

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 58
PINE STREET UNDERPASS REPLACEMENT
(SITE 34-179)
CITY OF THOROLD, ONTARIO
G.W.P. 2365-09-00, W.P. 2367-09-01
P.O. 2010-E-0073**

GEOCRES No. 30M3-275

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out at the location of the proposed replacement of the Pine Street Underpass at Highway 58 in the City of Thorold, Ontario. This investigation was carried out as part of a consolidated assignment to rehabilitate or replace six (6) bridge structures at five (5) site locations in Thorold and St. Catharines, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide borehole location plan and soil strata drawings with stratigraphic profiles and cross-sections, records of boreholes, laboratory test results and written descriptions of the subsurface conditions. A model of the subsurface conditions was developed for the site based on the data obtained from the present investigation.

Thurber carried out the investigation as a foundation sub-consultant to McCormick Rankin, a member of MMM Group (MRC) under MTO Purchase Order No. 2010-E-0073.

The following MTO document has been referenced in the preparation of this report:

- Department of Highways Ontario (DHO) drawing titled "Pine Street Underpass, King's Highway 58 (Prop.), Welland Co., Thorold Twp., General Layout", Site No. 34-179, W.P. No. 10.65 dated November 1965 (Reference 1).

2 PROJECT AND SITE DESCRIPTION

The existing Pine Street Bridge is a double span bridge carrying Pine Street over Highway 58 in the City of Thorold, The Regional Municipality of Niagara. The lands surrounding the bridge site are primarily occupied with light residential dwellings. In the general vicinity of the bridge, Highway 58 has been constructed in a cut and the Pine Street grade appears to be at the original ground surface.

From published geological information, the bridge site is located within the physiographic region known as the Iroquois Plain. The bridge site is also located in the vicinity of the Niagara Escarpment, which is located to the south of the City of St. Catharines. In this area, a deposit of silty clay to clayey silt till overlies dolostone bedrock of the Lockport Formation.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out in two phases. The first phase involved drilling and sampling Boreholes PS-04 and PS-05 at Highway 58 grade on April 16 and 17, 2012, respectively. The second phase consisted of drilling and sampling Boreholes PS-01 to PS-03 and PS-06 to PS-08 on May 28 and 29, 2012 at Pine Street grade.

Boreholes PS-01 and PS-07 were drilled at the north and south approaches, respectively. Boreholes PS-02 and PS-03 were drilled near the north abutment, Boreholes PS-04 and PS-05 near the Pier, and Boreholes PS-06 and PS-08 near the south abutment. The borehole depths ranged from 2.2 to 11.2 m (Elevations 167.6 to 172.3 m).

There is no GEOCRE report for this site, however simplified plots of two boreholes (#1031 and #1032) are shown on a Department of Highways Ontario drawing for the existing bridge (Reference 1). The subsurface information shown on these simplified plots does not include SPT 'N' values or any other indication of soil strength and density/consistency, nor groundwater conditions.

Solid stem augers and coring techniques were used to advance the current boreholes through soil and bedrock. Overburden samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Boreholes PS-02, PS-05, and PS-08 were advanced a minimum of 3.0 m into bedrock by NXL size diamond coring. The remaining boreholes were terminated upon auger and/or split spoon refusal on probable bedrock or boulders.

Groundwater conditions in the open boreholes were observed upon completion of drilling. A standpipe piezometer, consisting of 19 mm diameter Schedule 40 PVC pipe with a 1.5 m long slotted screen, was installed in Boreholes PS-03 and PS-06. The installation details for the piezometers are summarized below along with the backfill details for other boreholes without piezometer installation.

Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
PS-01	None installed	Backfilled with bentonite holeplug and cuttings to 0.6 m, dry concrete from 0.6 m to 0.15 m, then asphalt coldpatch to surface.
PS-02	None installed	Backfilled with bentonite holeplug and cuttings to 0.6 m, dry concrete from 0.6 m to 0.15 m, then asphalt coldpatch to surface.

Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
PS-03	6.6 / 171.6	Filter sand from 6.6 m to 3.4 m, then bentonite holeplug from 3.4 m to 0.15 m. Flushmount protector installed at surface.
PS-04	None installed	Backfilled with bentonite holeplug and cuttings to 0.6 m, dry concrete from 0.6 m to 0.15 m, then asphalt coldpatch to surface.
PS-05	None installed	Backfilled with bentonite holeplug and cuttings to 0.6 m, then asphalt coldpatch to surface.
PS-06	7.3 / 170.9	Filter sand from 7.3 m to 3.4 m, then bentonite holeplug from 3.4 m to 0.15 m. Flush mount protector installed at surface.
PS-07	None installed	Backfilled with bentonite holeplug and cuttings to 0.6 m, dry concrete from 0.6 m to 0.15 m, then asphalt coldpatch to surface.
PS-08	None installed	Backfilled with bentonite holeplug and cuttings to 0.7 m, dry concrete from 0.6 m to 0.15 m, then asphalt coldpatch to surface.

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, processed the soil and rock core samples in labelled containers and wooden core boxes, respectively, for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and properties including the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are summarized on the Record of Borehole sheets included in Appendix A. At least 25% of the recovered soil samples were subjected to grain size distribution analysis. Atterberg Limits tests were carried out on selected samples of silty clay till and clayey silt till to determine the plasticity characteristics. The results of these tests are plotted on the figures included in Appendix B.

Point load testing was carried out on selected rock cores retrieved from Boreholes PS-02, PS-05, and PS-08. The results of these tests are presented on the Record of Borehole sheets (as estimated UCS) included in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference should be made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these records and on the “Borehole Locations and Soil Strata” drawings included in Appendix C. General descriptions of the stratigraphy based on boreholes drilled during the current investigation are given in the following paragraphs. It should be noted that the factual information presented in the Record of Borehole sheets governs any interpretation of site conditions.

In general, the subsurface stratigraphy at the site consists of a pavement structure of asphalt or concrete underlain by gravelly sand to sand and gravel fill with silty clay fill and silty sand fill encountered locally in Borehole PS-04. The fill typically overlies native silty clay till. Silty sand till was encountered locally in Borehole PS-08 below the silty clay to clayey silt till. The above soils are underlain by dolostone bedrock of the Lockport Formation.

5.1 Pavement Structure

Pavement structure consisting of asphalt overlying granular fill materials was encountered in the two boreholes drilled on Highway 58 (Boreholes PS-04 and PS-05). The thickness of the asphalt ranged from 175 to 210 mm.

The six boreholes drilled on Pine Street (Boreholes PS-01 to 03 and PS-06 to 08) encountered concrete with rebar at the surface overlying granular fill materials. The concrete was 125 mm thick in all six boreholes.

The granular fill consisted of gravelly sand to sand and gravel in all boreholes except Borehole PS-04 which encountered 1.3 m of silty clay fill overlying 0.7 m of silty sand fill. The thickness of the granular fill ranged from 0.7 to 2.2 m, with the lower boundary of the granular fill encountered at depths of 0.8 to 2.3 m (Elevations 178.1 to 170.7 m).

SPT ‘N’ values recorded in the granular fill ranged from 2 to 38 blows for 0.3 m penetration, indicating a variable relative density from very loose to dense. An SPT ‘N’ value of 50 blows for 0.1 m penetration was recorded in Borehole PS-05 at the fill-bedrock interface. In Borehole PS-04, the measured ‘N’ values of 12 and 10 blows indicate a stiff consistency and a compact relative density for the silty clay fill and silty sand fill, respectively. The moisture content of samples of the fill ranged from 8 to 20%.

5.2 Silty Clay Till

Silty clay till was encountered below the granular fill in Boreholes PS-01 to PS-03 and PS-06 to PS-08. This cohesive till typically contained trace sand and was brown in colour becoming grey with increasing depth.

Boreholes PS-01, PS-03, PS-06, and PS-07 were all terminated upon auger refusal. In these boreholes, the till was 5.7 to 6.7 m thick and the boreholes were terminated at depths of 6.6 to 7.5 m (Elevations 171.6 to 170.7 m). The silty clay till was fully penetrated in Boreholes PS-02 and PS-08 and was found to be 4.7 to 6.1 m thick. In these boreholes, the lower boundary of the cohesive till was encountered at depths of 6.9 to 7.0 m (Elevations 171.2 to 171.3 m).

SPT 'N' values recorded in the silty clay till ranged from 8 to 26 blows for 0.3 m penetration, indicating a stiff to very stiff consistency. The measured moisture content of samples of the silty clay till ranged from 20 to 28%.

Selected samples of silty clay till were subjected to gradation analysis and Atterberg Limits testing. The results of these tests are summarized in the tables below as well as on the Record of Borehole sheets included in Appendix A. Figures B1 and B2 present the grain size distribution curves for these samples, and Figures B3 and B4 illustrate the results of the Atterberg Limits tests on plasticity charts.

Soil Particles	Percentage
Gravel	0 to 3
Sand	0 to 16
Silt	43 to 73
Clay	23 to 57

Soil Particles	Percentage
Liquid Limit	27 to 51
Plasticity Index	12 to 28

The results of the Atterberg Limits tests indicate that the silty clay till has a variable plasticity, ranging from low plastic (CL) to high plastic (CH) and generally has an intermediate plasticity (CI). It should be noted that glacial tills inherently contain cobbles and boulders.

5.3 Silty Sand Till

Silty sand till was encountered locally in Borehole PS-08 below the silty clay till. The silty sand till was grey in colour and contained some gravel and some clay.

The silty sand till was 1.2 m thick with a lower boundary encountered at a depth of 8.2 m (Elevation 170.0 m). A single SPT 'N' value of 65 blows for 0.3 m penetration was recorded in the silty sand till indicating a very dense relative density. The moisture content of the silty sand till was measured to be 8 % in one sample. It should be noted that glacial tills inherently contain cobbles and boulders.

5.4 Bedrock

The soils described above are underlain by bedrock which was proven by coring in Boreholes PS-02, PS-05, and PS-08. The remaining five boreholes were terminated upon auger and/or split spoon refusal on probable bedrock or boulders. The following table summarizes the depths and elevations of bedrock or auger refusal encountered at the borehole locations.

Proposed Foundation Element	Borehole Number	Depth to Bedrock or Auger Refusal (m)	Elevation of Top of Bedrock or Auger Refusal (m)
North Approach	PS-01	6.6	171.6
North Abutment	PS-02*	6.9*	171.3*
	PS-03	6.6	171.6
Pier	PS-04	2.2	170.2
	PS-05*	1.6*	170.7*
South Abutment	PS-06	7.3	170.9
	PS-08*	8.2*	170.0*
South Approach	PS-07	7.5	170.7

* Bedrock proven by coring

Boreholes 1032 and 1031 (Reference 1) indicate that the bedrock is at Elevations 171.8 and 170.9 m near the north and south abutments, respectively.

Based on the rock cores from the current investigation, the bedrock was described as thinly bedded, grey, dolostone. The bedrock was typically in a moderately to slightly weathered state. Occasional joints and vugs were observed in the bedrock cores.

Total Core Recovery (TCR) of the bedrock ranged from 95 to 100%. The Rock Quality Designation (RQD) values generally ranged from 75 to 98%, indicating a fair to excellent rock quality. Lower RQD values of 46% and 61% were recorded for the rock cores retrieved from Borehole PS-05, drilled on Highway 58. The Fracture Index (FI) of the rock, expressed as fractures or joints per 0.3 m of core, was generally less than 5, except for a highly fractured zone encountered near the bedrock surface at the location of Borehole PS-05.

Point load tests were carried out at regular intervals on selected rock cores. The estimated Unconfined Compressive Strength (UCS) of the bedrock as inferred from the point load tests ranged from 133 to 185 MPa, indicating a very strong intact rock strength.

5.5 Water Levels

Standpipe piezometers were installed in selected boreholes to facilitate monitoring of groundwater levels. The water levels observed in the open boreholes on completion of

drilling are summarized below along with the groundwater levels measured in the standpipe piezometers.

Borehole	Date	Water Levels		Comment
		Depth (m)	Elevation (m)	
PS-01	May 29, 2012	DRY		Open borehole
PS-02	May 28, 2012	DRY		Open borehole
PS-03	July 16, 2012	5.5	172.7	Piezometer
PS-04	April 16, 2012	2.1	170.3	Open borehole
PS-05	April 17, 2012	DRY		Open borehole
PS-06	May 29, 2012	DRY		Open borehole
	July 16, 2012	7.1	171.1	Piezometer
PS-07	May 29, 2012	DRY		Open borehole
PS-08	May 28, 2012	DRY		Open borehole

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe climatic events.

Once groundwater monitoring is completed, all piezometer installations will be decommissioned shortly in accordance with Ministry of the Environment Regulation 903 and its Amendments (the water well regulation under the OWRA).

6 MISCELLANEOUS

Borehole locations were established in the field relative to the location of the existing structure. The ground surface elevations and coordinates at all borehole locations were established by surveyors arranged by MRC upon completion of drilling. Underground utility clearances were obtained for the borehole locations prior to drilling.

Elite Drilling Services of St. Catharines, Ontario supplied a truck-mounted CME-75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field investigation was supervised by Mr. Ryan Kromer, E.I.T. and Mr. Dave Ametrano of Thurber. Geotechnical laboratory testing was carried out in Thurber's Toronto Area laboratory.

Overall planning and supervision of the field program was conducted by Mr. Luke Gilarski, E.I.T. and Mr. Sydney Pang, P.Eng. Interpretation of the data and preparation of this report was carried out by Ms. Lindsey Blaine, E.I.T.

The report was reviewed by Messrs. Sydney Pang, P.Eng. and P.K. Chatterji, P.Eng., who is a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report provides an interpretation of the geotechnical data in the factual report and presents foundation design assessments for the existing structure and recommendations for proposed foundation units of the Pine Street Underpass bridge. This assessment is based on a GA drawing of the proposed bridge replacement dated June 2012 prepared by MRC, a set of design drawings of the existing bridge (including Reference 1), and subsurface information provided by a borehole investigation carried out between April 16 and May 29, 2012 by Thurber.

The existing Pine Street Underpass bridge, which is generally in a north-south orientation, carries Pine Street over Highway 58 which has an east-west alignment. The existing bridge consists of two spans of cast-in-place post-tensioned concrete voided deck slab supported by one pier and two abutments. Each span is approximately 19.8 m in length. Existing general layout and footing layout drawings show that the pier, consisting of four columns, is supported by four squared footings founded on bedrock and each abutment is supported on a continuous spread footing founded on native soils overlying bedrock.

The terms of reference requires that the structure be replaced. The preliminary GA drawing indicates that the replacement superstructure consists of two 20 m long spans of precast prestressed concrete box girders with semi-integral abutments. Each abutment is to have two 6.0 m long concrete wingwalls. Initial consideration was given to reusing the existing foundations to support a new bridge deck and, accordingly, an evaluation of the existing foundations as presented in Section 8 was carried out. It was subsequently decided, however, that new spread footing foundations will be used for support at both the pier and the abutments. The new pier and abutment footings are to be constructed at approximately the same locations as the existing ones. It is understood that the existing bridge superstructure and substructure will be removed prior to constructing the new bridge.

8 EVALUATION OF EXISTING FOUNDATIONS

There is no existing geotechnical/foundation report available in GEOCRESS for this site. Simplified plots of two boreholes, #1031 and #1032, are shown on a DHO General Layout drawing (Reference 1). This subsurface information does not include SPT 'N' values, or other indication of soil strength and density/consistency, nor groundwater conditions.

The current borehole investigation consists of eight (8) boreholes. Boreholes PS-01 and PS-07 were drilled near the north and south approaches, respectively. Boreholes PS-02 and PS-03 were drilled near the north abutment, Boreholes PS-04 and PS-05 near the pier, and Boreholes PS-06 and PS-08 near the south abutment.

Based on these boreholes, the subsurface at the bridge site consists of a pavement structure of concrete and granular fills overlying native, stiff to very stiff silty clay till which grades into a silty sand till at some locations. The above soils are underlain by Dolostone bedrock. The groundwater level was typically measured and observed to be at or slightly above bedrock. Perched water may exist within the glacial tills above bedrock.

Spread footings have been used for foundation support of the existing structure. At the abutments, continuous footings were designed to be founded on native soils overlying bedrock while the footings at the piers were to be founded directly on bedrock. It is noted that the design drawings do not show footing bearing capacities. Information on footing dimensions and founding elevations has been converted into S.I. units and presented in the following Table 1, which also shows our assessed bearing capacities based on current borehole information, if the existing foundations were to be used to support a new bridge deck.

Table 1
Foundation Dimensions and Assessed Geotechnical Resistances
Existing Bridge Foundations

Foundation Information	Abutments	# of Footings	Piers	# of Footings
Footing Width	3.5 m	1 at each abutment (continuous footing)	1.68 m	4 (squared footings)
Footing Length	16.3 m		1.68 m	
Founding Elevation	173.6 m		171.3 m	From east to west
			171.2 m	
			171.1 m	
		171.0 m		
Founding Materials	Stiff to very stiff, Silty Clay Till		Dolostone bedrock*	
Allowable Bearing Capacity (Working Stress Design)	250 kPa		3,500 kPa	

The “Section at Pier” from the Reference 1 drawing indicates that the existing pier footings are founded on bedrock. Based on Boreholes PS-04 and PS-05, however, these pier footings appear to be founded some 0.5 to 0.8 m above bedrock. This discrepancy could be due to differences of survey datum between then and now. Based on the relatively small size of these footings comparing to the abutment footings, we consider it reasonable to assume that these footings are indeed founded on bedrock.

9 FOUNDATION ALTERNATIVES

Consideration was given to various alternate foundation systems, taking into account the site stratigraphy, existing bridge configuration and the proposed design information.

The following lists the various foundation types that were considered:

- Spread footings on bedrock at the pier; spread footings on native till or on compacted Granular A pad resting on bedrock at the abutments
- Augered caissons (drilled shafts) socketted into bedrock
- Augered steel H-Piles socketted into bedrock.

A comparison of these foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

At this site, the choice of foundation types depends primarily on depths to bedrock. For depths to bedrock, below the fill, of less than 2 m at the pier, spread footings on bedrock is a feasible and practical option of foundation support for the new bridge. For depths to bedrock, below native till, up to about 3 m at the abutments, spread footings on undisturbed native till or on compacted Granular A pad founded on bedrock are both feasible options.

Augered caissons and augered H-piles may be considered for providing foundation support. For such shallow depths to bedrock at this site, however, these deep foundations are not considered to be cost effective and recommendations on these alternatives will not be further developed. Driven H-piles are not considered feasible at this site.

Based on the above, it is recommended that spread footings on bedrock be used at the pier as for the existing bridge. At the abutments, consideration should be given to using spread footings on native till or spread footings on compacted Granular A pad founded on bedrock. It is technically feasible to found the abutment footings on bedrock. However, due to the relatively larger and higher abutment walls that would be required, it is understood that this would not be a cost effective option and therefore no further recommendations have been developed.

10 RECOMMENDATIONS FOR PROPOSED FOUNDATIONS

Engineering analysis was conducted to estimate the design geotechnical resistances for the proposed footings at the piers and the abutments. The analysis was carried out based on the requirements for spread footings stipulated in the CHBDC (2010) and with reference to the CFEM (4th Edition).

10.1 Vertical Resistance

Results of the analysis are presented in Table 2 below. The footing dimensions for the new bridge may vary from those shown in Table 1 for the existing bridge.

Table 2
Recommended Design Geotechnical Resistances
New Bridge Foundations

Foundation Information	Abutment Footings	Pier Footings
Abutment Footings on Native Soils		
Founding Elevations	At or below 173.6 m	At or below 170.2 m
Foundation Materials	Stiff to occasionally very stiff Silty Clay Till	Dolostone bedrock
Factored Geotechnical Resistance at ULS (Limit States Design)	375 kPa	3,500 kPa
Geotechnical Resistance at SLS (25 mm settlement) (Limit States Design)	250 kPa	Not Applicable
Abutment Footings on Compacted Granular A Pad		
Foundation Materials	Compacted Granular A Pad on Dolostone bedrock	-
Factored Geotechnical Resistance at ULS (Limit States Design)	900 kPa	-
Geotechnical Resistance at SLS (25 mm settlement) (Limit States Design)	600 kPa (≤ 2 m thick pad) 450 kPa (2 to 3.2 m thick pad)	-

Notes *: A compacted Granular A pad satisfying MTO requirements may be used at each of the abutments.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistances must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

10.1.1 Spread Footings on Granular A Pad

At both abutment locations, spread footings may be founded on an engineered fill pad resting on the dolostone bedrock.

If an engineered fill pad is used at this site, sub-excavation of the native soils will be required to expose the bedrock surface. Based on the GA drawing, it is estimated that the thickness of the fill pad would be less than 2 m at the north abutment and range from 2.3 to 3.2 m at the south abutment. The elevations of the base of the engineered fill should correspond to the top of bedrock discussed in Section 5.4.

The engineered fill should consist of OPSS Granular “A” compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at $\pm 2\%$ of its optimum moisture content (OPSS 501) and generally conforming to the geometry illustrated in Figure E1. Placement and compaction of the Granular A pad must be carried out in the dry.

10.2 Horizontal Resistance

Resistance to sliding of a cast-in-place concrete footing on the lightly over-consolidated portion (below the crust) of the silty clay till and the compacted Granular A Pad may be computed on the basis of ultimate coefficients of friction of 0.45 and 0.6, respectively. Resistance to sliding of a cast-in-place concrete footing on the dolostone may be computed on the basis of an ultimate coefficient of friction of 0.8. The above are “ultimate” values requiring a degree of sliding movement to occur to fully mobilize the resistance.

10.3 Frost Protection

Frost protection is not required for footings founded on bedrock. Frost protection should be provided to all spread footings founded on soils including the Granular A materials forming the fill pads. This may take the form of 1.2 m of earth cover, or equivalent thermal insulation, in any direction over the underside of the footing.

10.4 Subgrade Preparation

At the pier footings on bedrock, and at the abutment footings where granular pads on bedrock are used, the surface of the exposed rock should be cleared of debris and soils that might have accumulated during the sub-excavation, and be cleaned. All shattered and loosened rock fragments should be removed from the footprint of the footing or granular pad. Mass concrete fill may be used to raise the founding grade where necessary. Inspection should be carried out to confirm that the bedrock conditions, as exposed at the founding level, are consistent with the design assumptions.

If the silty clay till is to form the founding subgrade for the new footings, the founding surface should be stripped of debris, loosened/softened or otherwise disturbed soils. Limited sub-excavation below the existing footing subgrade may be required at some locations. Inspection should be carried out to confirm that the subgrade conditions, as exposed at the founding level, are consistent with the design assumptions. The approved

subgrade surface should be covered with a layer of lean mix concrete of at least 100 mm thick for protection purposes.

11 LATERAL EARTH PRESSURES

Lateral earth pressures acting on the abutment walls and wingwalls may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K \cdot (\gamma h + q)$$

where: P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 10.1)

γ = unit weight of retained soil (see Table 10.1)

H = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 10.1 below.

Table 10.1 – Earth Pressure Coefficients (K)

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A and Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Embankment Fill $\phi = 30^\circ, \gamma = 20.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48	0.33	0.54
At rest (Restrained Wall)	0.43	-	0.47	-	0.50	-
Passive (Movement towards soil mass)	3.7	-	3.3	-	3.0	-

12 APPROACH CUTS

The existing north and south approach cut slopes at the Pine Street Underpass are typically vegetated with grass. Available information indicates that these cut slopes have been designed to have an inclination of 2H : 1V. The forward slopes under the bridge decks are covered with

concrete slope paving as indicated on the design drawings. There is no visual evidence to date of major distress associated with the approach slopes.

The current design calls for construction of compacted Granular A pads at the approach locations. These pads are to form the core of the new approach embankment. It is anticipated that the approach slopes at a final configuration of 2H : 1V or flatter will remain stable provided that they are constructed in accordance with the recommendations in this report.

Should any alterations to the approach fill configuration other than what is discussed above be required, slope stability assessment will have to be carried out to confirm that the reconstructed slopes will remain stable with acceptable factors of safety.

13 ROADWAY PROTECTION

Roadway protection may be required during the course of the bridge replacement works. An item titled "Protection System" as per OPSS 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall. It is anticipated that the soldier piles will need to be extended into the stiff to very stiff silty clay till in order to develop the required toe resistance. At highway grade where depth to bedrock is shallow, the soldier piles may have to be socketted into bedrock. It is anticipated that the shoring system may be stiffened by cross bracings, where applicable.

A temporary soldier pile and lagging wall may be designed using the parameters given below:

γ	=	20 kN/m ³
γ_w	=	10 kN/m ³
K_a	=	0.33 (approach fills)
	=	0.36 (silty clay till)
K_p	=	3.0 (approach fills)
	=	2.7 (silty clay till)

The designer of the roadway protection system should check whether the depth of pile is sufficient to provide base fixity.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. All shoring systems should be designed by a Professional Engineer experienced in such designs.

14 EXCAVATION AND BACKFILL

Temporary excavations will be required during the course of the bridge replacement works. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purpose of OHSA, the native soils and existing fills at the site may be classified as Type 3 soils. Excavations may need to be carried out up to the order of 1 m below the groundwater level. In accordance with OHSA, unsupported open cut slopes are feasible where space permits but should not be steeper than 1H : 1V for Type 3 soils. Flatter slopes may be required below the groundwater level.

Excavation and backfilling for foundation construction should be carried out with reference to the requirements in OPSS 902. Removal of cobbles and/or boulders may be required at some locations. Backfill to the abutments should consist of Granular A or Granular B Type II materials meeting the gradation and relevant requirements stipulated in OPSS 1010. Compaction procedures and equipment to be used adjacent to the existing structures must be in accordance with the relevant OPSS 501 requirements.

15 GROUNDWATER AND SURFACE WATER CONTROL

It is anticipated that the amount of perched water within the fills at the abutment locations would be limited. Groundwater from water-bearing sand and silt interlayers within the silty clay till should also be minimal. For excavations at the abutment and pier locations, groundwater control will likely be limited to diverting surface runoff and sump pumping. Filtered sumps must be designed properly so that construction drainage water containing eroded soil and fines does not flow onto existing the highway and roadways.

The design of unwatering systems for the excavations is the responsibility of the Contractor who is expected to retain dewatering specialists for this task.

16 CONSTRUCTION CONCERNS

During construction, the Contract Administrator (CA) should employ experienced foundation/geotechnical staff to observe foundation construction activities.

The approach cuts at the existing Pine Street Underpass site are considered stable at an inclination of 2H : 1V. There is no visual evidence or report to date of major distress associated with the approach slopes.

Excavation for removal of the existing substructure, construction of the granular pads and reinstatement of the approaches will result in alterations to the slope configurations. Visual inspection will have to be carried out periodically to confirm that the temporary slopes and final reconstructed slopes will remain stable throughout and after construction.

THURBER ENGINEERING LTD.

Report Preparation by:
Sydney Pang, P.Eng.,
Associate, Senior Foundations Engineer



Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact



Appendix A

Record of Borehole Sheets

19-1351-221

RECORD OF BOREHOLE No PS-01

1 OF 1

METRIC

2365-09-00 LOCATION Pine St. Underpass N 4 775 226.1 E 329 010.1 ORIGINATED BY RK
HWY 58 BOREHOLE TYPE Solid Stem Augers/CME 75 COMPILED BY AN
DATUM Geodetic DATE 2012 05 29 - 2012 05 29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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	20 40 60 80 100	20 40 60 80 100		
178.2													
0.0	CONCRETE with rebar. (125mm)												
0.1	SAND and GRAVEL Brown Moist (FILL)						178						
177.4													
0.8	Silty CLAY, trace sand Stiff to Very Stiff Brown (TILL)		1	SS	18		177						
			2	SS	19								0 3 50 47
			3	SS	12		176						
			4	SS	23		175						
							174						
	Becomes grey		5	SS	17		173						
							172						0 7 52 41
171.6			6	SS	18								
6.6	END OF BOREHOLE AT 6.6m UPON AUGER REFUSAL. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.6m, DRY CONCRETE TO 0.15m, THEN ASPHALT COLDPATCH TO SURFACE.												

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-02

1 OF 2

METRIC

2365-09-00 LOCATION Pine St. Underpass N 4 775 184.4 E 329 009.7 ORIGINATED BY RK
HWY 58 BOREHOLE TYPE Solid Stem Augers/CME 75/NXL Conng COMPILED BY AN
DATUM Geodetic DATE 2012.05.28 - 2012.05.28 CHECKED BY LPG

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			20 40 60 80 100	20 40 60 80 100					GR	SA	SI	CL
178.2																	
0.0	CONCRETE with rebar: (125mm)						178										
0.1	Gravelly SAND																
177.4	Brown Moist (FILL)																
0.8	Silty CLAY, trace sand Stiff to Very Stiff Brown (TILL)		1	SS	13		177										
			2	SS	19		176										
			3	SS	14		175							0	0	66	34
			4	SS	14		174										
	Becomes grey		5	SS	11		173							0	1	53	46
			6	SS	18		172										
171.3	Auger refusal at 6.9m						171						FI				
6.9	Slightly weathered, thinly bedded, occasional voids, grey, very strong, DOLOSTONE BEDROCK Vertical joint at: 6.8m depth (150mm) 7.3m depth (150mm)		1	RUN			170						1				
			2	RUN			169						2				
													0				
													2				
168.3																	

Continued Next Page

+ 3 x 3 Numbers refer to
Sensitivity 20 15 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-02

2 OF 2

METRIC

2365-09-00

LOCATION

Pine St. Underpass N 4 775 184.4 E 329 009.7

ORIGINATED BY RK

HWY 58

BOREHOLE TYPE

Solid Stem Augers/CME 75/NXL Coring

COMPILED BY AN

DATUM Geodetic

DATE

2012.05.28 - 2012.05.28

CHECKED BY LPG

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _P	W	W _L		
9.9	<p>Continued From Previous Page</p> <p>END OF BOREHOLE AT 9.9m. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.6m, DRY CONCRETE TO 0.15m, THEN ASPHALT COLDPATCH TO SURFACE.</p>																

RECORD OF BOREHOLE No PS-03

1 OF 1

METRIC

2365-09-00

LOCATION

Pine St. Underpass N 4 775 184.2 E 329 022.1

ORIGINATED BY RK

HWY 58

BOREHOLE TYPE Solid Stem Augers/CME 75

COMPILED BY AN

DATUM Geodetic

DATE

2012.05.29 - 2012.05.29

CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
178.2														
0.0	CONCRETE with rebar: (125mm)													
0.1	Gravelly SAND Brown Moist (FILL)													
177.3														
0.9	Silty CLAY, trace sand Stiff to Very Stiff Brown (TILL)		1	SS	10									
			2	SS	19									
			3	SS	15									
			4	SS	12									
	Becomes grey		5	SS	12									
	Some sand, trace gravel		6	SS	24									
171.6														
6.6	END OF BOREHOLE AT 6.6m UPON AUGER REFUSAL. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul.16/12 5.5 172.7													

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-04

1 OF 1

METRIC

2365-09-00

LOCATION

Pine St. Underpass N 4 775 160.0 E 329 004.7

ORIGINATED BY DA

HWY 58

BOREHOLE TYPE Solid Stem Augers/CME 75

COMPILED BY AN

DATUM Geodetic

DATE

2012.04.16 - 2012.04.16

CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
172.4														
0.0	ASPHALT: (175mm)													
0.2	Silty CLAY, some sand, trace gravel Stiff Brown (FILL)		1	SS	12		172							
170.9							171							
1.5	Silty SAND, trace to some gravel Compact Brown Moist (FILL)		2	SS	10									
170.2			3	SS	50/									
2.2	END OF BOREHOLE AT 2.2m UPON AUGER REFUSAL. WATER LEVEL AT 2.1m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.6m, DRY CONCRETE TO 0.15m, THEN ASPHALT COLDPATCH TO SURFACE.				0.075									

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-05

1 OF 1

METRIC

W.P. 2365-09-00 LOCATION Pine St. Underpass N 4 775 159.8 E 329 028.0 ORIGINATED BY DA
HWY 58 BOREHOLE TYPE Solid Stem Augers/CME75/NXL Coring COMPILED BY AN
DATUM Geodetic DATE 2012.04.17 - 2012.04.17 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
172.3														
0.0	ASPHALT: (210mm)													
0.2	SAND and GRAVEL, trace silt, trace clay Dense Brown Moist (FILL)		1	AS			172							
			1	SS	38									
170.7			2	SS	50/		171							
1.6	Moderately weathered, thinly bedded, grey, very strong, DOLOSTONE BEDROCK Broken cores at 1.7m, 1.8m Vertical joint from 1.8m to 1.9m Horizontal joints at 2.4m, 3.4m, 3.5m Calcite interbed (25mm thick) at 4.5m		1	RUN	0.100		170							
			2	RUN										
			3	RUN			169							
167.6	Horizontal joints at 4.5m						168							
4.7	END OF BOREHOLE AT 4.7m. NO FREE WATER IN BOREHOLE UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.6m, THEN ASPHALT COLDPATCH TO THE SURFACE.													

+³ . x³ : Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-06

1 OF 1

METRIC

2365-09-00 LOCATION Pine St. Underpass N 4 775 131.2 E 329 010.7 ORIGINATED BY RK
HWY 58 BOREHOLE TYPE Solid Stem Augers/CME 75 COMPILED BY AN
DATUM Geodetic DATE 2012.05.29 - 2012.05.29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			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	20 40 60 80 100		
178.2												
0.0	CONCRETE with rebar:(125mm)											
0.1	Gravelly SAND						178					
177.4	Brown Moist (FILL)											
0.8	Silty CLAY, trace sand Very Stiff to Stiff Brown (TILL)		1	SS	19		177					
			2	SS	20		176					0 0 43 57
			3	SS	13		175					
			4	SS	13		174					
	Becomes grey		5	SS	8		173					
			6	SS	13		172					0 3 52 45
170.9							171					
7.3	END OF BOREHOLE AT 7.3m UPON AUGER REFUSAL. BOREHOLE OPEN AND DRY UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul.16/12 7.1 171.1											

ONTMT4S 1221 GPJ 1/3/13

+ 3 , × 3 : Numbers refer to
Sensitivity 20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-07

1 OF 1

METRIC

2365-09-00

LOCATION Pine St. Underpass N 4 775 103.2 E 329 023.2

ORIGINATED BY RK

HWY 58

BOREHOLE TYPE Solid Stem Augers/CME 75

COMPILED BY AN

DATUM Geodetic

DATE 2012.05.29 - 2012.05.29

CHECKED BY LPG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100		
178.2													
0.0	CONCRETE with rebar. (125mm)												
0.1	SAND and GRAVEL Brown Moist (FILL)												
177.4													
0.8	Silty CLAY, trace sand Very Stiff Brown (TILL)		1	SS	21								
			2	SS	22								
			3	SS	26								
			4	SS	23								
	Becomes grey		5	SS	16								0 4 53 43
	Becomes stiff		6	SS	13								0 4 73 23
170.7													
7.5	END OF BOREHOLE AT 7.5m UPON AUGER REFUSAL. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.6m, DRY CONCRETE TO 0.15m, THEN ASPHALT COLDPATCH TO SURFACE.												

ONTMT4S 1221.GPJ 1/3/13

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-08

1 OF 2

METRIC

2365-09-00

LOCATION

Pine St. Underpass N 4 775 131.0 E 329 023.2

ORIGINATED BY RK

HWY 58

BOREHOLE TYPE

Solid Stem Augers/CME 75/NXL Coring

COMPILED BY AN

DATUM Geodetic

DATE

2012.05.28 - 2012.05.28

CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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	20 40 60 80 100	20 40 60 80 100		
178.2													
0.0	CONCRETE: (125mm)												
0.1	Gravelly SAND Loose to Very Loose Brown Wet (FILL) Silty clay layer from 0.9m to 1.2m		1	SS	8		178						
			2	SS	2		177						
175.9							176						
2.3	Silty CLAY Stiff to Very Stiff Brown (TILL)		3	SS	11								0 0 58 42
			4	SS	17		175						
	Becomes grey		5	SS	8		174						
							173						
	Some sand, trace gravel		6	SS	15		172						0 11 60 29
171.2							171						
7.0	Silty SAND, some gravel, some clay Very Dense Grey Moist (TILL)		7	SS	65								
170.0	Auger refusal at 8.2m						170						
8.2	Slightly weathered, thinly bedded, grey, very strong, DOLOSTONE BEDROCK		1	RUN			169						
	Sub-horizontally joints												

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PS-08

2 OF 2

METRIC

2365-09-00 LOCATION Pine St. Underpass N 4 775 131.0 E 329 023.2 ORIGINATED BY RK
HWY 58 BOREHOLE TYPE Solid Stem Augers/CME 75/NXL Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.28 - 2012.05.28 CHECKED BY LPG

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	W _P W W _L	WATER CONTENT (%)	20 40 60			
Continued From Previous Page														
167.0			2	RUN			168						0	SCR=90% RQD=75% UCS=185MPa (Average)
													3	
													0	
													2	
11.2	END OF BOREHOLE AT 11.2m. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.7m, DRY CONCRETE TO 0.15m, THEN ASPHALT COLDPATCH TO SURFACE.													

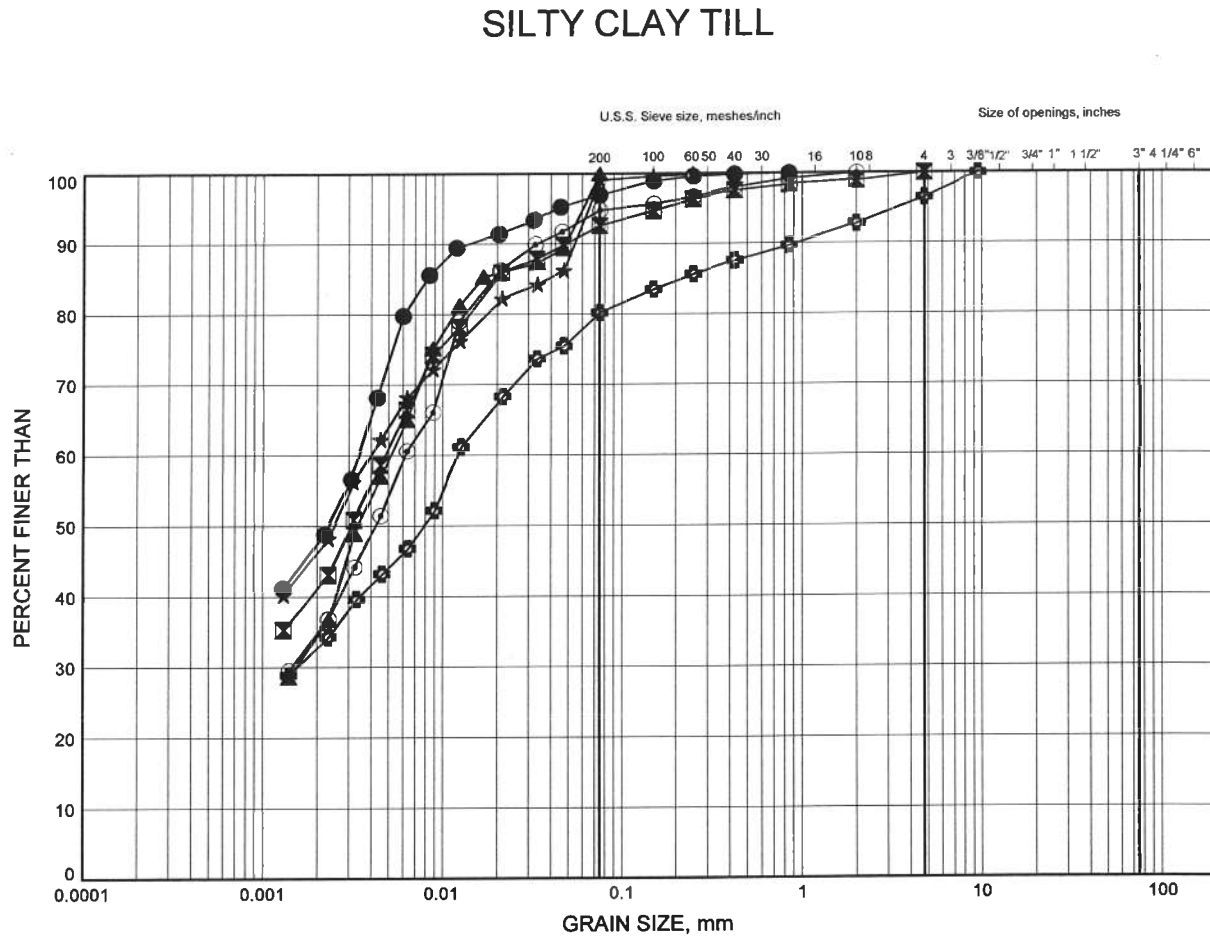
Appendix B

Laboratory Test Results

19-1351-221

5 Bridges, Welland and St. Catharines
GRAIN SIZE DISTRIBUTION

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PS-01	1.83	176.37
⊠	PS-01	6.32	171.88
▲	PS-02	2.59	175.61
★	PS-02	4.88	173.32
⊙	PS-03	3.35	174.85
⊕	PS-03	6.40	171.80

GRAIN SIZE DISTRIBUTION - THURBER 1221.GPJ 11/29/12

Date November 2012
 W.P.# 2365-09-00

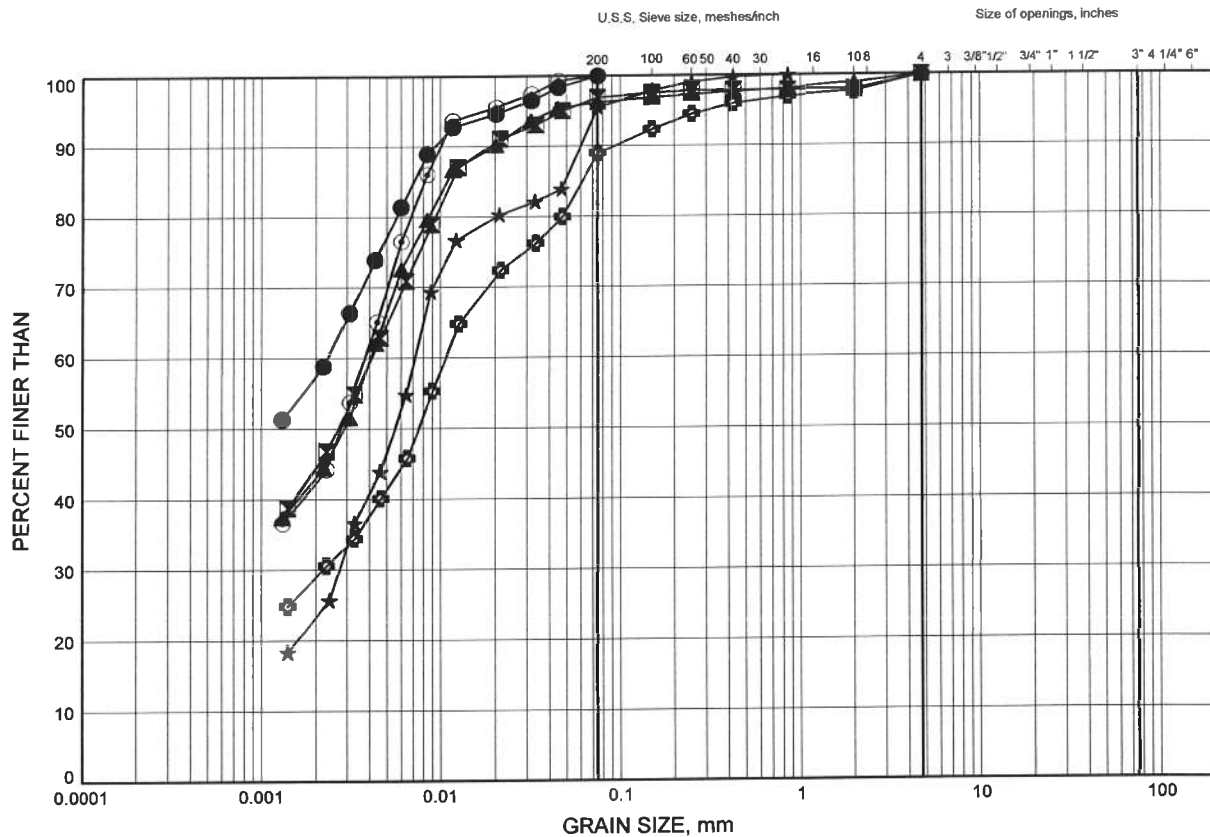


Prep'd AN
 Chkd. SKP

5 Bridges, Welland and St. Catharines GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PS-06	1.83	177.05
■	PS-06	6.40	172.48
▲	PS-07	2.59	175.61
★	PS-07	4.88	173.32
○	PS-08	2.59	175.61
⊕	PS-08	6.40	171.80

Date November 2012

W.P.# 2365-09-00

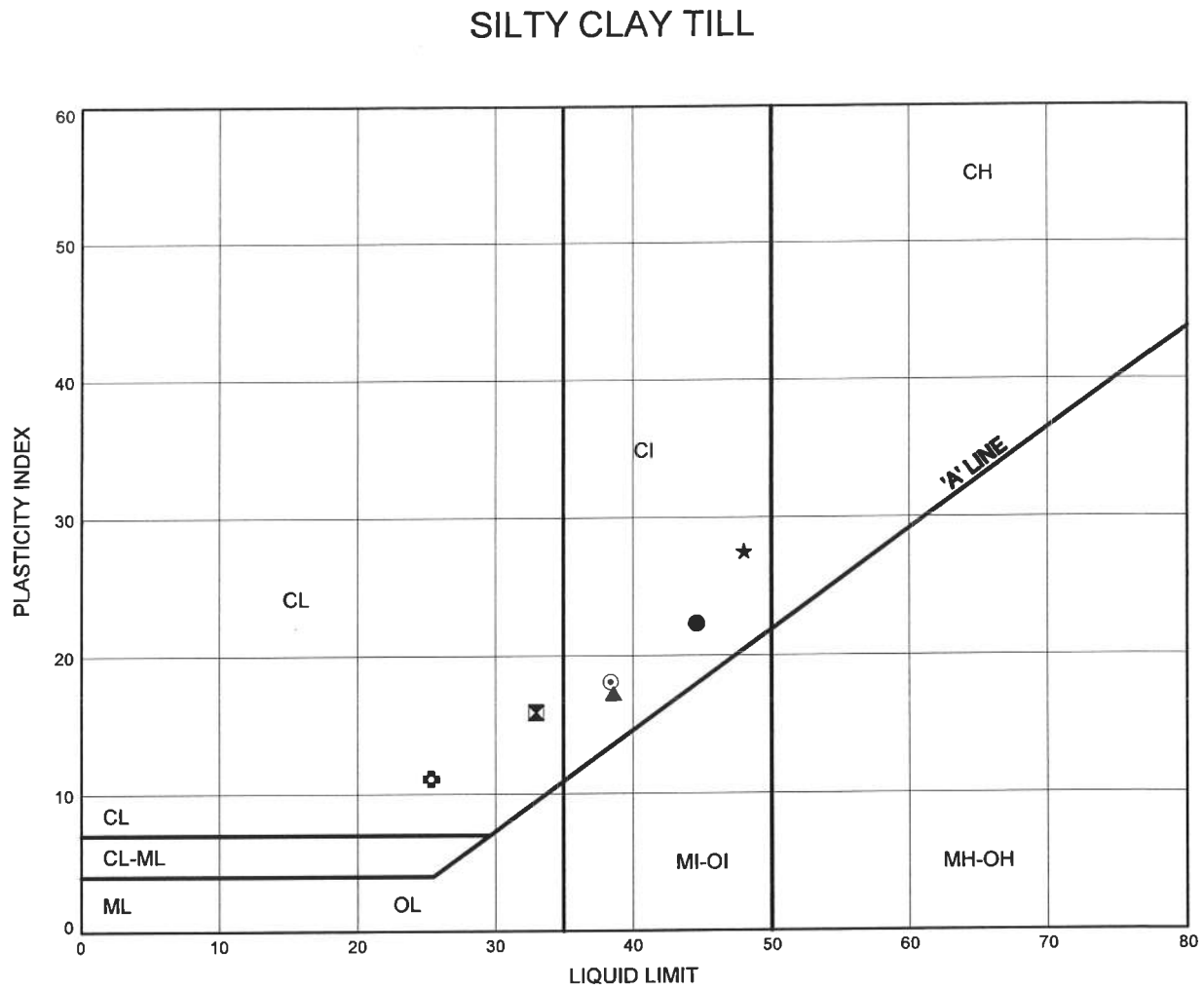


Prep'd AN

Chkd. SKP

5 Bridges, Welland and St. Catharines
ATTERBERG LIMITS TEST RESULTS

FIGURE B3



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PS-01	1.83	176.37
⊠	PS-01	6.32	171.88
▲	PS-02	2.59	175.61
★	PS-02	4.88	173.32
⊙	PS-03	3.35	174.85
⊕	PS-03	6.40	171.80

Date November 2012

W.P.# 2365-09-00



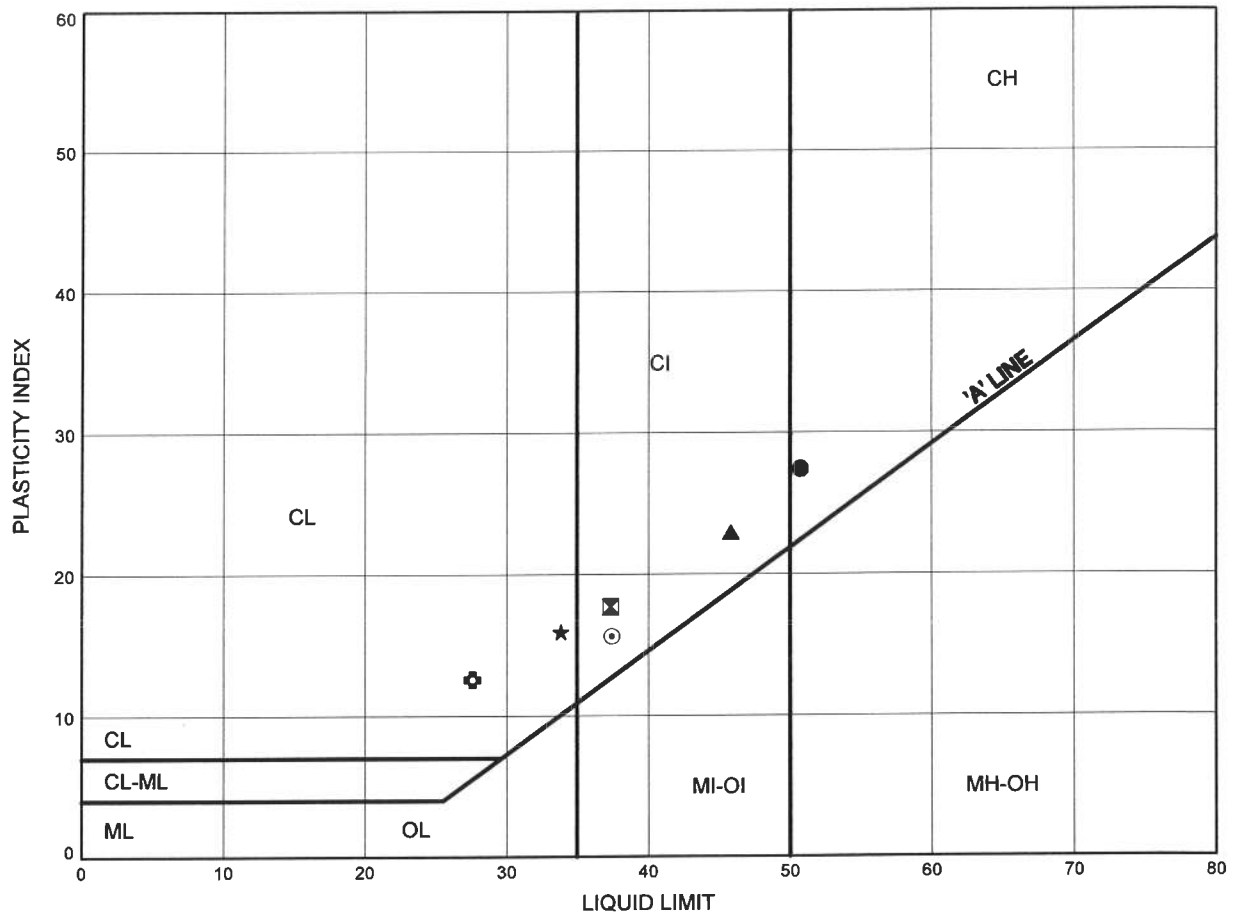
Prep'd AN

Chkd. SKP

5 Bridges, Welland and St. Catharines
ATTERBERG LIMITS TEST RESULTS

FIGURE B4

SILTY CLAY TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PS-06	1.83	177.05
⊠	PS-06	6.40	172.48
▲	PS-07	2.59	175.61
★	PS-07	4.88	173.32
⊙	PS-08	2.59	175.61
⊕	PS-08	6.40	171.80

Date November 2012
W.P.# 2365-09-00

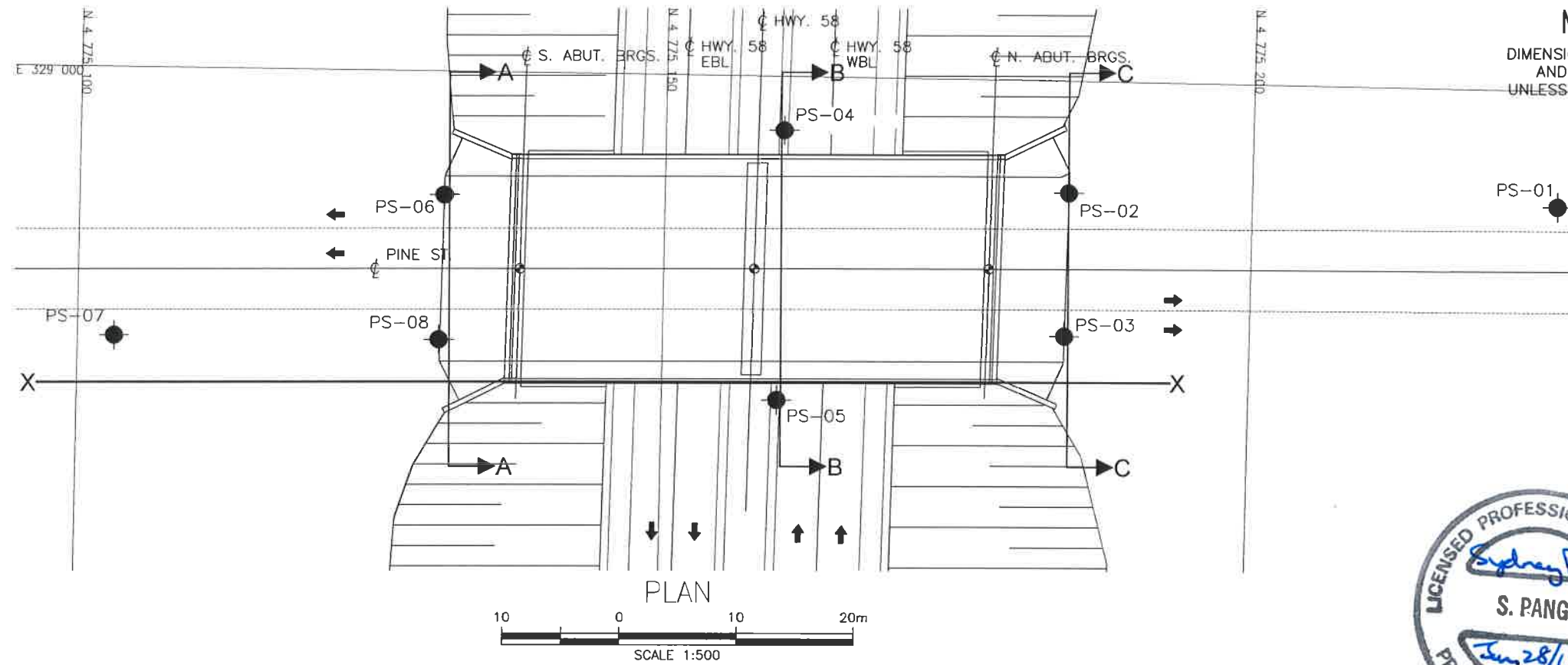


Prep'd AN
Chkd. SKP

Appendix C

Drawings titled “Borehole Locations and Soil Strata”

19-1351-221



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



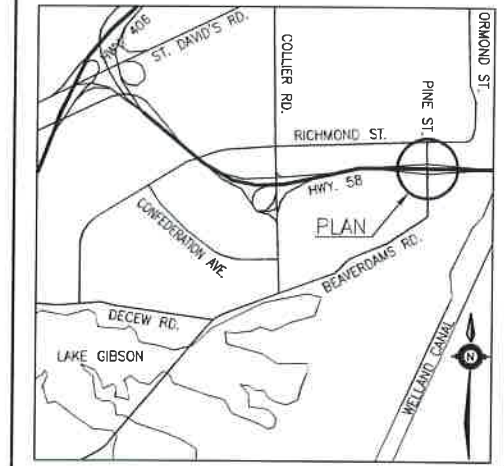
CONT No
GWP No 2365-09-00
WP No 2367-09-01

HIGHWAY 58
PINE STREET UNDERPASS
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

MRC McCORMICK RANKIN
A member of MMR GROUP

THURBER ENGINEERING LTD.

SHEET
73



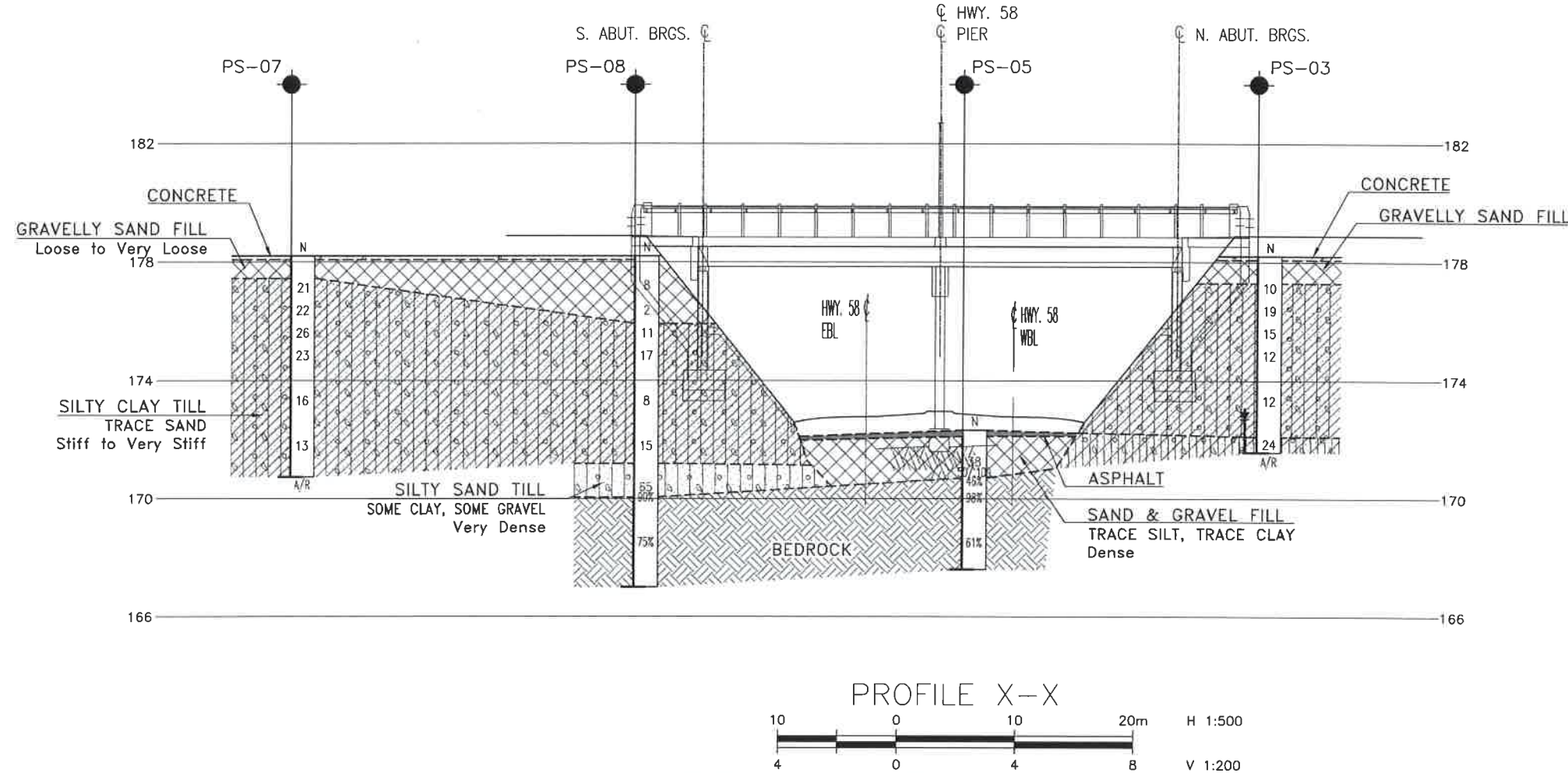
LEGEND

◆	Borehole
◆	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60' Cone, 475J/blow)
PH	Pressure, Hydraulic
↓	Water Level
↑	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
PS-01	178.2	4 775 226.1	329 010.1
PS-02	178.2	4 775 184.4	329 009.7
PS-03	178.2	4 775 184.2	329 022.1
PS-04	172.4	4 775 160.0	329 004.7
PS-05	172.3	4 775 159.8	329 028.0
PS-06	178.2	4 775 131.2	329 010.7
PS-07	178.2	4 775 103.2	329 023.2
PS-08	178.2	4 775 131.0	329 023.2

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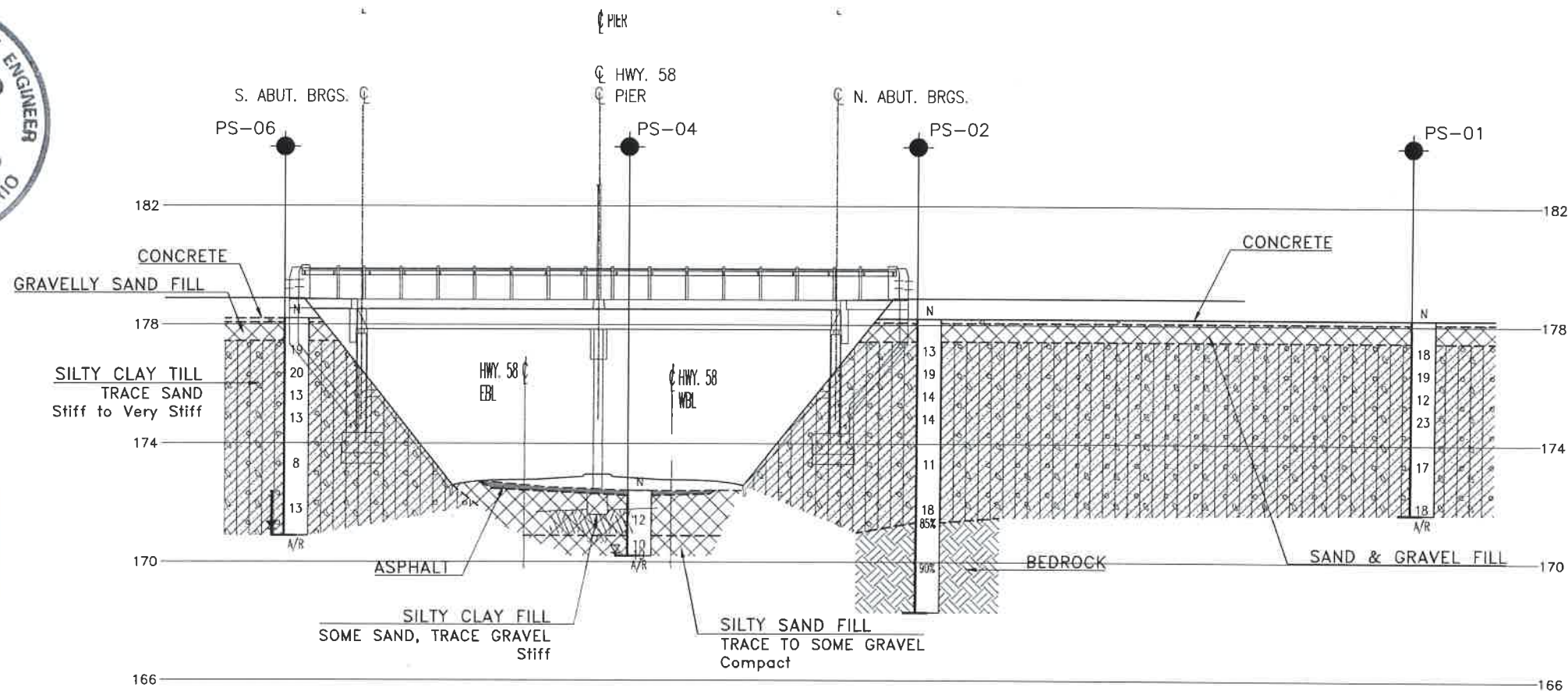
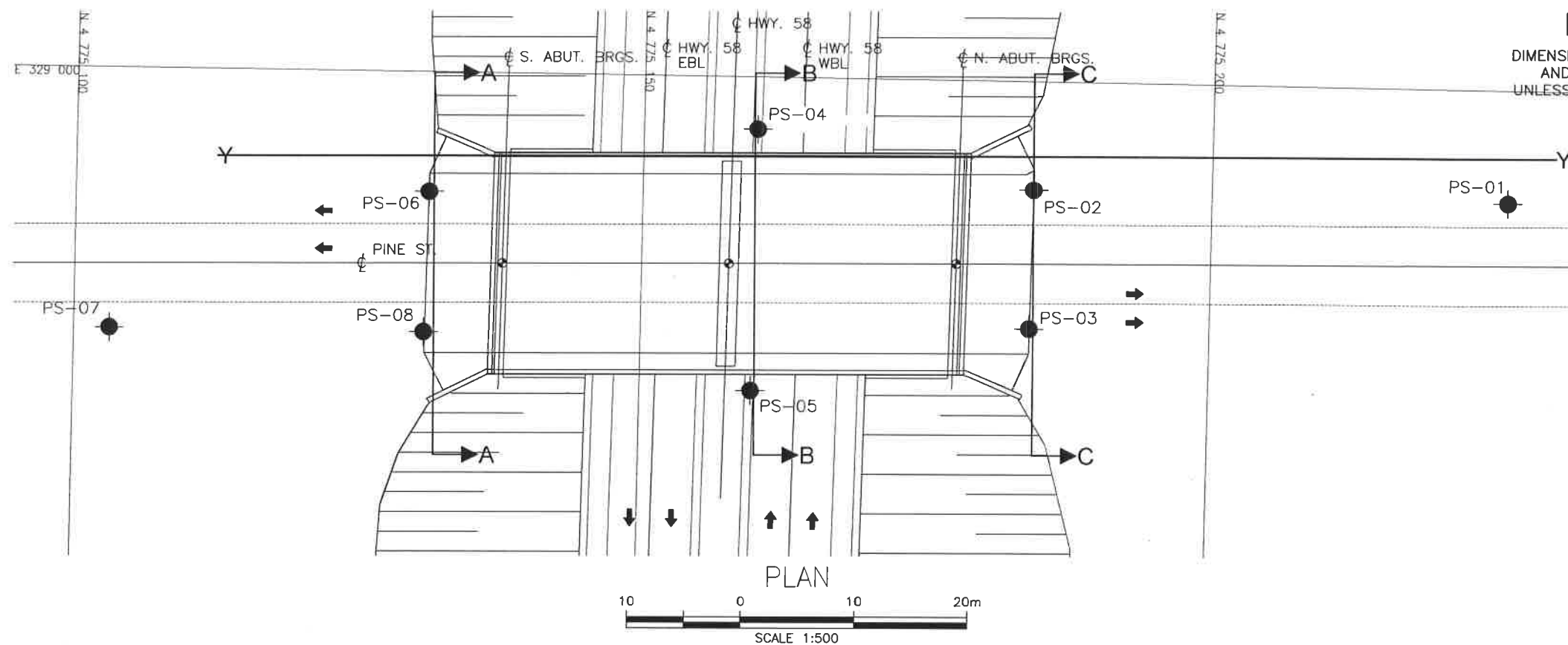
GEOCRES No. 30M3-275



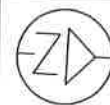
REVISIONS

DATE	BY	DESCRIPTION
DESIGN LG	CHK SKP	CODE
DRAWN MFA	CHK PKC	SITE 34-179
STRUCT	DWG 2	

DATE JAN. 2013



CONT No
GWP No 2365-09-00
WP No 2367-09-01



HIGHWAY 58
PINE STREET UNDERPASS
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

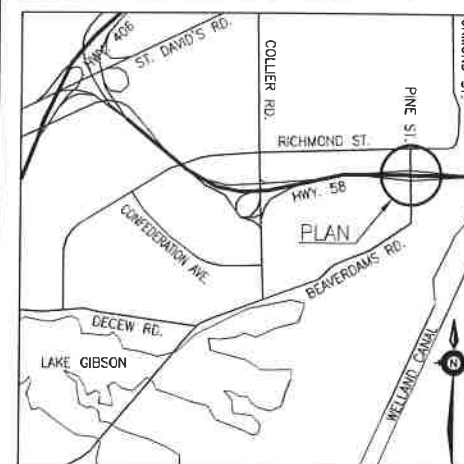
SHEET
74



McCORMICK RANKIN
A member of MMM GROUP



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

- ◆ Borehole
- ◆ Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60' Cone, 475J/blow)
- PH Pressure, Hydraulic
- W Water Level
- W Head Artesian Water
- P Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
PS-01	178.2	4 775 226.1	329 010.1
PS-02	178.2	4 775 184.4	329 009.7
PS-03	178.2	4 775 184.2	329 022.1
PS-04	172.4	4 775 160.0	329 004.7
PS-05	172.3	4 775 159.8	329 028.0
PS-06	178.2	4 775 131.2	329 010.7
PS-07	178.2	4 775 103.2	329 023.2
PS-08	178.2	4 775 131.0	329 023.2

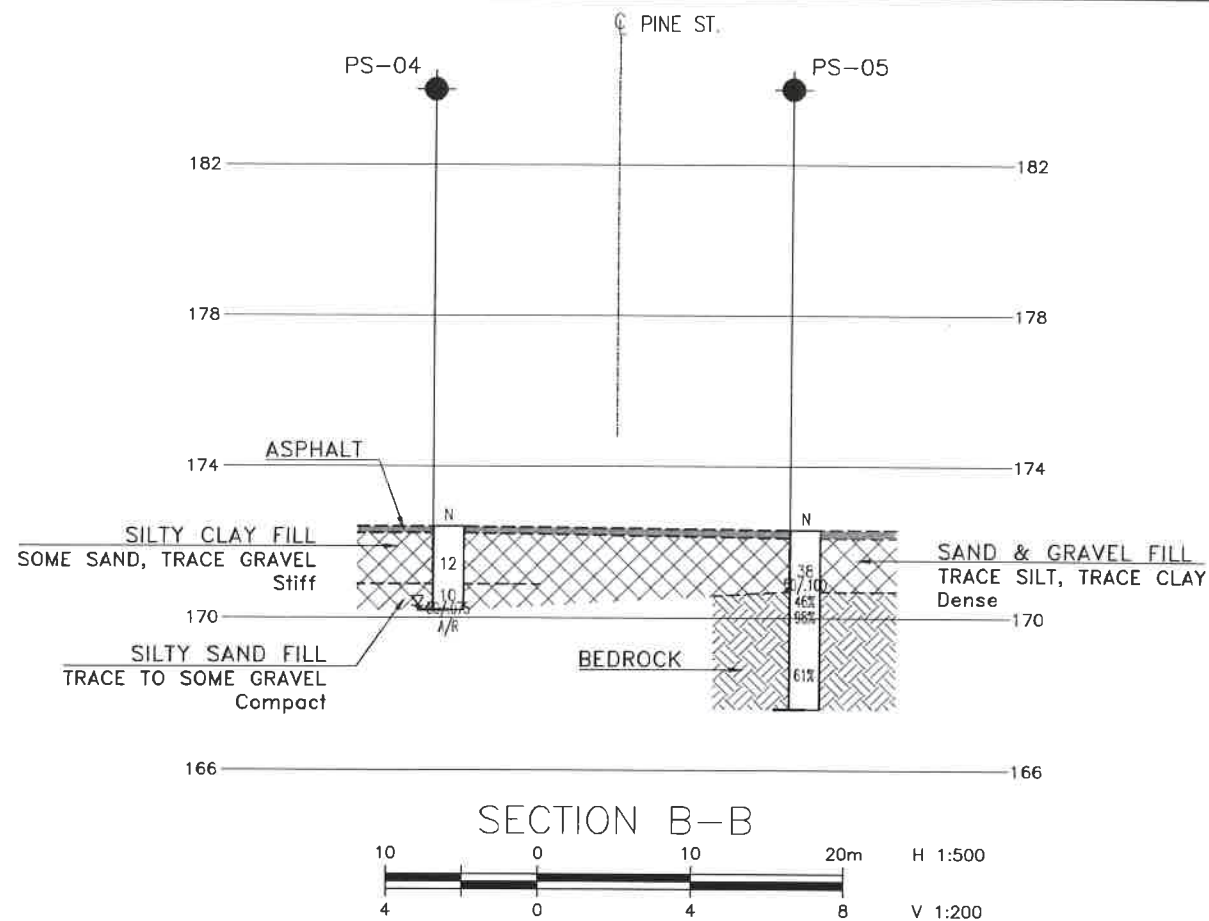
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GEOCRE No. 30M3-275

REVISIONS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			</
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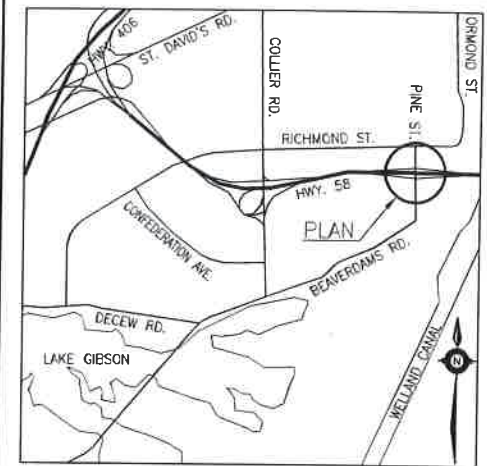
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
GWP No 2365-09-00
WP No 2367-09-01

HIGHWAY 58
PINE STREET UNDERPASS
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
75



KEYPLAN
LEGEND

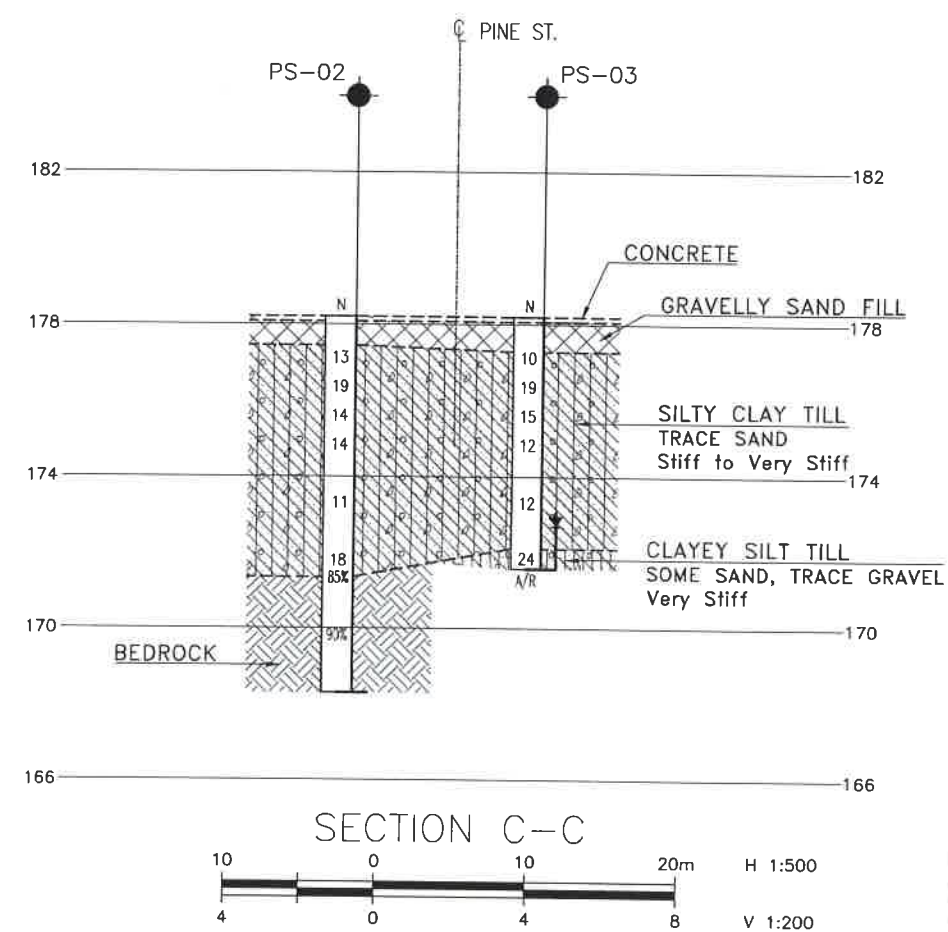
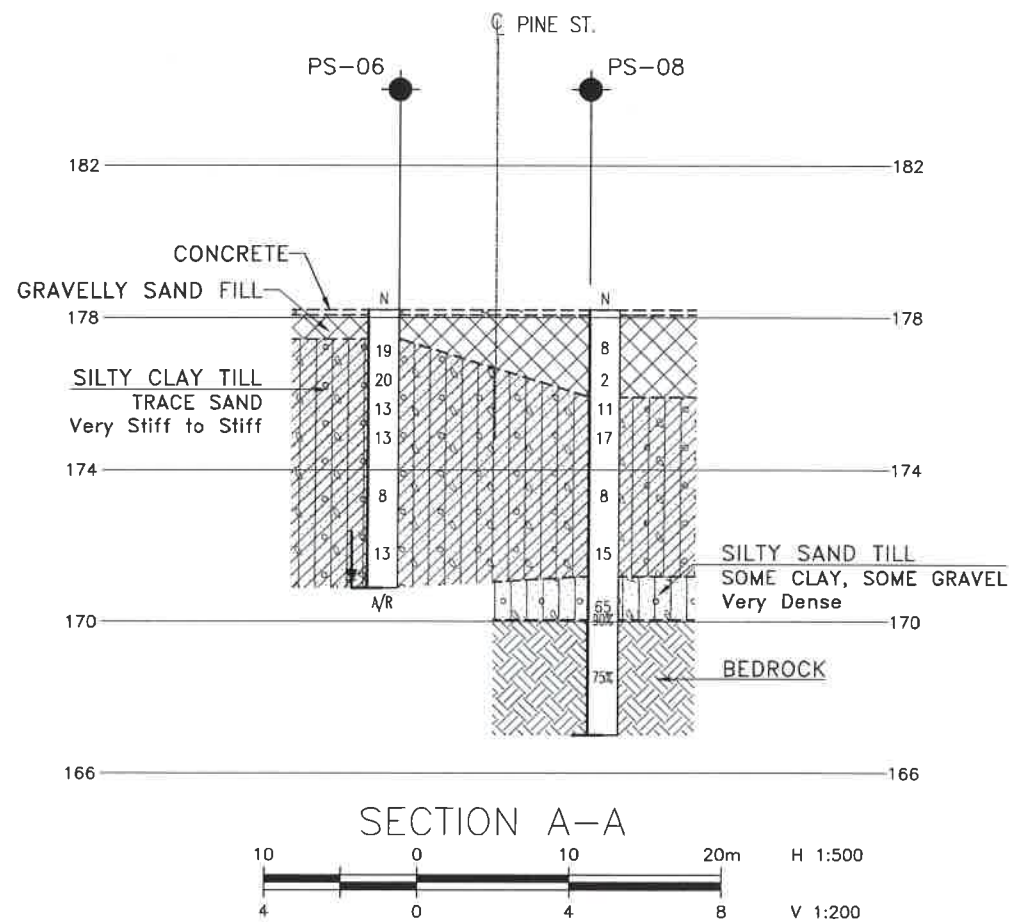
◆	Borehole
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GEOCREs No. 30M3-275



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	LG	CHK SKP	CODE
DRAWN	MFA	CHK PKC	SITE 34-179
LOAD	DATE	JAN. 2013	
STRUCT	JDWG	4	