. The N-values are confirmed by the cone test 20 results in the 9 to 14 blows range.

The water content determinations on fill samples ranged from about 8 to 35%. The higher value was affected by the presence of peat inclusions in the sample.

4.5.2 Topsoil/Peat

Discontinuous layers of topsoil and peat typically 100 mm thick and locally 200 and 500 mm thick were found at the surface of eight out of the twelve boreholes drilled in the north embankment area.

4.5.3 Silty sand

Below the fill and/or topsoil/peat units the boreholes encountered a layer of cohesionless sand with variable amounts of silt and gravel. The sand units extended to depths varying from about 0.7 to 4.0 m depths, elevations 217.0 to 220.1, typically 2.0 to 2.9 m depths. This cohesionless unit exhibited a loose to compact relative density with penetration resistance N-values ranging from 4 to 22. The material is judged to be typically compact with typical N-values of 10 to 18.



The particle size distribution charts of three samples of the sand/silty sand are included in the envelope of Figure 2. Water content determinations ranged from 18 to 32%.

4.5.4 Silty Clay

Underlying the sand, a discontinuous deposit of cohesive firm to soft silty clay trace sand with silt layers was encountered under the southern section of the proposed north embankment (boreholes 5, 6, 10, 11 and 12). The silty clay extended to about 4.2 to 5.6 m depths, elevations 215.0 to 215.6. In borehole 12, the locally 1.0 m thick silty clay layer was interbedded from 3.2 to 4.2 m depths within a clayey silt deposit described on the following section of this report. Borehole 10 was terminated by refusal at 5.5 m elevation 215.4 within this deposit. The N-values in this layer ranged from 2 to 6. The field vane tests were 94 and 42 kPa; the higher value was likely inflated by the silt layers present in the deposit.

The particle size distribution charts of three samples of the silty clay in boreholes 5, 6 and 11 are included in the envelope shown in Figure 3 and the plasticity chart on Figure PC-1. Water content determinations ranged from 28 to 40%. The Atterberg liquid limits of the three samples ranged from 35 to 38 and the plastic limits from 20 and 21, indicating plasticity index values of 14 to 18.

4.5.5 Clayey Silt

Below the topsoil in borehole 18, sand/sandy silt and silty clay layers in the other boreholes a layer of cohesive clayey silt trace sand was encountered and extended to 1.2 to 7.5 m depths, elevations 212.0 and 219.4. The clayey silt layer in borehole 12 was interbedded with silty clay between 3.2 and 4.2 m depths. Boreholes 5, 6, 11, 15 and 18 terminated at the bottom of this deposit by refusal on probable bedrock. In the southern section of the embankment represented by boreholes 5, 6, and 11, 12 and 14, the clayey silt was very soft to firm with N-values between 1 and 4. In the northern section represented by boreholes 15, 17, 18 and 19 and cone test 20, the clayey silt was stiff to very stiff with typical values of 10 to 27 and values over 50 where the sampler encountered the bedrock surface.



The particle size distribution charts of five samples of the clayey silt are shown on Figure 4 and the plasticity chart, Atterberg limits are shown on the Figure PC-2. Water content determinations ranged from 22 to 36% in the southern section and between 18 and 28% within the northern stiff to very stiff section. The Atterberg liquid limits on the samples ranged from 20 to 31 and the plastic limits from 15 to 21 indicating plasticity index values from 5 to 10.

4.5.6 Sand/ Sandy Silt/ Silt

Discontinuous cohesionless deposits consisting of sand with silt, sandy silt or silt with sand and variable gravel content were encountered below the clayey silt units and extended to the termination depths of boreholes 12, 13, 14, 16, 17 and 19 and estimated in cone test 20. These boreholes terminated at depths ranging between 1.9 and 6.1 m, elevations 213.5 and 218.8 by refusal on probable bedrock. The cone test terminated at 5.8 m elevation 217.2 on competent soil. The relative density of the materials was variable, from loose to dense with N-values ranging from 7 to 48 and was judged to be typically compact.

Natural moisture content determinations in these cohesionless materials varied from 16 to 18%.

4.5.7 Bedrock

Bedrock was contacted at depths ranging from 6.6 to 7.5 m, elevations 212.0 and 218.9 in the boreholes and was proved by coring in borehole 6. A bedrock outcrop was noted between boreholes 16 and 17 west of the centreline of the embankment. We refer to the bedrock description previously provided for the north abutment for a complete description of the bedrock.

4.5.8 Groundwater

Most of the boreholes drilled along the north embankment encountered groundwater during and upon completion of drilling except boreholes 5 and 6. The water levels ranged from about 0.3 m above the ground surface in borehole 12, drilled from the top of the ice covering a local pond to about 2.7 m in borehole 19. The groundwater levels ranged between elevations 218.8 to 220.8.



The groundwater levels at this site are subjected to seasonal fluctuations and variations due to rainfall patterns.

5. CLOSURE

The subsurface investigation was carried out under the supervision of Mr. M. Rapsey and direction of Mr. C. M. P. Nascimento, P. Eng., Senior Project Engineer. Marathon Drilling Inc. supplied the drilling equipment. This report was prepared by Mr. C. M.P. Nascimento, P. Eng. and reviewed by Mr. Brian R. Gray, MEng, P. Eng, MTO Designated Principal Contact.

Sincerely,

Peto MacCallum Ltd.

A handwritten signature in cursive script, reading 'C. M. P. Nascimento'.

Carlos M.P. Nascimento, P.Eng.
Senior Project Engineer



A handwritten signature in cursive script, reading 'Brian R. Gray'.

Brian R. Gray, MEng, P. Eng,
MTO Designated Principal Contact





TABLE 1
LIST OF ATTERBERG PLASTICITY TEST RESULTS

SOIL TYPE	BOREHOLE / SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
SILTY CLAY Trace Sand	BH 1 / SA 3	35	20	15
	BH 5 / SA 4	37	20	17
	BH 6 / SA 5	38	20	18
	BH 9 / SA 4	36	19	17
	BH 11 / SA 5	35	21	14
CLAYEY SILT Trace Sand	BH 1 / SA 5	31	19	12
	BH 4 / SA 4	26	16	10
	BH 4 / SA 7	32	19	13
	BH 5 / SA 6	27	17	10
	BH 6 / SA 7	26	18	8
	BH 6 / SA 8	24	17	7
	BH 12 / SA 5	20	15	5
	BH 14 / SA 4	31	21	10



TABLE 2
ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
1	6	3.8 – 4.6	97	77	3.8 – 7.4	GRANITIC GNEISS: Pink and grey, fine crystalline, becoming dark grey, fine to medium crystalline, slight banding, with occasional white carbonate seams, occasional pyrite mineralization, high strength, unweathered to slightly weathered, close to moderate spaced dipping to vertical becoming flat partings, rough planar, oxidized to slightly altered, with rust colour on parting surface, good becoming fair quality.
	7	4.6 – 6.1	98	76		
	8	6.1 – 7.4	100	72		

RQD = Rock Quality Designation

Originated: JW
 Compiled: MR
 Checked: CN



TABLE 3
ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
4	10	6.7 – 7.6	100	89	6.7 – 10.1	GRANITIC GNEISS: Dark grey, fine crystalline, banded, with occasional white carbonate seams, high strength, unweathered to slightly weathered, close to moderate spaced flat to dipping partings, rough planar, tight to oxidized with rust colour on parting surface, one 20 mm thick silty infilling at 8.2 m depth, good becoming excellent quality.
	11	7.6 – 9.1	100	82		
	12	9.1 – 10.1	100	100		

RQD = Rock Quality Designation

Originated: JW
 Compiled: MR
 Checked: CN

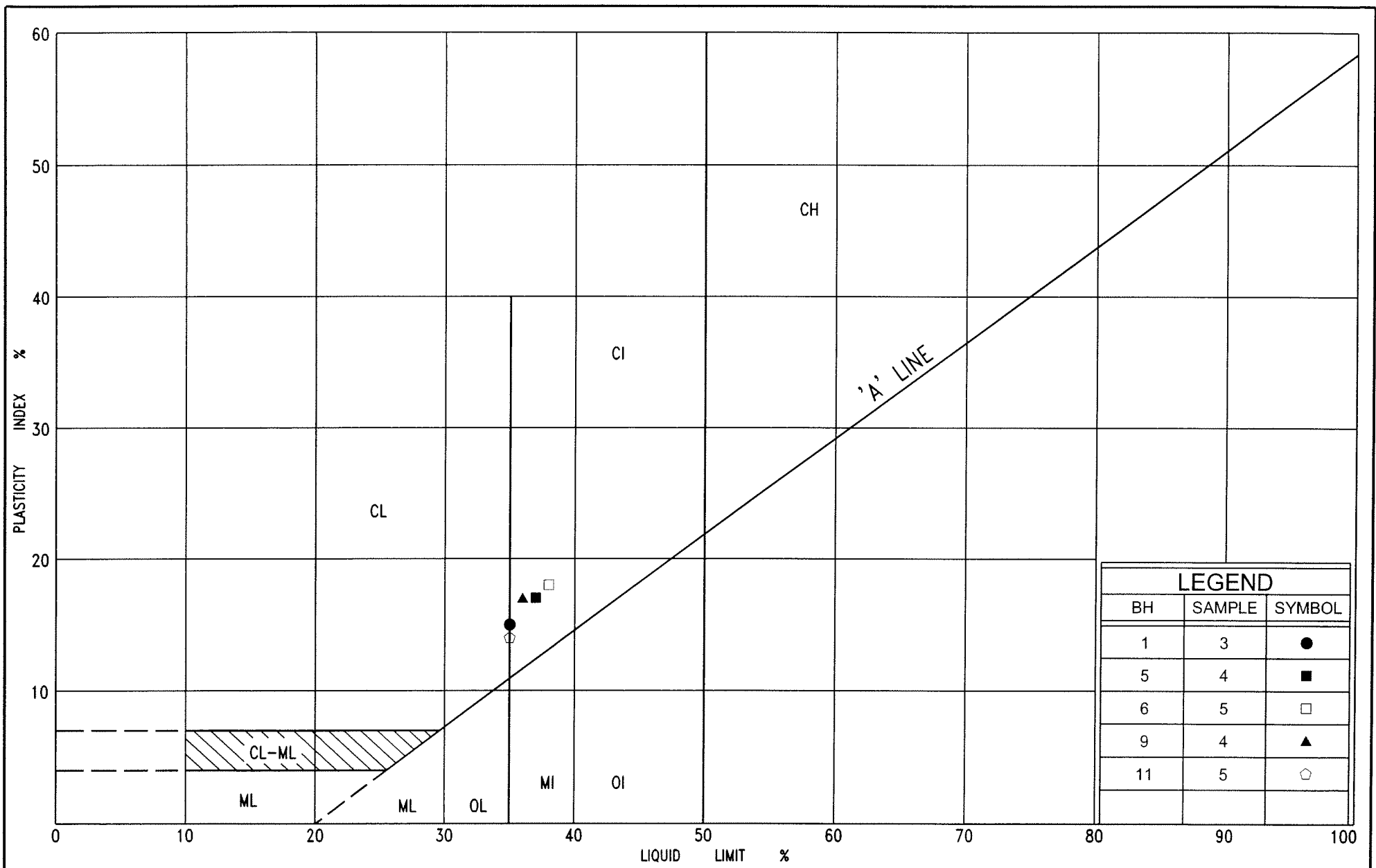


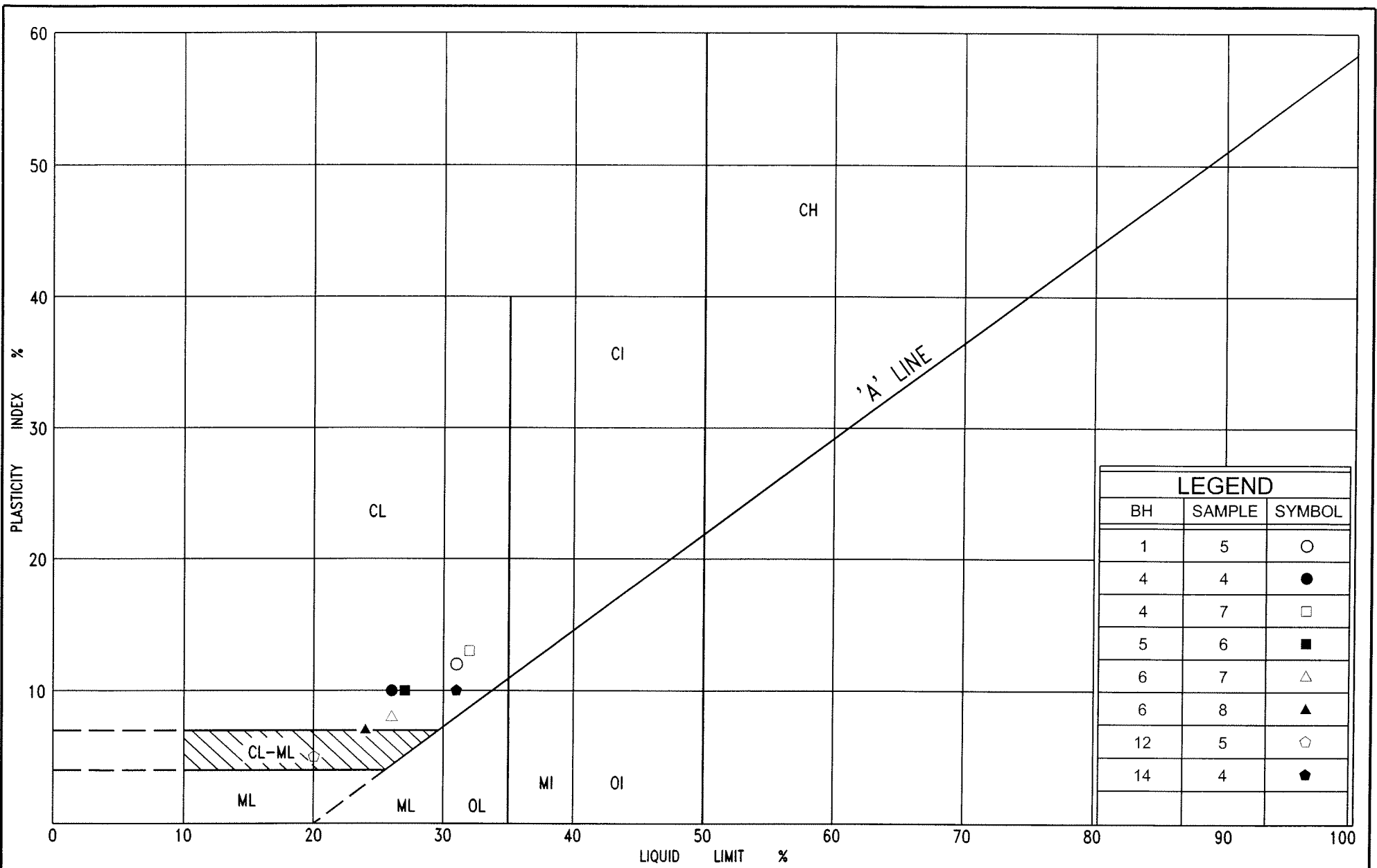
TABLE 4
ROCK CORE DESCRIPTION

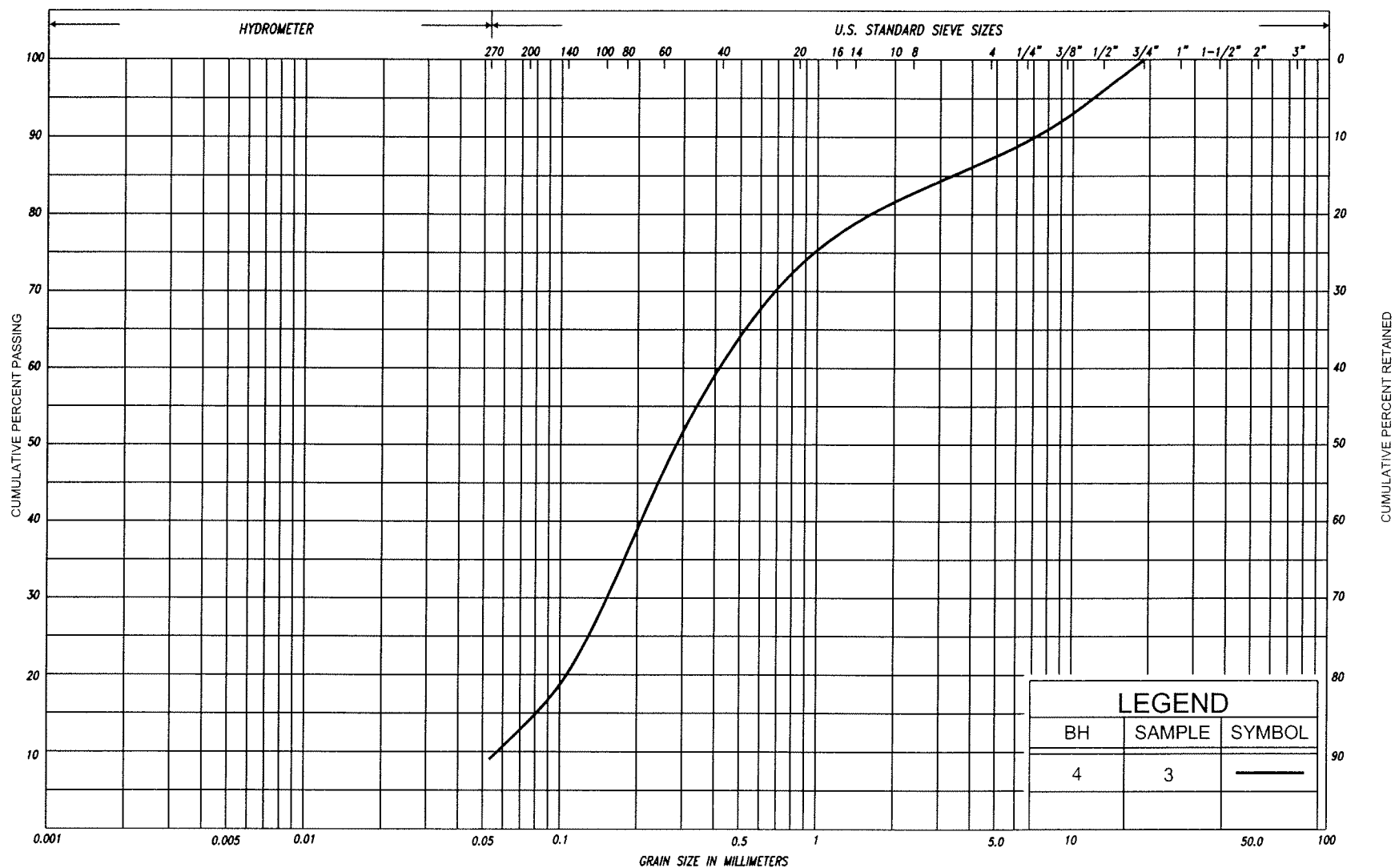
CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
6	10	6.6 – 7.7	100	93	6.6 – 7.7	GRANITE: Light grey, medium crystalline, high strength, unweathered, close to moderate spaced flat cross joints, rough planar, excellent quality.
	11	7.7 – 9.2	100	97	7.7 – 9.8	GRANITIC GNEISS: Dark grey, fine crystalline, becoming light grey, some pink, with vertical banding, high strength, unweathered, close spaced flat to dipping partings, becoming vertical, rough planar, tight to oxidized, generally dark on partings, occasional. thin white carbonate seams, excellent quality.
	12	9.2 – 9.8	100	95		

RQD = Rock Quality Designation

Originated: JW
 Compiled: MR
 Checked: CN

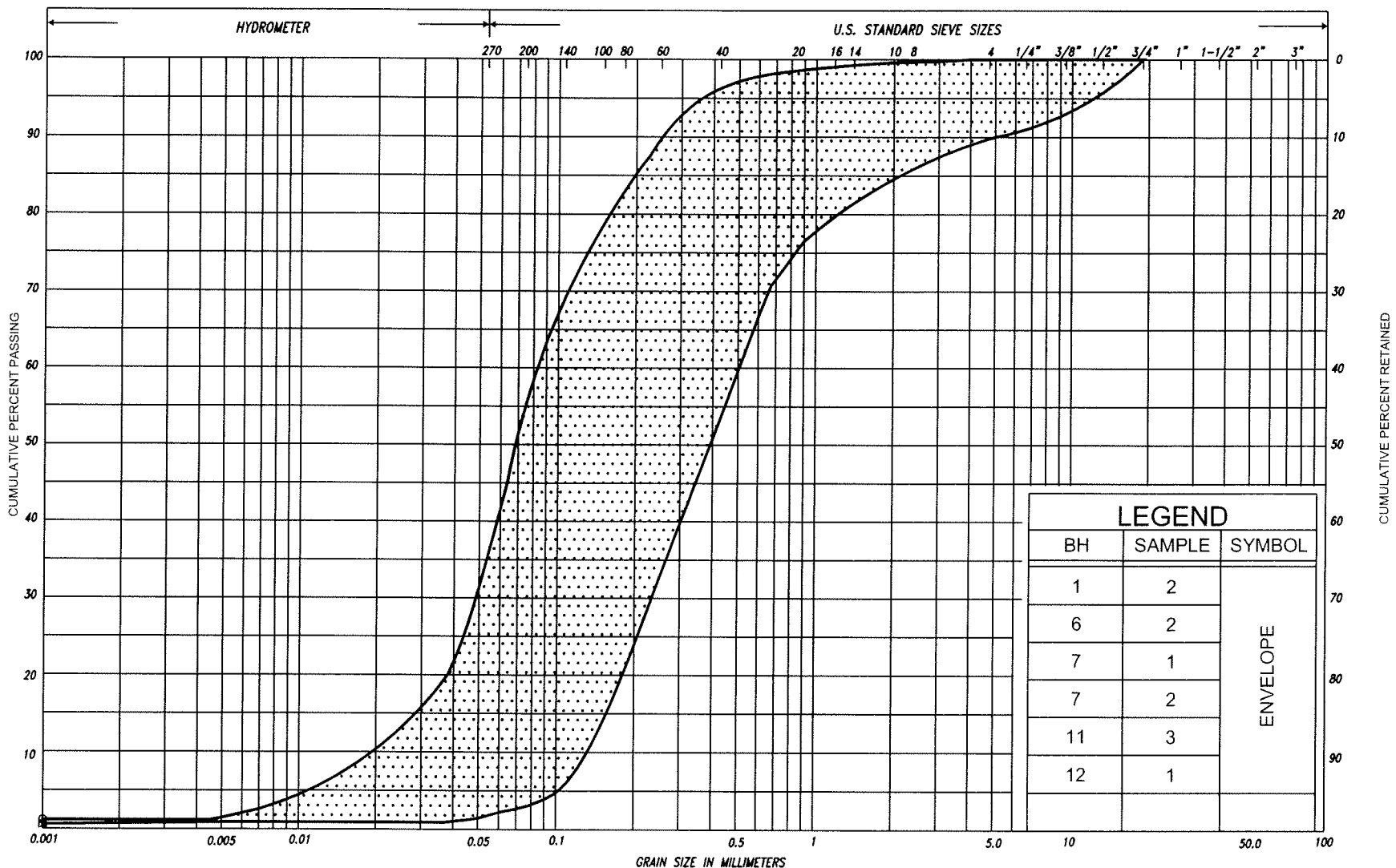




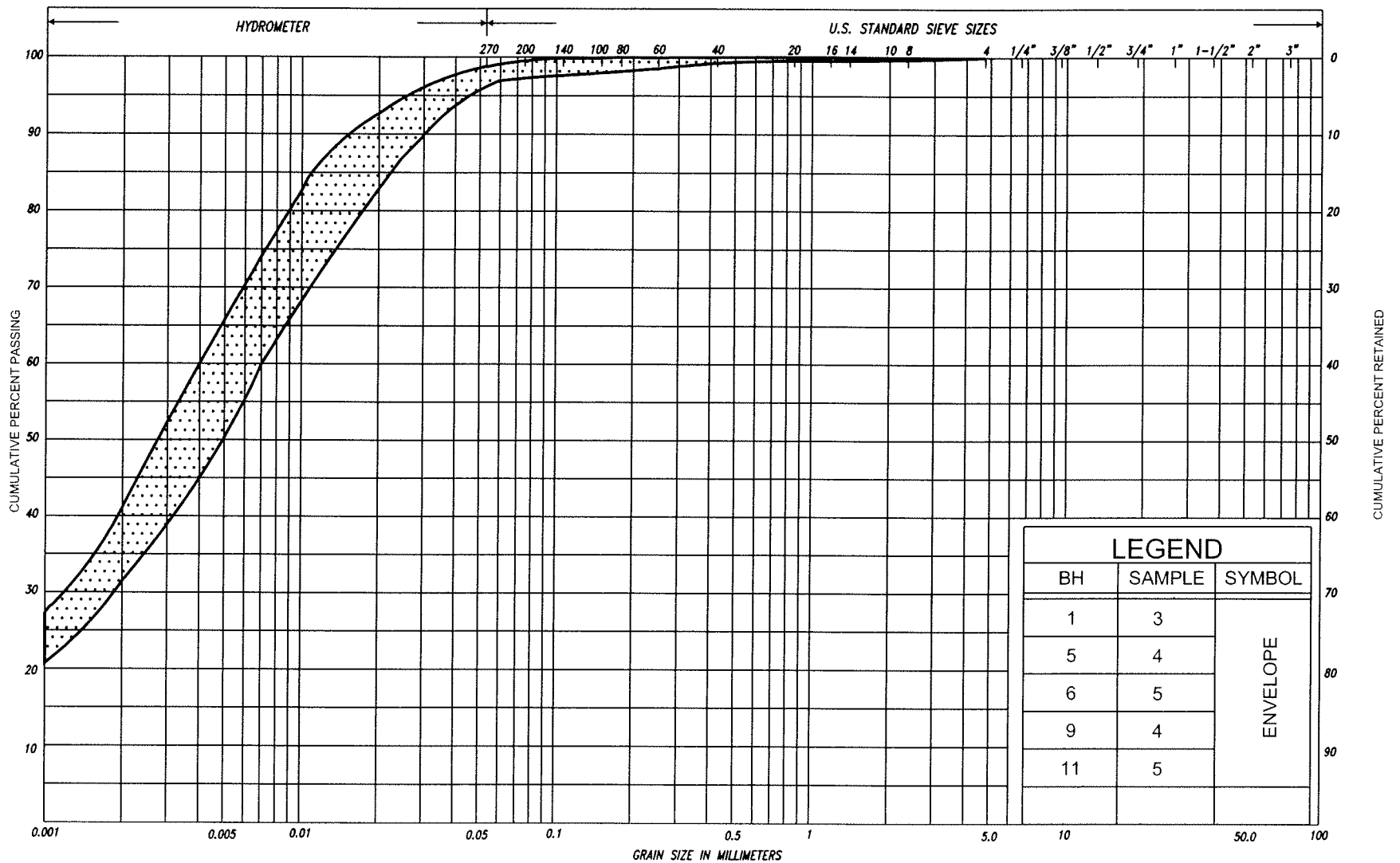


LEGEND		
BH	SAMPLE	SYMBOL
4	3	—

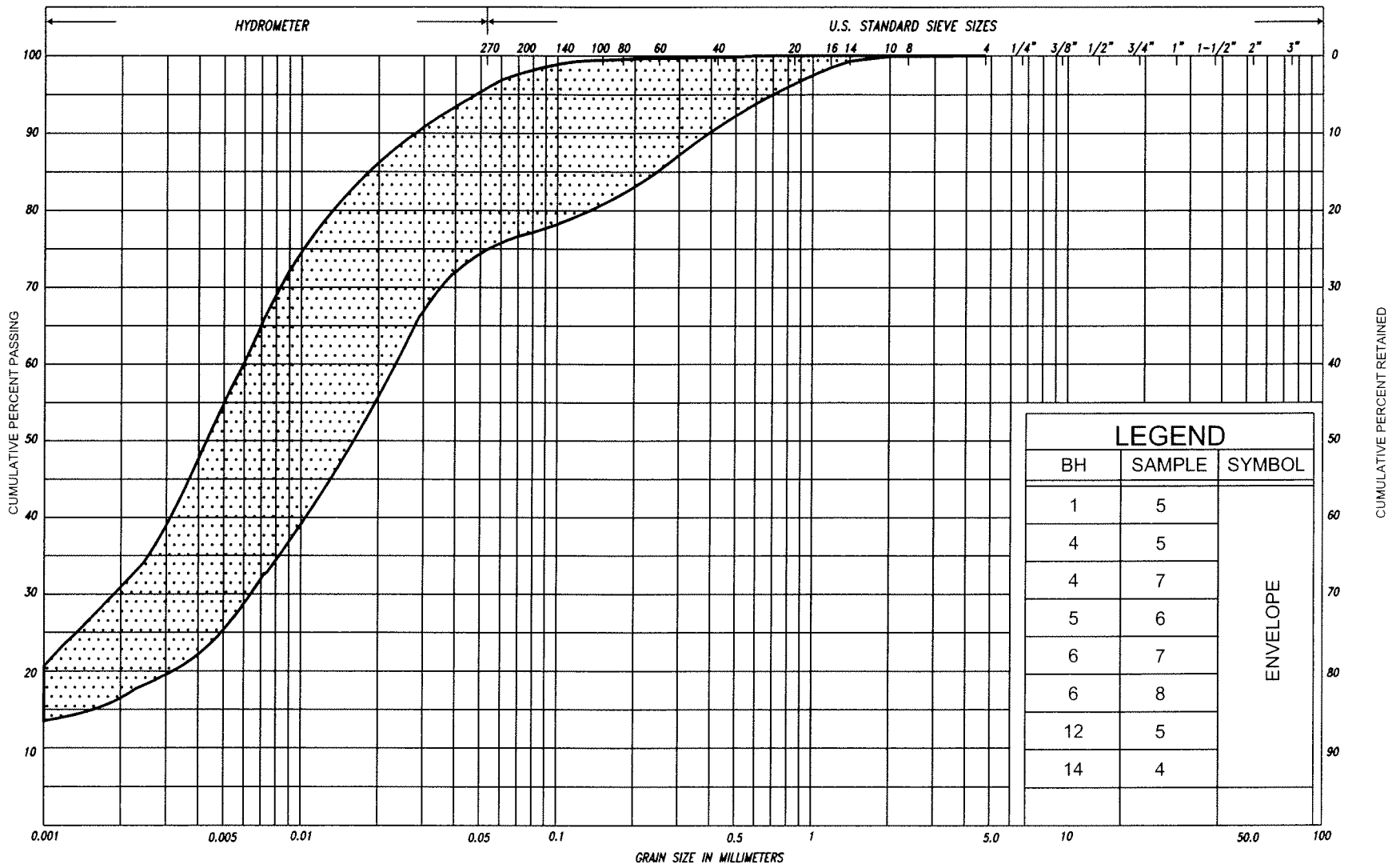
SILT & CLAY				FINE SAND			MEDIUM SAND		COARSE SAND		GRAVEL		COBBLES	UNIFIED
														M.I.T.
														U.S. BUREAU



SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED
CLAY	FINE		MEDIUM	COARSE	FINE		MEDIUM	COARSE	GRAVEL			COBBLES	M.I.T.
	SILT				SAND				GRAVEL				U.S. BUREAU
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL					
				SAND									



SILT & CLAY				FINE		MEDIUM		COARSE		GRAVEL				COB BLES	UNIFIED			
				SAND														
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.		
	SILT																	
CLAY		SILT			V. FINE	FINE	MED.	COARSE	SAND							GRAVEL		U.S. BUREAU



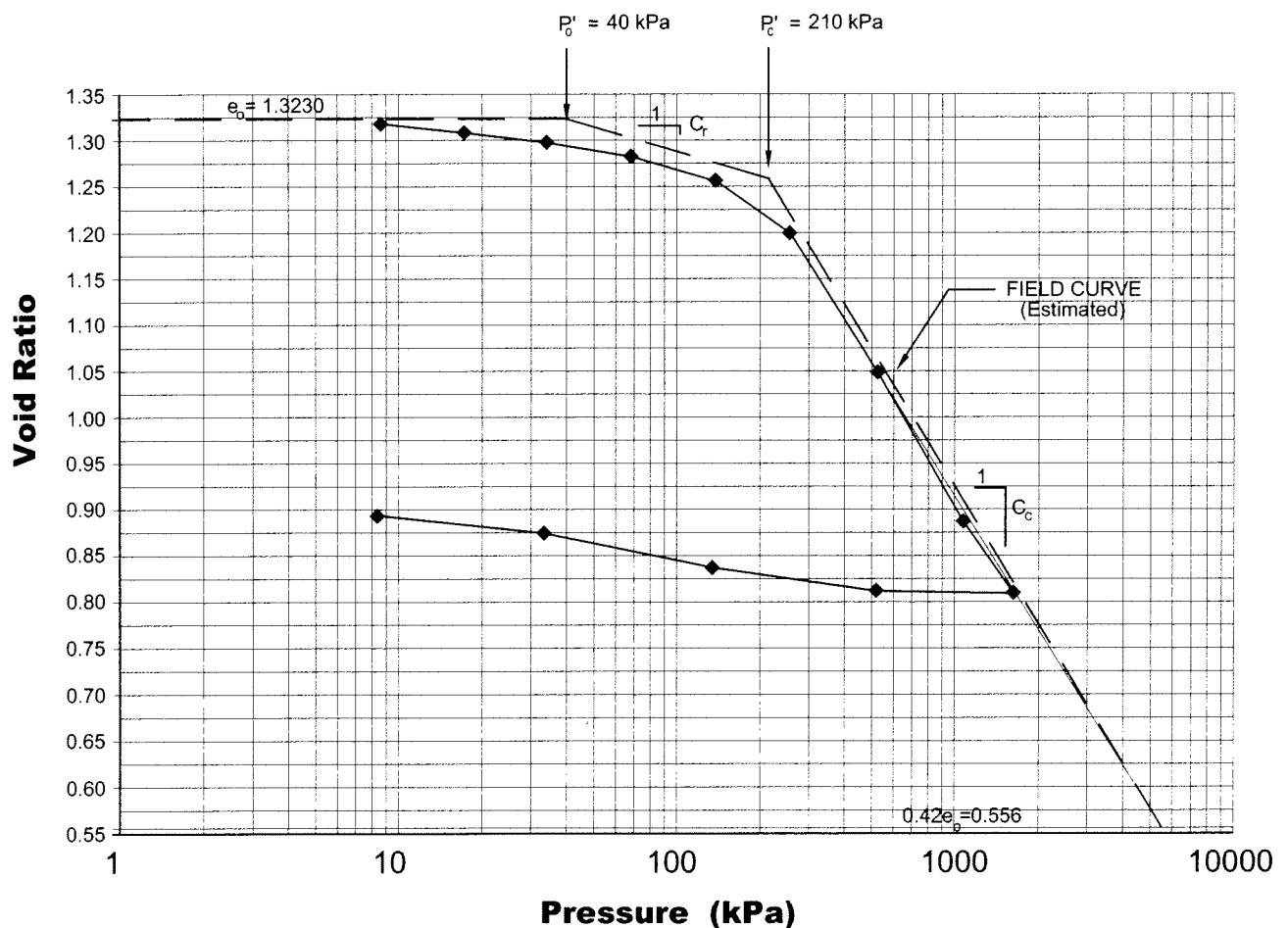
SILT & CLAY				FINE		MEDIUM	COARSE	GRAVEL	COBBLES	UNIFIED
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL	COBBLES	M.I.T.	
CLAY	SILT			V. FINE	FINE	MED.	COARSE	GRAVEL		U.S. BUREAU
				SAND						

Laboratory Consolidation Test Results

Highway 11/17
Pedestrian Structure
2.6km North of South Junction of Highway 11 and 17
District 54, Sudbury, North Bay, Ontario

BOREHOLE 9, SAMPLE 4
DEPTH 3.0 - 3.6 m

Void Ratio versus Log of Pressure



SOIL TYPE: SILTY CLAY, trace sand

$e_0 = 1.32$

$w_0 = 48\%$

$\gamma = 17.0 \text{ kN/m}^3$

$P'_0 = 40 \text{ kPa}$

$P'_c = 210 \text{ kPa}$

$C_c = 0.50$

$C_r = 0.06$

$W_L = 36$

$W_P = 19$

$PI = 17$

FIGURE: 5

HIGHWAY 11/17

CITY OF NORTH BAY

G.W.P. 5019-01-00

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	l	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	l	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	l	COMPRESSION INDEX
C_s	l	SWELLING INDEX
C_α	l	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	l	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	l	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	l, %	POROSITY	e_{max}	l, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	l, %	WATER CONTENT	e_{min}	l, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	s_r	%	DEGREE OF SATURATION	I_D	l	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	l	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	l	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	l	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	l	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kn/m ³	SEEPAGE FORCE
e	l, %	VOID RATIO						

METRIC

[illegible]

+⁷, ×⁵: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH 2

1 of 1

METRIC

G.W.P. 3159-05-00 LOCATION Co-ords: 5 131 732 N; 308 660 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE December 06, 2006 CHECKED BY C.N.

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 (%)				
218.4	Ground surface							20 40 60 80 100						
0.0	Topsoil													
0.1	Sand, with silt		1	SS	2		218							
	Very loose Brown Moist to loose													
213.2			2	SS	9									
1.2	Silty clay, trace sand thin layers of grey silt						217							
	Soft to Grey Wet very soft		3	SS	4									
							216							
			4	SS	WH**									
							215							
214.6			5	SS	1									
3.3	End of borehole Refusal on probable bedrock													

RECORD OF BOREHOLE No BH 3

1 of 1

METRIC

G.W.P. 5159-05-00 LOCATION Co-ords: 5 131 756 N; 308 677 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE December 04, 2006 CHECKED BY C.N.

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			20	40	60	80	100					
219.9 0.0	Ground surface																
	Sand with silt, trace gravel		1	SS	11		219										
	Compact Dark Brown (FILL)		2	SS	22												
218.5 1.4	Sand with silt, trace gravel		3	SS	10		218										
	Compact Grey Wet																
217.4 2.5	Clayey silt, trace sand silty clay layers		4	SS	4		217										
	Soft to firm Grey Wet		5	SS	6												
			6	SS	2		216										
215.4 4.5	Sand, with silt		7	SS	8		215										
	Loose Grey Wet		8	SS	46/ 15cm												
	with gravel																
213.8 6.1	Dense						214										
	End of borehole																
	Refusal on probable bedrock																
	2006 12 04																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																

METRIC

[illegible]

ON MOT VER3 06TF060-FINAL GPJ ON MOT.GDT 3/5/2007 10:42:55 AM

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 5

1 of 1

METRIC

G.W.P. 5159-05-00 LOCATION Co-ords: 5 131 781 N; 308 703 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE December 05, 2006 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT 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 (%)				
219.5 0.0	Ground surface Sand, with silt inclusions of fine fibrous peat		1	SS	5	219	20 40 60 80 100	+ FIELD VANE	20 40 60	o	0 1 67 32			
218.9 0.6	Loose Dark Wet brown (FILL)		2	SS	16									
	Silty sand, trace clay		3	SS	18									
	Compact Dark Wet brown		4	SS	5									
217.3 2.2	Silty clay, trace sand silty layers		5	SS	2									
	Firm to Grey Wet soft		6	SS	2									
215.3 4.2	Clayey silt, trace sand thin layers of grey silty clay		7	SS	4									
	Soft Grey Wet					215				o	0 2 82 16			
						214								
						213				o				
212.0 7.5	End of borehole Refusal on probable bedrock					212								
	Borehole dry on completion of drilling													
	Field vane results distorted due to silt layers													

RECORD OF BOREHOLE No BH 6

1 of 1

METRIC

G.W.P. 5154-05-00 LOCATION Co-ords: 5 131 776 N; 308 709 E ORIGINATED BY M.R.
 DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers+NQ Rock Coring COMPILED BY M.R.
 DATUM Geodetic DATE December 05, 2006 CHECKED BY C.N.

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 (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE									
219.4	Ground surface						20	40	60	80	100	20	40	60						
0.0	Topsoil		1	SS	5															
0.1	Silty sand, trace clay		2	SS	12											0 62 37 1				
	Loose Brown Wet to compact		3	SS	15															
217.2																				
2.2	Silty clay, trace sand silt layers		4	SS	5															
	Firm to Grey Wet soft		5	SS	3											0 1 65 34				
			6	SS	3															
215.6																				
4.4	Clayey silt, trace sand		7	SS	2											0 2 79 19				
	Very soft Grey Wet to firm		8	SS	1											0 3 80 17				
			9	SS	3															
212.8																				
6.6	Granite Bedrock Very strong Unweathered Excellent quality		10	RC NQ	REC 100%											RQD = 93%				
	Granitic Gneiss Bedrock Very strong Unweathered Excellent quality		11	RC NQ	REC 100%											RQD = 97%				
			12	RC NQ	REC 100%											RQD = 95%				
209.6																				
8.8	End of borehole																			

RECORD OF BOREHOLE No BH 7

1 of 1

METRIC

G.W.P. 5159-05-00 LOCATION Co-ords: 5 131 743 N; 308 647 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE December 06, 2006 CHECKED BY C.N.

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 (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+ FIELD VANE	×						LAB VANE
218.9	Ground surface						20	40	60	80	100	20	40	60		GR SA SI CL
0.0	Sand, some silt trace clay, trace gravel Compact Brown Moist		1	SS	10							○			17.8	8 76 (16)
	sand and silt		2	SS	21							○				0 46 53 1
217.5	Wet															
1.4	Silty clay, trace sand numerous thin layers of grey silt		3	SS	2								○			
	Firm Grey Wet		4	SS	5								○			
	Soft to firm		5	SS	WH**								○			
			6	SS	WH**								○			
	occ. thin layers of grey silt		7	SS	WH**								○			
	Reddish brown/grey															
			8	SS	WH**								○			
211.6	End of borehole															
2.3	Refusal on probable bedrock															
	Vane tests carried out in borehole drilled 1.5m southerly															
	* Borehole dry on completion of drilling															
	WH** Represents penetration due to weight of hammer and rods															
	Unit weight is maximum dry density for moisture-density relationship test (ASTM D698). Optimum moisture content=12%															

METRIC

$+^7, \times^5$. Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH 9

1 of 1

METRIC

G.W.P. 5159-05-00 LOCATION Co-ords: 5 131 718 N; 308 641 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE C. F. H. S. A. +Dynamic Cone Penetration Test COMPILED BY M.R.
DATUM Geodetic DATE January 15, 2007 CHECKED BY C.N.

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			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
218.4	Ground surface												
218.4	Topsoil		1	SS	7		218						
216.9	Sand, some silt Loose to Brown. Moist Compact		2	SS	19		217						
216.9	Wet						216						
215.9	Silty clay, trace sand num. thin layers of silt Firm Grey Wet		3	SS	6		215						
213.9	End of borehole		4	TW	-		214						
213.7	End of dynamic cone penetration test												
213.7	Refusal on probable bedrock												
4.5													
4.7													

RECORD OF BOREHOLE No BH 10

1 of 1

METRIC

G.W.P. 5-39-00-00 LOCATION Co-ords: 5 131 774 N; 308 722 E ORIGINATED BY M.R.
 DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
 DATUM Geodetic DATE January 16, 2007 CHECKED BY C.N.

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 (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE								
220.9	Ground surface					20	40	60	80	100	20	40	60						
0.0	Sand trace to with silt		1	SS	7														
	Loose Brown Moist																		
	with gravel		2	SS	9														
	with cobbles and boulders		3	SS	15/8cm														
218.8	(FILL)																		
2.1	Sand trace to with silt		4	SS	13														
	Compact Brown Wet		5	SS	18														
217.0																			
3.9	Silty clay num. thin layers of silt																		
	Firm Grey Wet		6	SS	4														
215.4																			
5.5	End of borehole																		
	Refusal on probable bedrock																		
					</														

METRIC

(%) STRAIN AT FAILURE

METRIC

$+^7, \times^5$: Numbers refer to Sensitivity

METRIC

(%) STRAIN AT FAILURE

METRIC


20
15 — 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No BH 15

1 of 1

METRIC

G.W.P. 5139-05-00 LOCATION Co-ords: 5 131 826 N; 308 740 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE January 16, 2007 CHECKED BY C.N.

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									
220.5	Ground surface																
219.8	Peat, fine fibrous Dark brown Sand, some silt		1	SS	4	▽* ▽*	220										
219.8	Loose Brown Wet clayey silt, trace sand layers of silty clay		2	SS	10		219										
218.4	Stiff Brown Wet		3	SS	12												
2.1	End of borehole Refusal on probable bedrock																
<div>* 2007 01 16</div> <div>▽ Water level observed during drilling</div> <div>▼ Water level measured after drilling</div>																	

* 2007 01 16

▽ Water level observed
during drilling

▽ Water level measured
after drilling

RECORD OF BOREHOLE No BH 16

1 of 1

METRIC

G.W.P. 5159-06-00 LOCATION Co-ords: 5 131 850 N; 308 723 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE January 15, 2007 CHECKED BY C.N.

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			20	40	60	80	100					
220.7	Ground surface																
0.0	Feat, fine fibrous Sand, some silt		1	SS	6	4*											
219.8	Loose Brown Wet						120										
0.9	Sandy silt		2	SS	22												
218.8	Compact Brown Wet																
1.9	End of borehole		3	SS	28/ 25cm		219										
	Refusal on probable bedrock																
	2007 01 15																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																

RECORD OF BOREHOLE No BH 17

1 of 1

METRIC

G.W.P. 9159-05-00 LOCATION Co-ords: 5 131 875 N; 308 720 E ORIGINATED BY M.R.
DIST Sjsbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE January 15, 2007 CHECKED BY C.N.

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
220.8	Ground surface							20	40	60	80	100					
0.0	Peat, fine fibrous		1	SS	4	▼* ▼*											
0.1	Dark brown Sand, with silt																
220.1	Very loose Brown Wet Clayey silt, trace sand		2	SS	12		220										
219.4	Stiff Brown Moist																
219.4	Sandy silt num. layers of silt, trace gravel and silty clay		3	SS	7		219										
1.4	Loose Grey Wet																
218.1	End of borehole Refusal on probable bedrock																
2.7																	
	2007 01 15																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																

RECORD OF BOREHOLE No BH 18

1 of 1

METRIC

G.W.P. 5159-05-00 LOCATION Co-ords: 5 131 900 N; 308 717 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.R.
DATUM Geodetic DATE January 16, 2007 CHECKED BY C.N.

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			20	40	60	80	100					
226.1	Ground surface																
0.0	Peat, fine fibrous Dark brown		1	SS	6	1*	220										
	Clayey silt, trace sand																
	Very stiff Brown Wet		2	SS	10/ 15cm		219										
218.9	End of borehole																
1.3	Refusal on probable bedrock																

* 2007 01 16

▽ Water level observed
during drilling

▽ Water level measured
after drilling

RECORD OF BOREHOLE No BH 19

1 of 1

METRIC

G.W.P.	5159-05-00	LOCATION	Co-ords: 5 131 925 N; 308 714 E	ORIGINATED BY	M.R.
DIST	Sudbury	HWY	11 & 17	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
DATUM	Geodetic	DATE	January 17, 2007	CHECKED BY	C.N.

[illegible]

RECORD OF PENETRATION TEST No BH 20

1 of 1 METRIC

G.W.P. 5159-05-00 LOCATION Co-ords: 5 131 924 N; 308 704 E ORIGINATED BY M.R.
DIST Sudbury HWY 11 & 1 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY M.R.
DATUM Geodetic DATE January 17, 2007 CHECKED BY C.N.

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
223.0	Ground surface							20	40	60	80	100			
0.0	Probable sand trace to with silt with gravel														
	Compact Brown Dry to moist														
	(Probable Fill)														
220.0															
3.0	Probable sand with silt														
	Compact Brown Wet														
219.0															
4.0	Probable clayey silt														
218.4	Very stiff Grey Wet														
4.6	Probable sand and gravel														
	Compact Brown Wet to dense														
217.2															
5.8	End of dynamic cone penetration test														

METRIC

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

CONT No
GWP No 5159-05-00

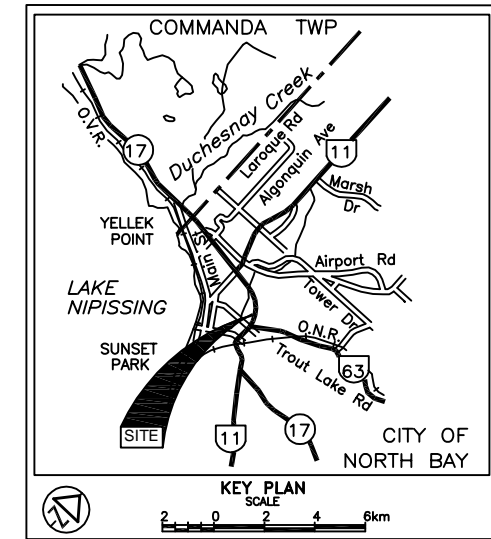
PEDESTRIAN UNDERPASS
HIGHWAY 11/17

BOREHOLE LOCATIONS & SOIL STRATA



SHEET

PMI Peto MacCallum Ltd.
CONSULTING ENGINEERS



LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60 Cone, 475 J / blow)
- W L at time of investigation Dec 2006-Jan 2007

BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
1	218.6	5 131 738	308 653
2	218.4	5 131 732	308 660
3	219.9	5 131 756	308 677
4	219.8	5 131 751	308 681
5	219.5	5 131 781	308 703
6	219.4	5 131 776	308 709
7	218.9	5 131 743	308 647
8	218.4	5 131 717	308 663
9	218.4	5 131 718	308 641
10	220.9	5 131 774	308 722
11	221.2	5 131 808	308 703
12	219.6	5 131 801	308 721
13	221.2	5 131 794	308 739

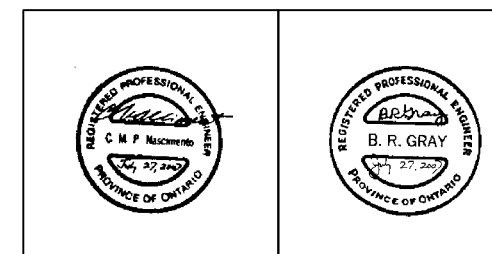
(Legend Continued on Dwg 2)

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

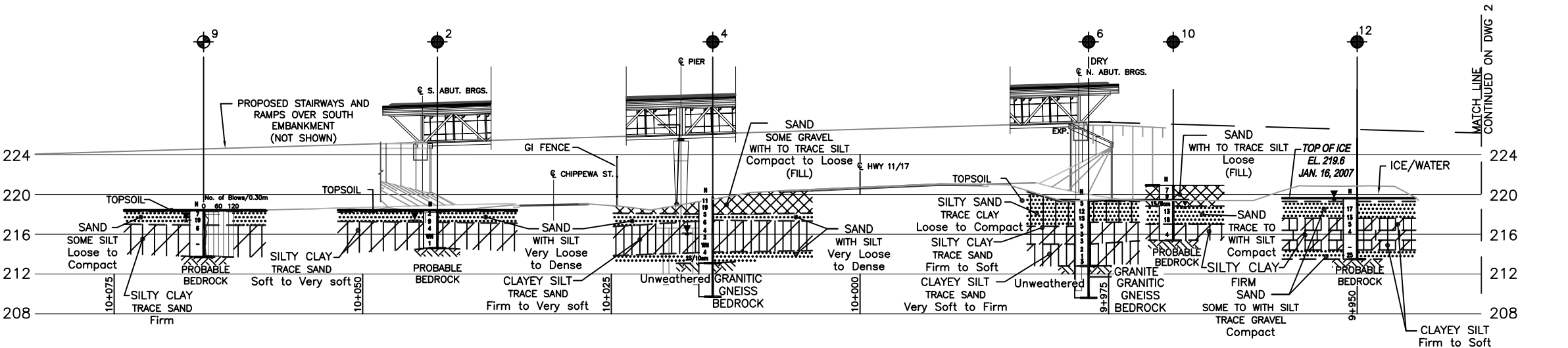
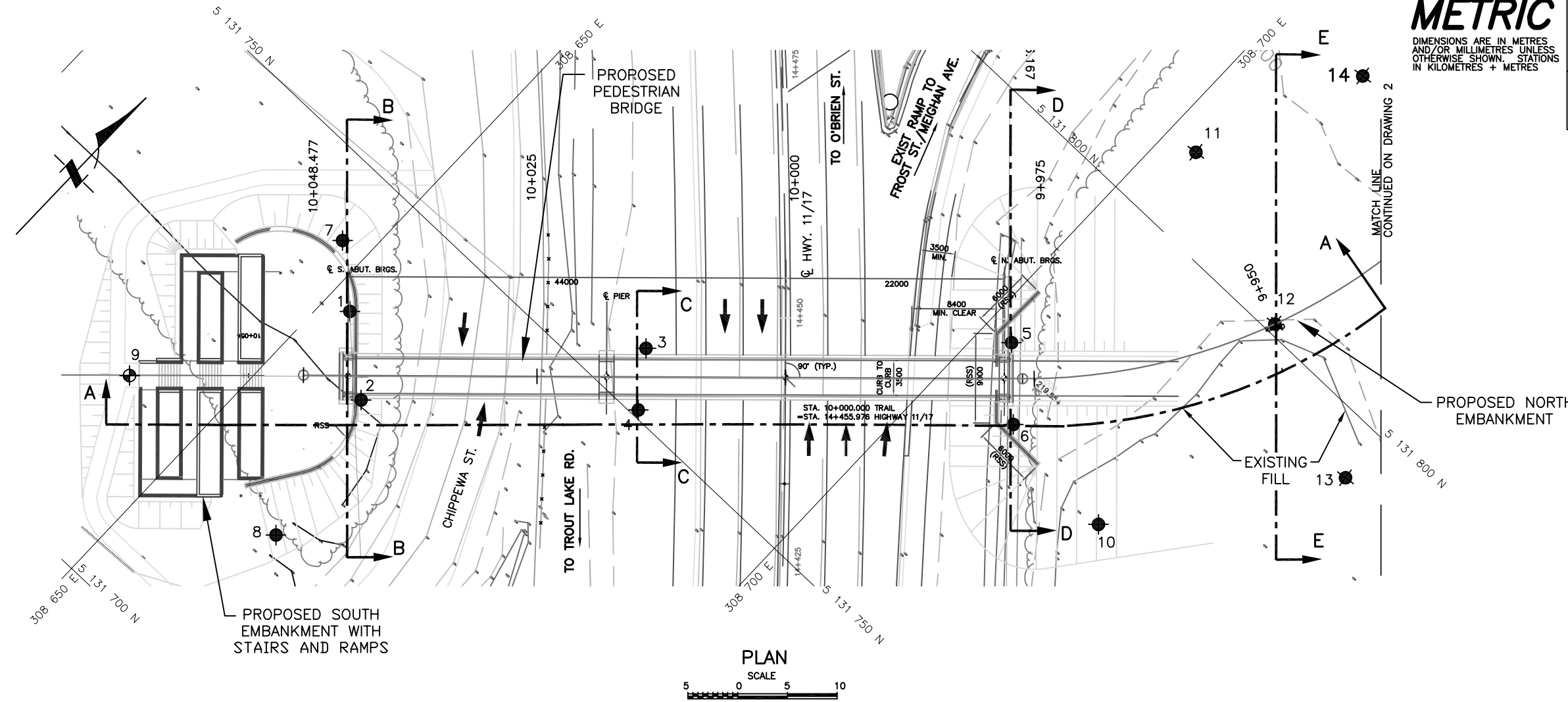
REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-110

HWY No	11 & 17	DIST	54
SUBM'D	GD	CHECKED	CN
DATE	JULY 27, 2007	SITE	43-370
DRAWN	NA	CHECKED	CN
APPROVED	BRG	DWG	1



REF.: STANTEC CONSULTING DRAWINGS
7219_ped_Bridge zone 10.dwg; 165000613_PED.GA



- NOTES:
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - FOR CONTINUATION OF PLAN SEE DRAWING 2 AND FOR SECTIONS SEE DRAWINGS 2, 3 AND 4.

METRIC

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

CONT No
GWP No 5159-05-00

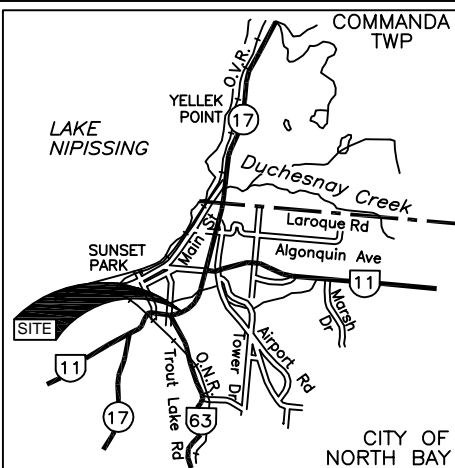
PEDESTRIAN UNDERPASS
HIGHWAY 11/17

BOREHOLE LOCATIONS & SOIL STRATA



SHEET

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

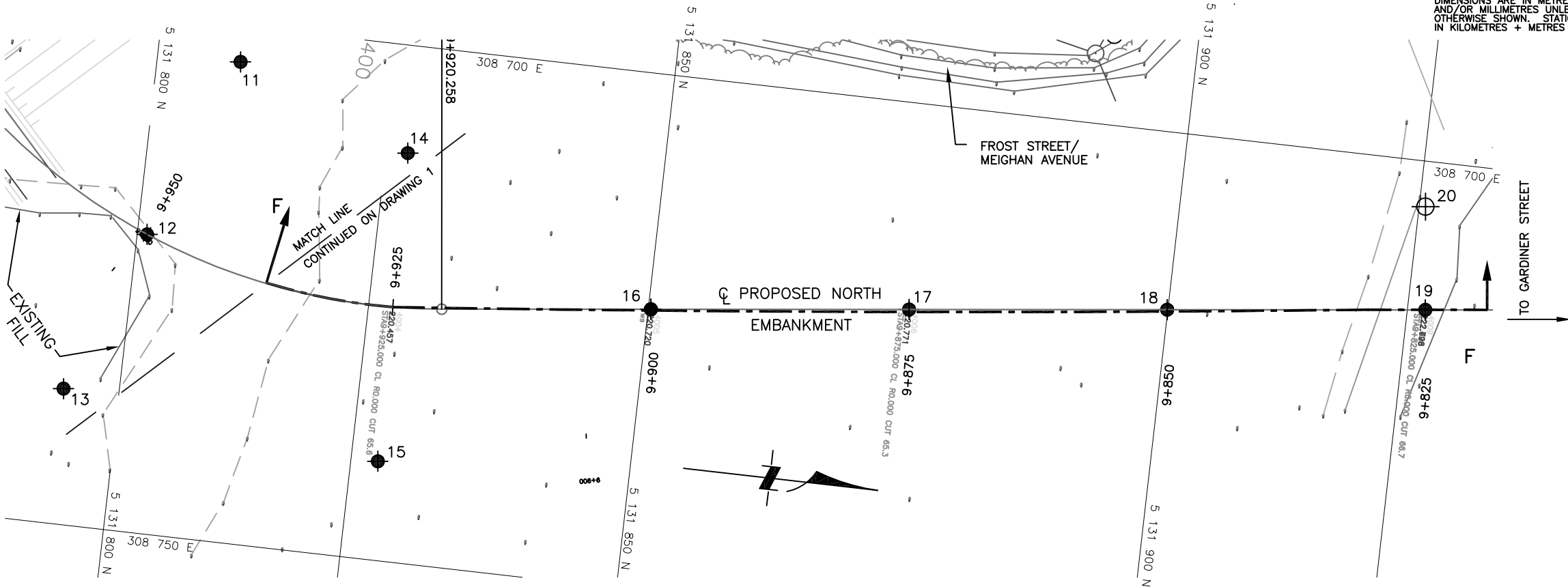


KEY PLAN
SCALE
0 2 4 6 km

LEGEND

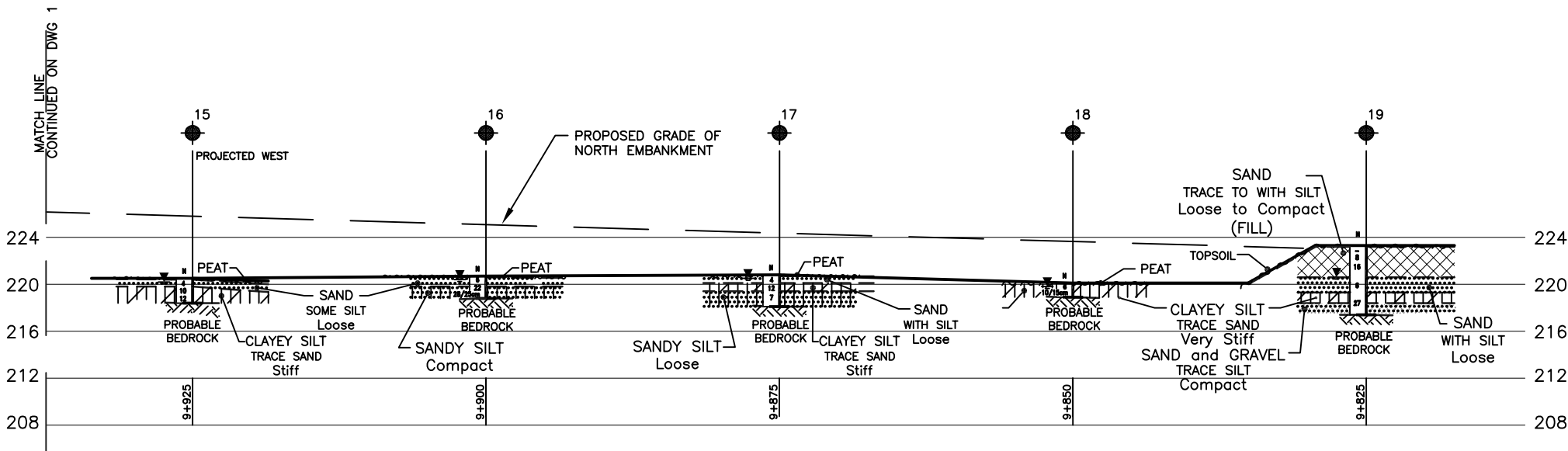
- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60 Cone, 475 J / blow)
- W L at time of investigation Dec 2006-Jan 2007

BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
(Legend Continued)			
14	220.5	5 131 825	308 710
15	220.5	5 131 826	308 740
16	220.7	5 131 850	308 723
17	220.8	5 131 875	308 720
18	220.1	5 131 900	308 717
19	223.3	5 131 925	308 714
20	223.0	5 131 924	308 704



PLAN (Continued)

SCALE
0 5 10



F - F

SCALE
0 5 10m

- NOTES:
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - FOR CONTINUATION OF PLAN AND SECTION SEE DRAWING 1.



NOTE

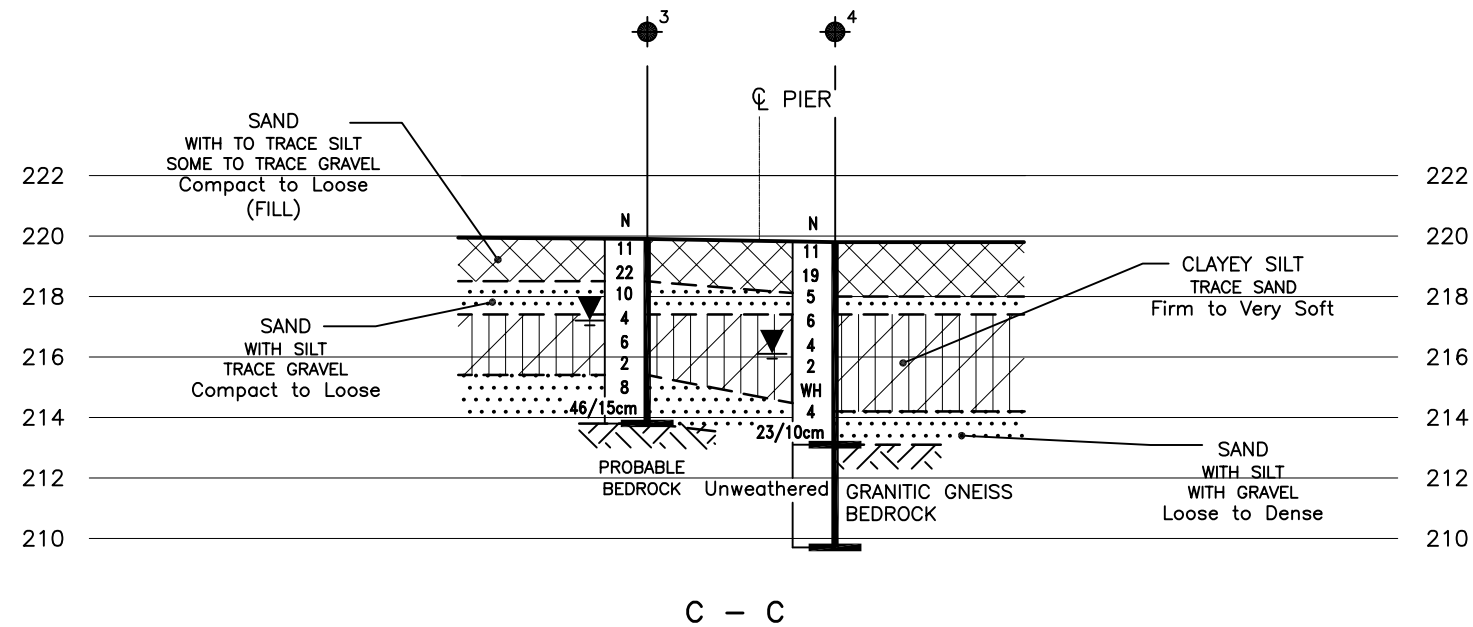
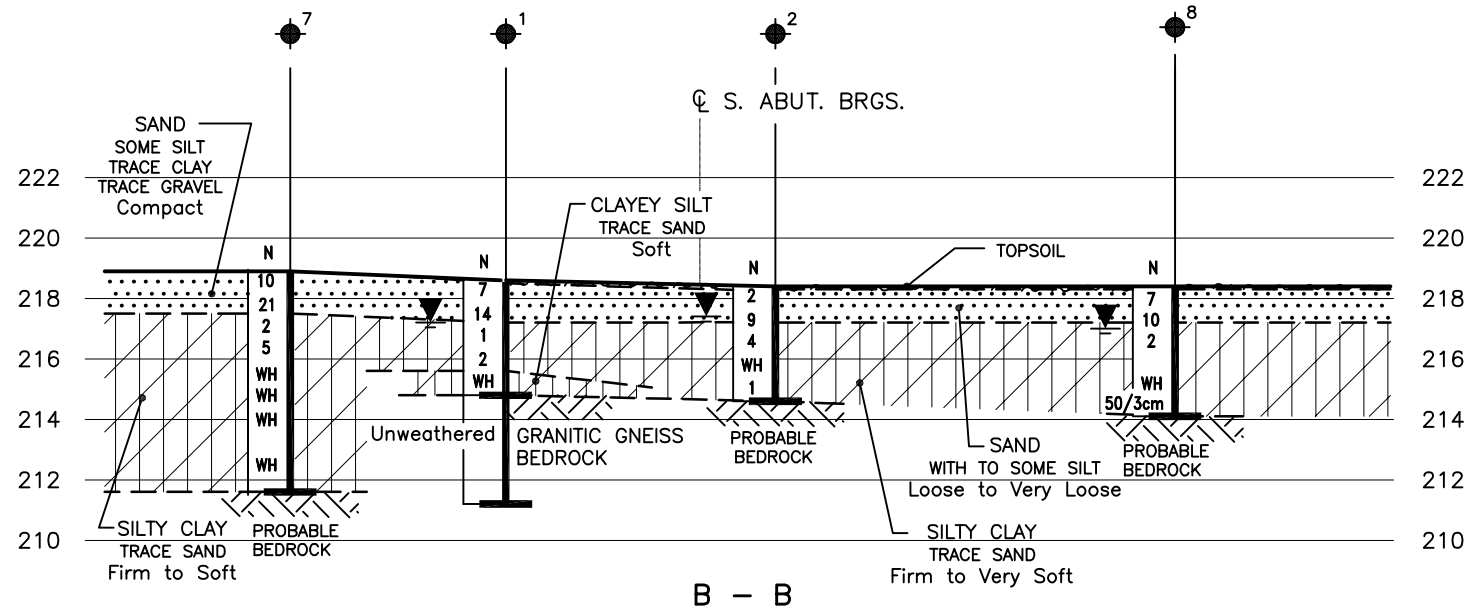
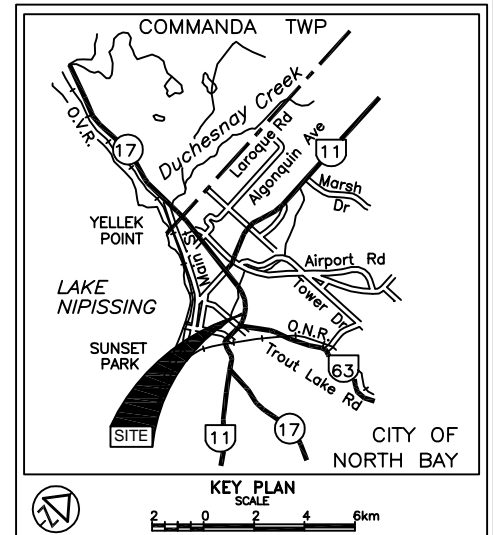
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

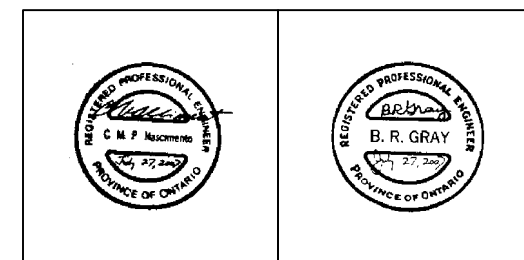
Geocres No. 31L-110

HWY No	11 & 17	DIST	54
SUBM'D	GD	CHECKED	CN
DRAWN	NA	CHECKED	CN
DATE	JULY 27, 2007	APPROVED	BRG
SITE	43-370	DWG	2

REF.: STANTEC CONSULTING DRAWINGS
7219_ped_Bridge zone 10.dwg; 165000613_PED.GA



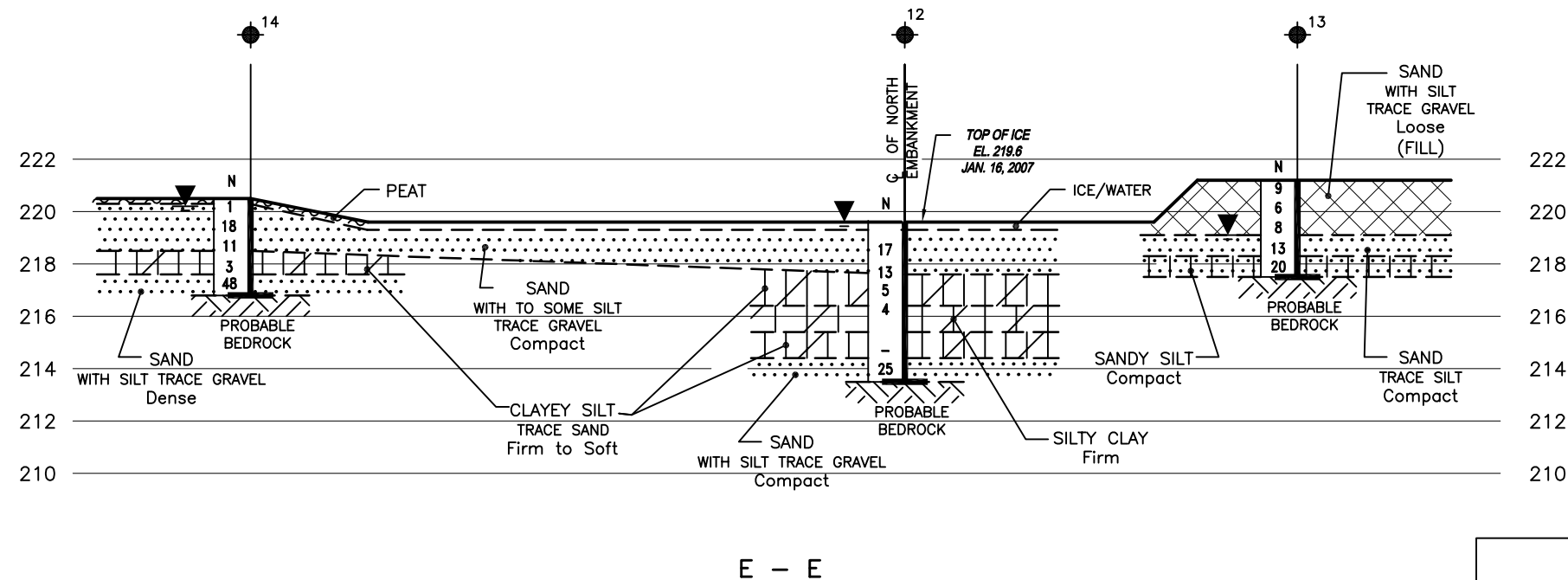
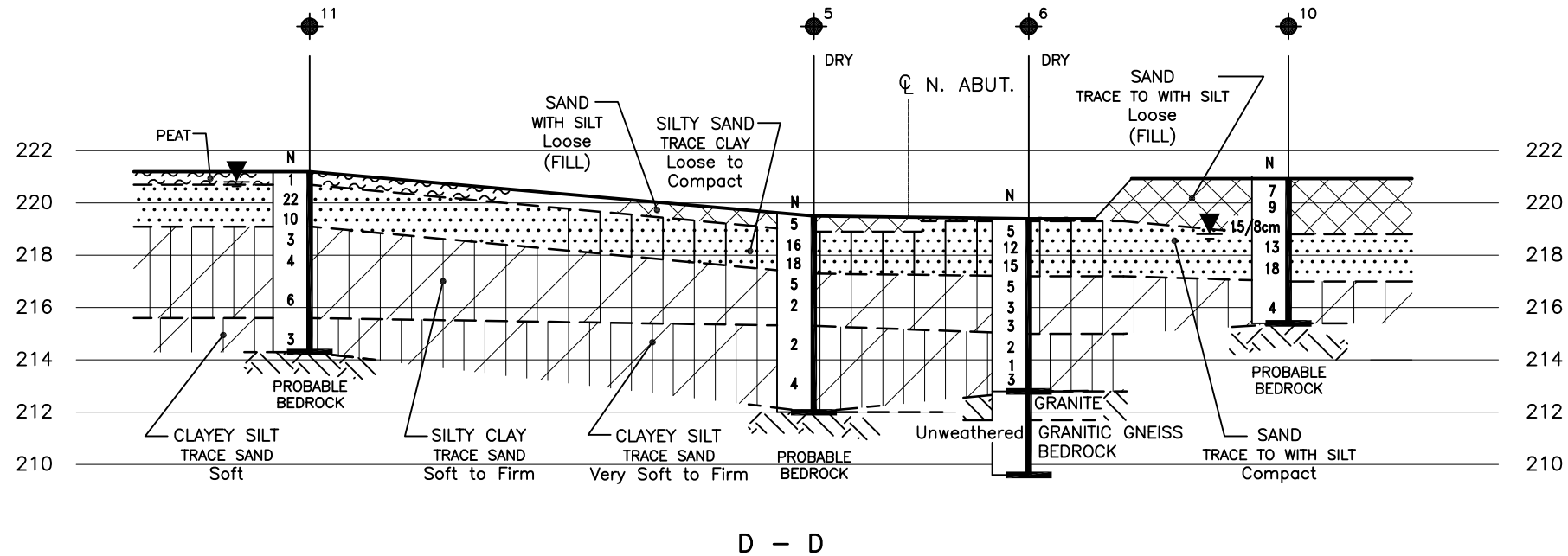
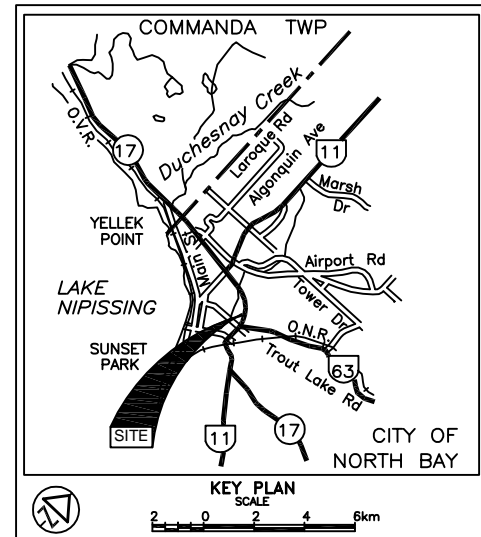
LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60° Cone, 475 J / blow)		
	W L at time of investigation Dec 2006-Jan 2007		
	Head		
	ARTESIAN WATER Encountered		
	PIEZOMETER		
BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
SEE DRAWING 1 FOR DETAILS			



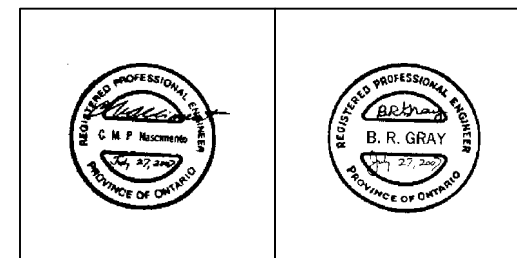
NOTE -
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-110			
HWY No	11 & 17	DIST	54
SUBM'D	GD	CHECKED	CN
DATE	JULY 27, 2007	SITE	43-370
DRAWN	NA	CHECKED	CN
APPROVED	BRG	DWG	3



LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60° Cone, 475 J / blow)		
	W L at time of investigation Dec 2006-Jan 2007		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
SEE DRAWING 1 FOR DETAILS			



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

REF.: STANTEC CONSULTING DRAWINGS
7219_ped_Bridge zone 10.dwg; 165000613_PED.GA

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 - FOR PLAN AND LOCATION OF SECTIONS SEE DRAWING 1.

REVISIONS					
	DATE	BY	DESCRIPTION		

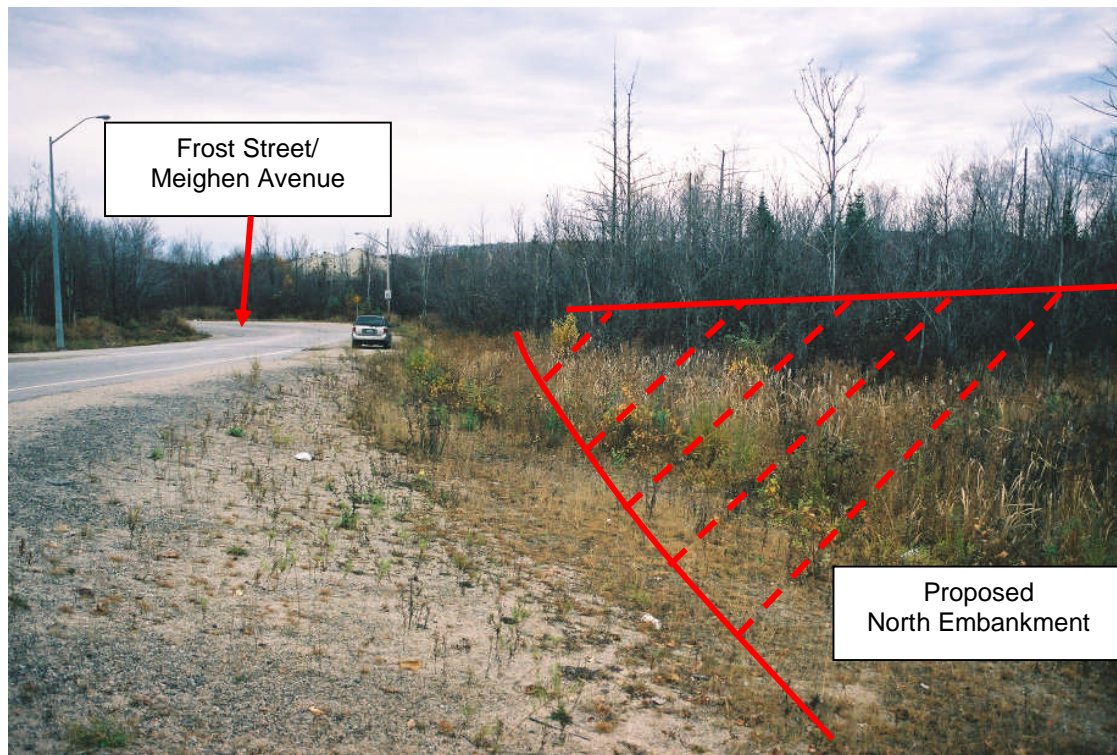
Geocres No. 31L-110

HWY No	11 & 17				DIST	54
SUBM'D	GD	CHECKED	CN	DATE JULY 27, 2007	SITE	43-370
DRAWN	NA	CHECKED	CN	APPROVED	BRG	DWG 4



APPENDIX A

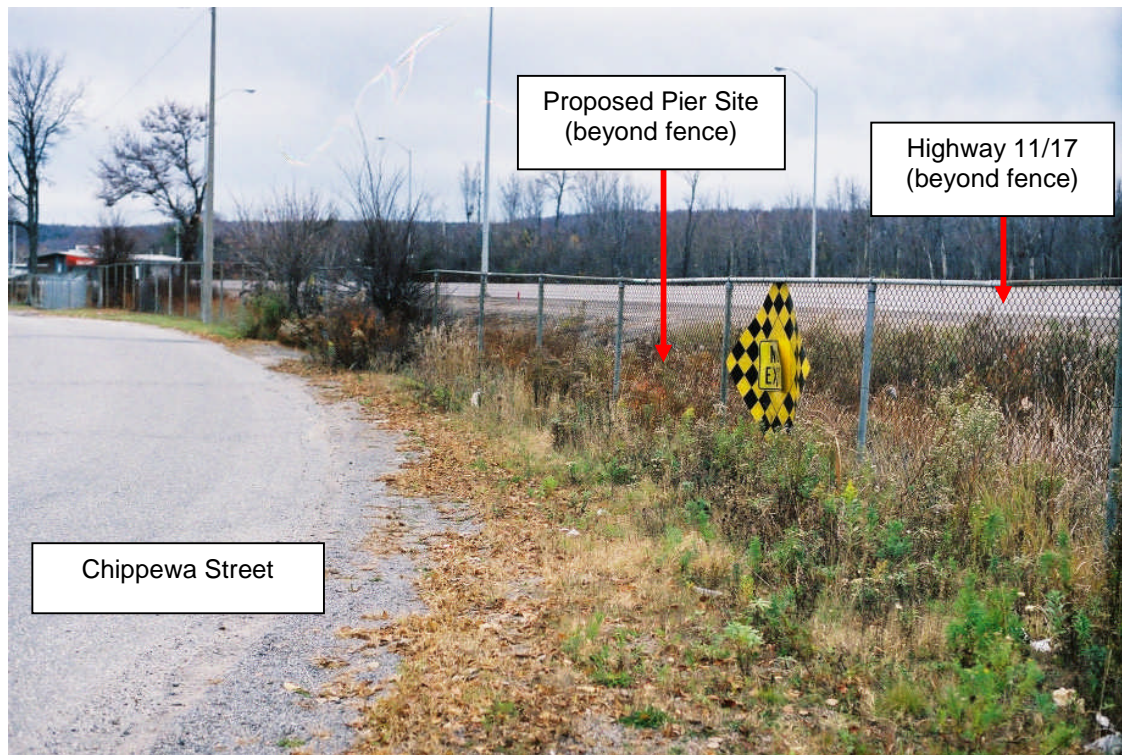
Site Photographs



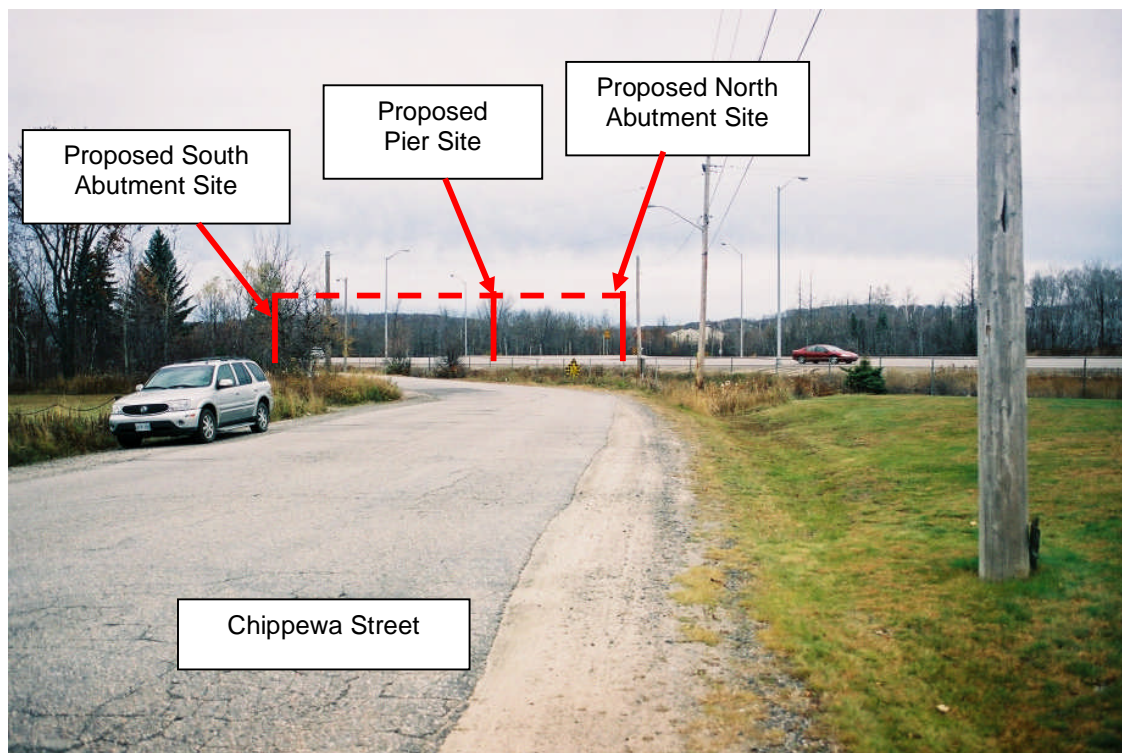
Photograph 1: Looking north along Frost Street/Meighen Avenue. Approximate location of proposed north embankment shown on photograph. (November 4, 2004)



Photograph 2: Looking west at off ramp from Highway 11/17. Proposed north abutment located beyond ditch at right of photograph. (November 4, 2004)



Photograph 3: Looking northwesterly from north end of Chippewa Street. Location of proposed pier is between fence and existing Highway 11/17. (November 4, 2004)



Photograph 4: Looking north along Chippewa Street. Location of south abutment and associated embankment is behind parked vehicle. (November 4, 2004)



APPENDIX B

Rock Core Photographs



BOREHOLE 1



BOREHOLE 4



BOREHOLE 6



**FOUNDATION DESIGN REPORT
FOR
PEDESTRIAN UNDERPASS - SITE NO. 43-370
OVER HIGHWAY 11/17, FROM MEIGHEN AVENUE/ FROST STREET
TO CHIPPEWA STREET
CITY OF NORTH BAY, ONTARIO
AGREEMENT NUMBER 5005-E-0069
GWP NO.: 5159-01-00**

PETO MacCALLUM LTD.
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Distribution:

- 5 cc: Stantec Consulting Ltd. for distribution to MTO,
Project Manager + one digital copy (PDF format)
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+ Drawings (AutoCAD format)
- 2 cc: Stantec Consulting Ltd.
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PML Ref.: 06TF060
Index No.: 046FDR
Geocres No.: 31L-110
July 31, 2007



TABLE OF CONTENTS

1. INTRODUCTION	1
2. BRIDGE FOUNDATIONS	4
2.1 General	4
2.2 Spread Footings	6
2.2.1 Footings on Native Soils.....	6
2.2.2 Footings on Engineered Fill.....	6
2.3 Deep Foundations	7
2.3.1 General	7
2.3.2 Driven Piles Axial Resistances	8
2.3.3 Driven Piles Lateral Resistance.....	9
2.3.4 Caisson Foundations Axial Resistances	10
2.3.5 Caisson Foundations Lateral Resistance.....	13
3. RETAINING WALLS	13
3.1 Retaining Wall Design	13
3.2 Sliding Resistance	16
4. EMBANKMENTS	16
5. EXCAVATION AND GROUNDWATER CONTROL	20
5.1 General Considerations	20
5.2 Road Protection Considerations	20
5.3 Groundwater Control Considerations	21
6. COMPARISON OF FOUNDATION ALTERNATIVES	22
7. CLOSURE	24

Table 1 – List of Standard Specifications Referenced in Report

Figure – Abutment on Compacted Fill Showing Granular 'A' Core



FOUNDATION DESIGN REPORT

for

Pedestrian Underpass - Site No. 43-370
Over Highway 11/17, from Meighen Avenue/ Frost Street
To Chippewa Street
City of North Bay, Ontario
GWP No. 5159-01-00

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of foundations, abutments and embankments for the proposed construction of a pedestrian underpass over Highway 11/17 in the City of North Bay, Ontario. The report was prepared for Stantec Consulting Limited (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The pedestrian underpass will be located immediately east of the off-ramp from Highway 11/17 at Meighen Avenue/Frost Street at about Sta. 14+450 (Highway 11/17 chainage). The bridge will extend from a location immediately east of the highway exit ramp at Meighen Avenue/Frost Street north of the highway to the north end of Chippewa Street on the south side of the highway.

It is understood that the bridge will be 66 m long and constructed as a two-span steel structure according to Drawing P1c "Preliminary General Arrangement" prepared by Stantec in November 2005. It is understood that the bridge was designed as a steel frame structure that will allow for future relocation. The south embankment will support a combination of precast concrete stairway and asphalt surfaced ramps with a plan area of about 30 by 35 m for pedestrian access to the south end of the structure. The north embankment will be about 170 m long and extend from the north end of the bridge at about Sta. 9+975 northerly to the intersection of Gardiner and Harrison Streets (Sta. 9+805).

We also understand that the maximum embankment heights are planned to be about 7.5 to 8.0 m high at the north abutment and 6.5 to 7.0 m high at the south abutment. The top of fill levels for the RSS walls are at about elevations 226.5 at the north and 224.5 at the south abutments.



In summary, the typical subsurface soil stratigraphy at the site comprises discontinuous layers of fill, topsoil or peat at the ground surface overlying upper deposits of loose to compact sand/sandy silt generally extending to about 0.7 to 4.0 m depths, elevations 216.9 to 220.1. At the north and south abutment locations the average thickness of the sand/sandy silt deposits is 1.9 and 1.2 m respectively. These cohesionless units are underlain by discontinuous cohesive deposits of typically very soft to firm silty clay/clayey silt layers to depths ranging from 1.2 to 7.5 m depths, elevations 211.6 to 219.8. At the north and south abutment locations the average thickness of the cohesive deposits is about 4.7 and 2.5 m thick, respectively. Under the proposed north embankment, from about Sta. 9+950 northerly, stiff to very stiff cohesive deposits overlay discontinuous typically compact sand/sandy silt soils extending to depths ranging from 1.9 to 6.1 m, elevations 213.5 to 218.8.

The soils mantle granite/granitic gneiss bedrock at variable depths ranging from 1.2 to 7.5 m, elevations 218.9 to 211.6 in the boreholes. The bedrock outcrops at about Sta. 9+885, to the west of the north embankment centreline.

The groundwater table at the site varies at each of the bridge foundation elements and embankments. In general, the groundwater table north of the highway was encountered at relatively shallower levels, from 0.3 m below the ground surface and in a shallow 0.3 m deep pond to 2.7 m depth (elevations 218.8 to 220.8). At the pier location, the groundwater was at about 2.6 and 3.7 m depths (about elevations 216.1 and 217.3). South of the highway, the groundwater level ranged from about 1.0 to 1.4 m depths (about elevations 217.0 to 217.4).

The findings of the investigation indicate that weak and compressible silty clay and clayey silt layers up to 5.3, 3.2 and 2.6 m thick underlay the bridge north abutment, pier and south abutment sites, respectively. Founding the abutments and pier on shallow footings bearing on these soils is not considered to be feasible due to the resulting excessive settlements. Although the south abutment and embankment could be founded on an engineered fill pad constructed on the underlying bedrock, the required removal of the 3.8 to 7.3 m deep silty clay and clayey silt layers to allow for the construction of the engineered fill pad would likely require the temporary installation of a costly dewatering system and road protection scheme near the Chippewa Street shoulders.



In view of the relatively light structure proposed for the pedestrian bridge it is considered that spread footings perched on engineered fill incorporated into the RSS wall fill is a feasible option. To this end, the RSS wall sites should be preloaded to reduce the settlements under the RSS wall fill and achieve sufficient subgrade strength gain to allow construction of the final fill height with an adequate safety factor against slope failure.

The new Pedestrian Underpass may also be established on the deep foundations, such as driven piles or caissons bearing on the bedrock that was found at the abutment and pier sites at 3.8 to 7.5 m depths. The foundations may also be established on alternative foundations such as minipiles installed into the bedrock. These schemes are typically designed and installed by specialist contractors.

We anticipate that the construction of the abutments and selected sections of the approach embankments will require preloading of the cohesive silty clay and clayey silt deposits to minimize the effect of long term settlements on the proposed RSS walls and precast stairways and reduce the negative skin friction on the piles or caisson foundations at the abutments. It is also anticipated that the first stage of the embankment construction should be no higher than 5.0 m for adequate stability of the slopes. Where the embankment is higher than 5.0 m, the upper layer of fill should be placed only after a 1.5 month preloading interval to allow for partial consolidation and strength gain of the cohesive soils.

It is considered that construction of the planned retaining walls at the abutments as RSS walls is feasible. The RSS walls should be selected to allow for minor settlements of the subgrade to occur without damage. At the south embankment where the proposed ramps and stairways are perched on the embankment fill, care should be exercised to preload the site in view of reducing future settlements. Alternatively, and particularly if no settlements are tolerated, the retaining wall footings could be founded on deep foundations. Special foundation schemes, such as minipiles driven/installed on the underlying bedrock surface may be considered for the RSS walls. These alternative foundations schemes are designed and installed by specialist contractors.

It is recommended that the construction of the north embankment be preceded by draining of the existing shallow pond and local drainage improvements to divert surface water away from the



proposed embankment subgrade. Since the surface water is relatively shallow due to the relatively pervious sandy soil at the surface underlain by relatively impervious clayey soils, the surface water run-off should be diverted away from the work site by means of strategically placed perimeter ditches to keep the excavations and subgrade in the dry. Grubbing and removal of local deleterious soils should be carried out with lightweight equipment to avoid punching these materials into the sandy subgrade. Immediately following this activity, the subgrade should be covered with a layer of geogrid and coarse gravel or crushed rock to provide a stable working surface for the construction.

Existing services and utilities under the new embankments should be relocated. Adequate compaction of the trench backfill should be provided to prevent post-construction settlements.

It is noted that the consultants assume no responsibility or liability for alerting the contractor and to “red-flag” all critical issues. The requirement to deliver acceptable construction quality remains the responsibility of the contractor.

A list of the standard specifications referenced in this report is compiled in Table 1. All elevations in this report are expressed in metres.

2. BRIDGE FOUNDATIONS

2.1 General

The native cohesive soils are typically soft to firm and generally considered not suitable to support the new structure foundations on spread footings since these soils are excessively compressible in the short and long term. Although the south bridge abutment foundation could be placed on an engineered fill pad which would be constructed after removing the 3.8 to 7.3 m thick silty clay and clayey silt layers, the construction of such an engineered fill pad is not considered to be practical for the proposed structure since it would require the installation of costly temporary groundwater control and road protection schemes near the Chippewa Street.



Cognisant of the relatively light and flexible structure being proposed, the abutments may be placed on perched footings bearing on the fill used for the support of the RSS wall. For this option, however the abutment sites will have to be preloaded to induce the substantial majority of the settlement under the total fill height.

The new pedestrian bridge structure foundations may also be founded on deep foundations such as driven piles or caissons founded on the bedrock underlying the abutment and pier sites at 3.8 to 7.5 m depths. The foundations may also be established on alternative foundations such as minipiles installed into the bedrock in particular at the south abutment where bedrock is only about 3.8 m below the ground surface. These minipiles schemes are typically designed and installed by specialist contractors.

Use of steel H-piles to support the abutment foundation loads will be dictated by structural design considerations such as minimum pile length. It is considered that due to portability requirements for the bridge construction, the integral or semi-integral abutment alternatives are not applicable to the project.

The seismic site coefficient for the stratigraphic conditions at this site is 1.0 [soil profile Type I, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6].

Based on the grain size and relative density/consistency of the soil cover at the site, it is considered that liquefaction of the soil is unlikely to occur (refer to clause 4.6.2 of the CHBDC).

All footings or pile caps subject to frost action should be provided with 2.0 m of earth cover (OPSD 3090.100) or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover.



2.2 Spread Footings

2.2.1 Footings on Native Soils

Construction of the spread footings on native soils is not considered practical at this site in view of the weak and compressible subsoil conditions.

2.2.2 Footings on Engineered Fill

Founding the abutments on engineered fill pads placed on the underlying bedrock after removal of the compressible cohesive layers is not considered practical due to the extensive dewatering and road protection requirements to carry out the approximately 4 to 7 m deep excavations.

However, in view of the relatively light structure proposed for the pedestrian underpass it is considered that spread footings perched on engineered fill incorporated into the RSS wall fill is a feasible option. To this end, the RSS wall sites should be preloaded to reduce the settlements under the RSS wall fill and achieve sufficient subgrade strength gain to allow construction of the final fill height with an adequate safety factor against slope failure. Details are provided in Section 4.

The spread footings should be perched within the RSS wall fill at a level that incorporates 2.0 m of earth cover for frost protection. The highest founding levels for these footings are estimated at elevation 222.0 at the south abutment and elevation 225.0 at the north abutment based on the proposed ground elevations in the GA drawings. Lower levels may be required for structural design and RSS wall design considerations. The interior facing of the RSS wall panels should be lined with a layer of extruded polystyrene at least 75 mm thick in front and below the footing level to provide adequate frost protection.

The RSS fill material below the footings should comprise OPSS Granular A material placed in 200 mm layers and compacted to 100% of the MTO test method LS-706 maximum dry density. Since compaction adjacent to the RSS wall panels is typically restricted to light equipment the thickness of the layers in this zone should be reduced to achieve a uniformly compacted subgrade. The engineered fill should extend laterally to a line inclined downward at 45° to the



horizontal originating at least 1.0 m from the top of the footing. This scheme is illustrated in Figure 1, appended. The limits of the required engineered fill pad should be clearly marked and surveyed in the field.

The recommended bearing resistance for a minimum 2.0 m wide footing placed on the structure fill is recommended as follows:

RESISTANCE	ENGINEERED FILL
Factored Bearing Resistance at ULS, kPa	900
Geotechnical Bearing Resistance at SLS, kPa	350

The resistance at SLS normally allows for 25 mm of compression of the founding medium. Differential settlement is expected to be less than 75% of this value. Smaller magnitude settlements will be induced by lower geotechnical resistance values, approximately on a directly proportional ratio.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of Clause 6.7.4 of the CHBDC.

The horizontal forces imposed on the foundations will be resisted in part by the friction forces developed between the underside of the footing and the engineered fill. An unfactored friction factor of 0.7 is recommended for footings placed on the engineered fill.

2.3 Deep Foundations

2.3.1 General

Conventional abutment designs are considered feasible at the pedestrian bridge site. The integral or semi-integral alternatives are not considered applicable due to the portability characteristics of the structure.

The general pile foundation design recommendations are provided on the following sections.



2.3.2 Driven Piles Axial Resistances

Piles for the north and south abutments and pier should be driven to refusal on the bedrock encountered at the estimated range of reference founding levels that are provided on the following table.

LOCATION	DEPTH * (m)	PILE FOUNDING ELEVATION *	RELEVANT BOREHOLES
North Abutment	6.6 to 7.5	212.0 to 212.8	5 and 6
Pier	6.1 to 6.7	213.1 to 213.8	3 and 4
South Abutment	3.8	214.6 to 214.8	1 and 2

Note (*) Depth and elevations are taken from the existing ground surface at the borehole locations to the top of the bedrock.

Since the pile depths will be very short after allowing for the frost protection of 2.0 m for the pile cap, the use of rigid insulation will be required to install the pile cap at a higher level and obtain the minimum structurally required pile length.

Since the presence of cobbles/boulders was not identified above the bedrock at the bridge foundation locations, the risk of damage during driving is considered to be low and, as a consequence, application of a reduction factor is not employed. The piles should be equipped with rock points such as the Titus "H" Bearing Pile Point, Rock injector Model and installed in accordance with the SP 903S01 for fixity into the rock due to the short pile lengths and to guard against potential sloping bedrock surfaces.

On the basis the above considerations the factored axial resistance at ultimate limit states (ULS) for the pile sections noted below is considered to be appropriate:

Pile Size	Factored Axial Resistance at ULS, kN
HP 310 x 79	1450
HP 310 x 110	2000

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the pile length required (about 4 to 8 m below existing grade), the design is



not expected to be governed by settlement since the required loads causing appreciable deformation of the pile are much larger than the ULS factored capacity.

If the construction requires a compacted granular fill pad for the installation of the abutment piles, the fill pad should comprise of OPSS Granular A or Granular B Type II materials to allow installation of the piles without damage.

The construction of the false abutment RSS walls will require initial preloading of the abutment areas to minimize the settlements due to the placement of the RSS-specific fill (refer to Section 4 of this report for fill placement details). Consequently, the soil settlements after pile installation will not induce significant drag down forces.

The piles will be driven through 4 to 7 m of native soils that typically comprise soft to firm or compact soils and to the underlying bedrock surface. It is considered, based on our experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed.

The pile sets should be prepared based on the section of steel pile to be selected, the energy of the hammer and setup that will be employed.

2.3.3 Driven Piles Lateral Resistance

The soil adjacent to the upper section of the piles is expected to comprise the typically loose to compact sands and silts. These soils are underlain by soft to firm silty clay/clayey silt layers.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The recommended lateral resistance for the pile sections noted previously is as follows:

RESISTANCE	SAND/SILT	GRANULAR BACKFILL	CLAYEY SILT/ SILTY CLAY
Factored Lateral Resistance at ULS, kN	100	120	120
Lateral Resistance at SLS, kN	25	50	35



The lateral resistance values assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended in Section 4 titled "Embankments". If greater resistance is required, batter piles should be installed.

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

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

2.3.4 Caisson Foundations Axial Resistances

The south and north abutments and piers could be founded on caissons socketed or doweled into the bedrock underlying the site. Some difficulties with the caisson installations should be expected due to the potential presence of wet cohesionless soils above the bedrock surface such as those encountered at the pier location. To this end the caissons should be advanced using temporary steel liners and in hole groundwater control may be required. These difficulties may lead to delays in advancing the caisson holes and preparing the adequate bearing surfaces on the rock surface.

To socket the caissons into the bedrock for fixity purposes, it will be necessary to limit the diameter of the caissons to those of the equipment available from the specialist contractors (usually 600 mm). For caissons doweled into rock, the diameter should be at least 900 mm to allow for worker access.



The length of the caisson sockets into rock should be a minimum of 0.5 m in view of the locally good to excellent rock quality. The need for a greater socket length to resist lateral loads should be assessed. Further comments regarding lateral resistance are provided in a subsequent section of this report.

The caissons should be designed using a factored end bearing resistance at ULS of 10,000 kPa and a factored bond stress at ULS of 1,000 kPa.

The full value for shaft adhesion may be employed in caisson design based on shaft adhesion only (end-bearing ignored) or the caissons may be designed based on end bearing only where the end bearing resistance exceeds the total shaft resistance. In cases where the total shaft resistance is greater than the end bearing resistance and the design is based on resistance being developed by both end-bearing and shaft adhesion, the mobilized resistance of the caisson should be limited to two times the end bearing resistance or the end bearing resistance plus 75% of the computed bond capacity, whichever is less.

Based on these values, the factored axial resistance at ULS for selected caisson diameters embedded into rock by 1.0 and 0.5 m is presented below:

CAISSON DIAMETER (m)	FACTORED AXIAL RESISTANCE AT ULS (kN)					
	1.0 m LONG SOCKET			0.5 m LONG SOCKET		
	End Bearing	Shaft Adhesion	Total	End Bearing	Shaft Adhesion	Total
0.60	2,825	1,885	4,710	2,825	940	3,765
0.76	4,535	2,385	6,920	4,535	1,190	5,725
0.91	6,500	2,850	9,350	6,500	1,425	7,925

The actual resistance values depend on the length of embedment to be determined based on structural design considerations.

The construction of the false abutment RSS walls will require initial preloading of the abutment areas to practically eliminate the settlements due to the placement of the RSS-specific fill (refer to



Section 4 of this report for fill placement details). Consequently, the soil settlements after caisson installation will not induce significant drag down forces.

If the caissons are anchored to the bedrock using steel dowels, a NSSP should be included in the tender documents to provide specific direction for the contractor during installation and testing of the dowels. Fractured rock should be removed from these areas.

Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 (CHBDC). If anchors are installed, a factored bond stress at the rock/grout interface of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) is recommended for design. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length.

The resistance at SLS allows for 25 mm compression of the caisson and founding medium. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement criteria since the load required to cause such deformation of the caisson and bedrock would be larger than the factored resistance at ULS.

The caissons should be installed and monitored in accordance with the requirements of MTO SP 903S01.

Although the rock surface appears to be relatively level in the boreholes, the contractor should allow for inclined rock surfaces which will require site specific drilling techniques.

Sealing the bottom of the steel liner on the bedrock surface may be difficult in view of the possible undulations/stepping on the surface of the bedrock. Placement of concrete by tremie will probably be necessary.



2.3.5 Caisson Foundations Lateral Resistance

Resistance to lateral loads will be provided by the 'horizontal bearing resistance' of the rock. The spacing between caissons in the direction transverse to the applied load should be at least three caisson diameters. The caisson spacing in the longitudinal direction should not be less than the socket length of the caisson to enable full mobilization of the lateral restraint. If the spacing is less than the socket length, the resistance should be reduced in direct linear proportion to the ratio of the two values.

The factored horizontal resistance at ULS of sound bedrock is considered to be 5,000 kPa. The resistance at SLS is greater and hence the factored resistance at ULS will govern the design.

Since the bedrock cores retrieved from boreholes drilled at the abutments are typically good to excellent quality, the bedrock is considered to be a 'frictional' material for the purpose of evaluating the point of contraflexure.

The coefficient of horizontal subgrade reaction, k_s (kN/m³) should be computed using the following equation to evaluate the point of contraflexure:

$$k_s = n_h z/b$$

where:

n_h = coefficient related to rock quality = 30,000 kN/m³

z = depth below bedrock surface, m

b = caisson diameter, m

3. RETAINING WALLS

3.1 Retaining Wall Design

The proposed retaining walls for the south stairways and ramps and/or abutment or false abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall above the bedrock surface and the compaction pressure imposed during placement of the backfill. The lateral earth pressure, p (kPa), may be computed using the



equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:

$$p = K (\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)
 γ = unit weight of free-draining granular material, kN/m³
 h = depth below final grade, m
 q = surcharge load, kPa, if present
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
 where ϕ = angle of internal friction of retained soil (35° for granular materials)
 δ = angle of friction between the soil and wall (23.5° for granular materials)

The seismic site coefficient for the conditions at this site was provided in Section 2.1.

Free-draining OPSS granular material or rockfill should be used as backfill behind the walls. The following parameters are recommended for design:

PARAMETER	GRANULAR A OR GRANULAR B TYPE II	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m ³	22.8	18.0
Coefficient of Active Earth Pressure, K_a	0.27	0.20
Coefficient of Earth Pressure At Rest, K_o	0.43	0.33
Coefficient Passive Earth Pressure, K_p	3.69	5.04

The coefficient of earth pressure at rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures.

A retained soil system (RSS) could also be employed for the abutment or false abutment wall. We believe that a high site performance rated RSS wall should be employed because the RSS will be constructed above the highway. The design, supply and construction of the RSS should be in conformance with SP 599S22 and SP 599S23. These proprietary systems should be designed



by the supplier and should consider the methods selected to construct the approach embankment fills which are discussed in the following Section 4 of this report.

The RSS walls should be placed on the native sandy soils after these soils have been preloaded. This foundation subgrade will not require subexcavation beyond removing disturbed soils and the bearing resistances to be used in the design should be as follows for the RSS wall levelling pad. A 0.5 m wide pad is recommended to minimize punching into the sand/silty sand subgrade.

Factored Geotechnical Resistance at ULS, kPa	250
Geotechnical Resistance at SLS, kPa	150

The geotechnical resistance at SLS was adjusted to allow a maximum settlement of about 20 mm of the founding subgrade for the RSS wall construction. The water level was considered to be at a depth of about 0.5 m below the RSS wall footing after construction.

The proposed stairways retaining walls over the south embankment should also be selected taking into consideration the construction of the embankment fill. The walls may be selected to withstand small settlements or where no settlements are allowed by the designer, the footings should be supported on deep foundations, including standard piles or minipiles, installed on the bedrock which underlies the site at 3.8 to 7.3 m below existing grade. Minipiles are a proprietary system and their design should be provided by the supplier.

The RSS supplier should also be responsible for specifying the type of backfill material employed, taking into consideration the engineering properties of the proprietary product, the design life of the structure, the pullout resistance required, drainage requirements and the predicted settlements noted in the following Section 4.

The supplier of the RSS should also be responsible for design of the structure (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance, etc.



3.2 Sliding Resistance

The design of the retaining walls should be checked for sliding resistance using the following geotechnical parameters for cast-in-place concrete foundations placed on fill or native cohesionless soils.

PARAMETER	GRANULAR A OR GRANULAR B TYPE II	SAND/SANDY SILT
Friction Angle, degrees	35	32
Cohesion, kPa	0	0
Unit Weight, kN/m ³	22.8	20.0

A weeping tile system and/or weep holes should be installed to minimise the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or non-woven Class II geotextile (with an FOS of 75-150 μm according to OPSS 1860) placed to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

4. EMBANKMENTS

The approach embankments may be constructed with earthfill or rockfill. It is understood, however that MTO prefers the use of earthfill embankments for providing landscaped slopes. The side slopes of the embankments should be inclined no steeper than 2H:1V for earthfill and 1.25H:1V for rockfill. Mid-height benches should be provided for the sections higher than 8 m in earthfill or 10 m in rockfill in accordance with OPSD 202.010.

It is estimated that the maximum embankment fill height will be in the range of 7.0 to 8.0 m respectively at the south and north abutments of the bridge structure. The south embankment will be underlain by loose to compact sand/sandy silt about 1.2 to 1.5 m thick which overlays typically soft to firm silty clay/clayey silt extending to 3.8 to 7.5 m depths. The north embankment will be generally underlain by about 2.0 m thick sand/sandy silt from the structure abutment to about Sta. 9+920 and tapering to 0.7 m northerly. The sandy soil overlays firm to stiff silty clay/clayey silt soils to depths ranging from 7.3 m at the structure to zero at a rock outcrop near Sta. 9+885 and increasing to about 6.0 m northerly.



Placement of the embankment fills directly on the native soils without preloading will induce excessive settlements in the order of 55 and 75 mm at the south and north abutment sites. The magnitude of the settlements is partly a function of the height of the embankment and will be smaller away from the structure where the embankments are lower. Removal of the compressible silty clay/clayey silt soils underlying the upper sandy soils is considered to be impractical since wide excavations and groundwater control would be required, as previously outlined in this report.

Based on the site stratigraphy and requirements for the bridge abutments and RSS walls outlined previously, it is envisaged that the embankments may be constructed over the existing native cohesionless soil using the following methodology.

Abutment and RSS Wall Areas

- At the north embankment site, drain the existing ponding water and divert all surface water, including stormwater discharge away from the work area. Perimeter ditches should be used during construction to maintain the groundwater at least 0.5 m below the existing ground surface.
- Existing services and utilities crossing the footprint of the embankments should be relocated. Adequate compaction of the new and existing trench backfill should be provided to prevent local post-construction settlements.
- Preload the proposed abutment and required RSS wall fill areas with 4.0 m of earth fill for at least a 1.5 month period. The preloading fill should be placed with 2H:1V side slopes. Where space restrictions require that the fill be placed with 1H:1V side slopes, granular fill should be used. Adjacent to roadways, the top and side slopes of the fill should be covered with tarpaulins for protection against erosion.
- Remove the preloading fill and the previously existing topsoil/peat layers using care to avoid punching these organic materials into the underlying sand/silt subgrade.
- Immediately cover the prepared subgrade with a layer of biaxial geogrid (25 by 35 mm max. aperture, 1.2 to 2.0 kN/m min. peak tensile strength) and backfill with a layer of Granular B Type II or Type III material at least 600 mm thick.
- Construct the pile/caisson foundations as required or proceed with construction of the RSS wall abutment in case of perched spread footing option.



- Construct the remaining embankment and RSS wall fill upon installation of the abutment foundations.

Other Embankment Sections:

- At the north embankment site, drain the existing ponding water and divert all surface water, including stormwater discharge away from the work area. Perimeter ditches should be used during construction to maintain the groundwater at least 0.5 m below the existing ground surface.
- Existing services and utilities crossing the footprint of the embankments should be relocated. Adequate compaction of the new and existing trench backfill should be provided to prevent local post-construction settlements.
- Remove the previously existing topsoil/peat layers using precautions to avoid punching these organic materials into the underlying sand/silt subgrade.
- Immediately cover the prepared subgrade with a layer of biaxial geogrid (25 by 35 mm max. aperture, 1.2 to 2.0 kN/m min. peak tensile strength) and backfill with a layer of Granular B Type II or Type III material at least 600 mm thick.
- Construct the remaining embankment fill. In the first stage place the embankment fill to a maximum height of 5.0 m. Where the required fill is higher than 5.0 m, the fill placement above the 5.0 m height should be discontinued for a period of about 1.5 months.

All materials for the granular pad over the geogrid layer and the material for the north embankment should be placed in 200 mm thick layers and compacted to 95% of the target density following the requirements of SP 902S01 and OPSS 501.

For the south embankment, the groundwater control and use of geogrid is not considered required in view of the groundwater conditions encountered. All of the south embankment fill should be compacted to 100% of the target density and strictly controlled as engineered fill, since the ramps and retaining walls will be supported on the fill.

It is estimated that the settlement of the sand/sandy silt layers will be about 20 mm and will occur immediately upon loading. The estimated settlements of the silty clay/clayey silt soils underlying



the north and south embankments are about 55 and 35 mm respectively. Upon application of a 4.0 to 5.0 m surcharge about 35 and 25 mm of these settlements will be essentially completed. The fill should be placed in two stages with the first stage limited to a 5.0 m height to ensure the stability of the embankments. The second stage should be placed about 1.5 month after the completion of the first stage to allow for partial consolidation and strength gain of the native soils.

The results of the slope stability analysis carried out using the Slope/W software package provided the results summarized on the following table:

Slope Stability Analyses Results

Condition	Earth Fill (2H:1V)			Rock Fill (1.25H:1V)			Rockfill (2H:1V)		
	Fill Height			Fill Height			Fill Height		
	5 m	8 m	8 m with pre-load (*)	5 m	8 m	8 m with pre-load (*)	5 m	8 m	8 m with pre-load (*)
Short-Term	1.4	1.0 (**)	1.3	1.4	1.0 (**)	1.3	1.6	1.1 (**)	1.5
Long-Term	-	-	1.5	-	-	1.5	-	-	1.8

Note: (*) Minimum pre-load of 1.5 m is required for strength gain of native soil.
 (**) Marginal value - placing fill above 5.0 m without pre-loading is not recommended.

As indicated by the results of the slope stability analyses, the slopes constructed as recommended in this report will be stable with short-term safety factors of about 1.3 to 1.5 and long-term safety factors of 1.5 and 1.8.

The embankments should be constructed in accordance with OPSD 201.010, 202.010, 208.010 and SP 206S03.

Earth fill slopes should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 and 572 for time constraints and type of seed and mulch required.



5. EXCAVATION AND GROUNDWATER CONTROL

5.1 General Considerations

Excavation for construction of the abutment pile or caisson caps may extend through new embankment fills, existing fill and the native soft to stiff/loose to compact soils to about 2.0 m depth below existing grades (for frost protection) in case extruded polystyrene insulation is not used to reduce the earth cover requirement.

These soils are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes above the groundwater level inclined at 45° to the horizontal should be stable. Flatter side slopes (about 3H:1V) will be required below the water level. We refer to the previous discussions in this report for the encountered groundwater levels at the work areas north and south of the highway and at the center pier.

The cohesionless soils below the groundwater should be classified as Type 4 soil if groundwater is not adequately controlled. For this condition, side slopes should be cut at 3H:1V.

5.2 Road Protection Considerations

Should construction and traffic staging require traffic adjacent to the expected maximum 2.0 m deep excavations, it is anticipated that a suitable roadway protection scheme following SP 105S19 will be required to support the walls of the excavation and adjacent traffic lanes during construction. The conditions for excavation depths less or equal to three (3) metres would apply.

It is noted that soldier pile and lagging schemes may not be considered adequate where the excavation will be carried out through sand or gravel fills or native sandy silt materials in particular under the water table. The contractor is responsible for the selection, design, preparation and performance of a detailed design for the road protection scheme.



5.3 Groundwater Control Considerations

The water level observed at the time of the field subsurface investigation (elevations 218.8 to 220.8 at the north abutment and elevations 217.0 to 217.4 at the south abutment) was near to 0.3 m above the anticipated level of excavation at the north abutment. Cognisant of the relatively high permeability characteristics of the sand/sandy silt layers, it is anticipated that temporary perimeter drainage ditches outletting away from the construction areas will be required to control seepage of ground and surface water run-off into the excavations and general embankment subgrade areas.

The perched groundwater that was observed within the sand/sand silt in the north abutment area boreholes 10 to 20 during or upon completion of drilling should be considered when excavating for the construction of the new embankment. Groundwater levels are subject to seasonal fluctuations and rainfall patterns. Seepage should be anticipated locally at the fill/native soil interface. It is anticipated that conventional sump pumping techniques will be sufficient to control normal seepage of groundwater into the general embankment and foundation excavations.

The contract documents should clearly state that groundwater control of the excavations is the contractor's responsibility.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.



6. COMPARISON OF FOUNDATION ALTERNATIVES

A comparison of the relative advantages and disadvantages related to each of the foundation alternatives discussed in the preceding paragraphs is presented below.

Footings on Native Soils or Bedrock

Advantages

Disadvantages

(Not Considered Practical)

Footings on Engineered Fill (RSS Wall Abutment)

Advantages

Disadvantages

- | | |
|--|---|
| <ul style="list-style-type: none">• Cost-effective option for proposed light structure• Groundwater control not required• May be constructed within time constraints | <ul style="list-style-type: none">• Requires interruption of RSS wall construction• Requires non-standard fill compaction detail |
|--|---|

Caissons to Bedrock

Advantages

Disadvantages

- | | |
|---|---|
| <ul style="list-style-type: none">• May be dowelled or socketed into rock for fixity• More suitable for shallow bedrock depths | <ul style="list-style-type: none">• Requires specialized construction equipment for socketing into bedrock• Installation may be difficult due to sandy soil at soil/rock interface (pier location)• May require unwatering of caisson holes |
|---|---|

Piles to Bedrock

Advantages

Disadvantages

- | | |
|---|---|
| <ul style="list-style-type: none">• May be removed if/when structure is relocated• Unwatering of foundation sites not required (below pile caps) | <ul style="list-style-type: none">• Less suitable than caissons for shallow bedrock depths• Requires rock points for fixity into bedrock |
|---|---|

Alternative Scheme (Minipiles)

Advantages

Disadvantages

- | | |
|---|--|
| <ul style="list-style-type: none">• Only lightweight equipment typically required• Unwatering of foundation sites not required (below pile caps) | <ul style="list-style-type: none">• Proprietary scheme to be provided by supplier/installer• Needs to be drilled into rock for fixity |
|---|--|



From the foundation perspective, the deep foundations and engineered fill within the RSS wall options are feasible for the bridge abutments.

The footings placed on engineered fill constructed within the RSS wall are considered the most cost-effective option. Since the construction of the engineered fill may be scheduled with the construction of the RSS wall, it is anticipated that construction will be completed within the time constraints for the project.

It is considered, therefore, that the placement of the abutment foundations on footings placed on the RSS engineered fill is the preferred alternative from the foundation technical and cost-effectiveness perspective.

For the required pier deep foundation it is considered that driven piles will be straightforward and will not require groundwater control. Hence the driven piles are considered the preferred alternative from the foundation and cost-effectiveness perspective for the pier foundation.

It is noted that the selection of the foundation scheme(s) will involve other considerations, such as overall cost and schedule that are to be evaluated by Stantec.

7. CLOSURE

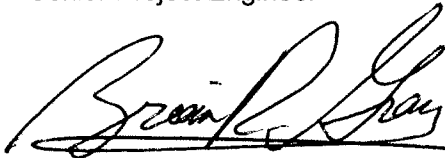
The report was prepared by Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.



C. M. P. Nascimento, P.Eng.,
Senior Project Engineer



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

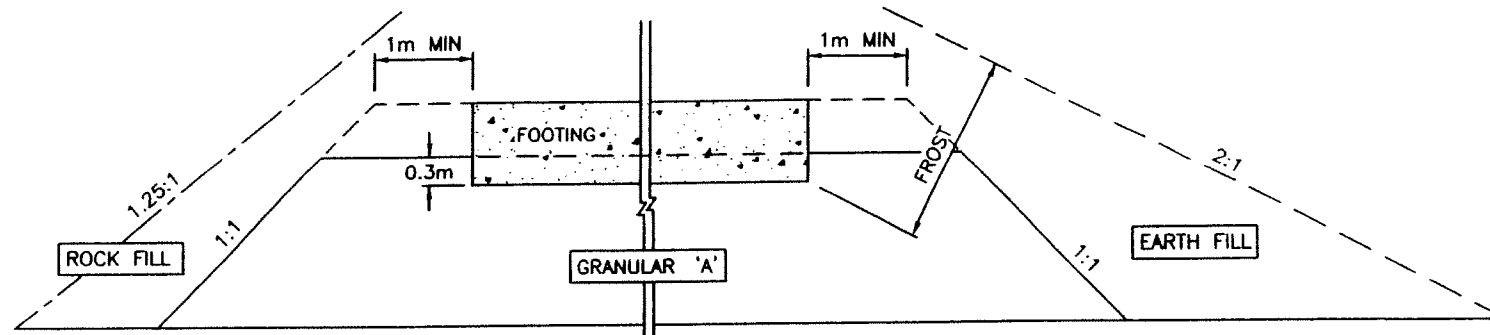


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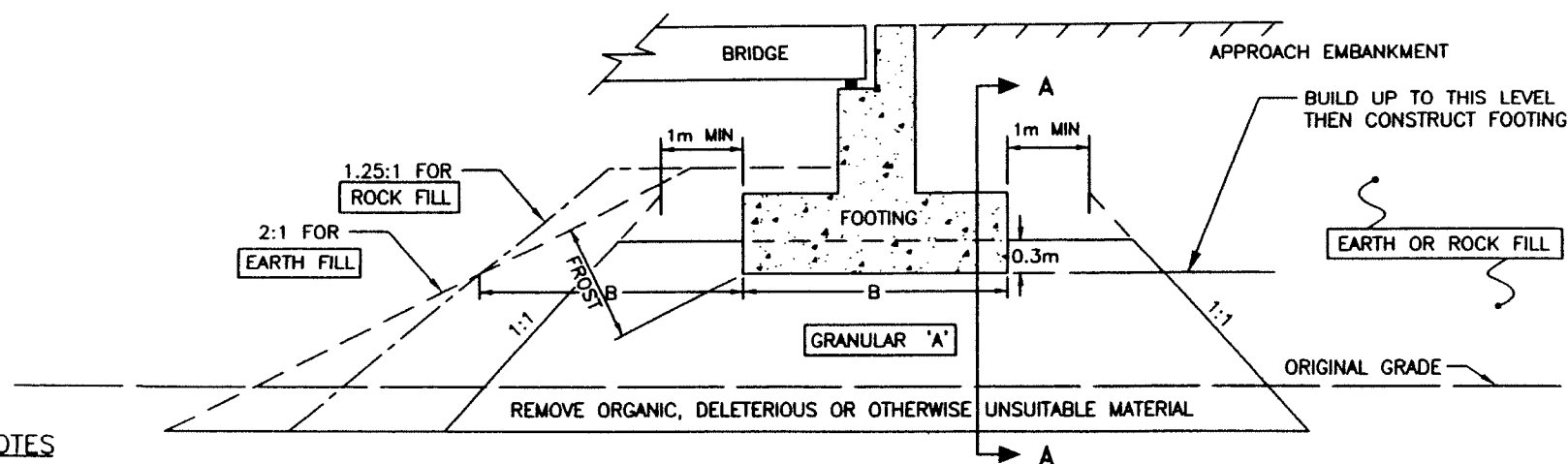
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
SP 105S19	Construction Specification for Protection Systems	November 2006
SP 206S03	Construction Specification for Grading	November 2006
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)	March 2001
SP 599S23	Requirements for Materials, Quality Control and Quality Assurance Testing and Acceptance Criteria for Precast Concrete Facing Elements Including Panels	March 2006
SP 902S01	Excavation and Backfilling of Structures	June 2006
SP 903S01	Construction Specification for Piling	November 2006
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock	November 2006
OPSS 501	Construction Specification for Compacting	November 2005
OPSS 571	Construction Specification for Sodding	November 2001
OPSS 572	Construction Specification for Seed and Cover	November 2003
OPSS 1860	Material Specification for Geotextiles	November 2004
OPSD 3090.100	Foundation Frost Depth for Northern Ontario	November 2005
OPSD 201.010	Rock Grading-Undivided Rural	November 2005
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment	November 2005
OPSD 208.010	Benching of Earth Slopes	November 2003



CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE