APPENDIX 'A'

GEOTECHNICAL/TEST CAISSON/BEDROCK REPORTS

City of Winnipeg

Waverley Street Underpass Upgrade
Preliminary Design
Geotechnical Report - Draft

Prepared by:

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Project Number:

60321148 (400.300.9)

Date:

January, 2015

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AECOM: 2012-01-06

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January 12, 2015

City of Winnipeg
c/o Mr. Mike Lau, Ph.D, P.Eng.
Partner
Dillon Consulting Limited
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Dear Mr. Lau:

Project No: **60321148 (403.19)**

Regarding: **Waverley Street Underpass Upgrade – Preliminary Design Geotechnical Report**

AECOM Canada Ltd. (AECOM) is pleased to submit our report for the above noted project.

Should you have any questions or require any additional information, please contact Faris Khalil at (204) 477-5381, directly

Sincerely,
AECOM Canada Ltd.

Ron Typliski, P.Eng.
Vice-President, Environment
Canada West Region

FK:cm

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Revision Log

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1. Introduction

The City of Winnipeg (The City) retained Dillon Consulting Limited (Dillon) and AECOM Canada Limited (AECOM) to provide preliminary design services for the proposed Waverley Street Underpass Upgrade. The proposed Waverley Street Underpass will replace the existing at-grade crossing of Waverley Street and CN Rivers Subdivision located between Taylor Avenue and Sterling Lyon Parkway. Along with the underpass construction, geometric and capacity improvements will be introduced at Taylor Avenue, Taylor Avenue and Waverley Street intersection and Wilkes Avenue/Hurst Way and Waverly Street intersection. The project will also include Active Transportation components and construction of retaining walls and new lift station and associated sewer line. Railway and road detours will be required during the construction period to facilitate the construction activities. General arrangement plan view and typical sections are illustrated on the conceptual project drawings in Appendix A. The key objectives of the project are to provide improvements in traffic operations, road safety and mobility.

This report documents the 2014 geotechnical investigation, discusses the geotechnical considerations, identifies design alternatives and provides related geotechnical recommendations in support of the preliminary design phase. Further geotechnical/hydro-geological investigation, full scale pile installation testing and comprehensive geotechnical engineering effort and hydro-geological studies will be required to supplement the assessment provided in this report and support the detailed design and construction phases.

The report is structured as follows:

1. Introduction
2. Geotechnical Investigation
    - Description of the completed field work and subsurface and groundwater conditions.
3. Foundations
    - Discussion of foundation alternatives for the underpass structure, lift station and retaining walls.
4. Temporary Excavations and Shoring
    - Discussion of available excavation support alternatives and geotechnical concerns associated with temporary excavations.
5. Permanent Excavations
    - Discussion of geotechnical concerns associated with permanent excavations including slope stability assessment.
6. Buoyancy and Uplift
    - Brief discussion of buoyancy concerns for buried structures.
7. Retaining Walls
    - Discussion of available wall alternatives and lateral loads.
8. Trenchless Pipe Installation
    - Description of locally available installation techniques and related concerns.
9. Road Subgrade
    - Subgrade characterization and preparation discussion.
10. Railway Detour
    - Discussion of railway grade design and construction.
11. Recommendations and Future Work
    - Summarize key recommendations and future work required.

The underpass structure foundation recommendations were prepared following the guidance of AREMA 2012. Limit State Design in accordance with the principles of AASHTO 2014 and CAN/CSA 2006 was adopted in preparing the recommendations for the lift station and retaining walls.

2. Geotechnical Investigation

2.1 Field Work

To accommodate the design development, the evolution of design options and to maintain project schedule it was necessary to undertake a staged approach to complete the field work. The field work was completed in three phases (I, II and III) as follows:

Phase I
Phase I drilling was completed during the period from July 09 to 15, 2014 and consisted of one intermediate test hole (TH14-01) and three deep test holes (TH14-02 to 14-04). The intermediate test hole was located at the southeast corner of Taylor Avenue/Waverley Street intersection in the vicinity of a proposed retaining wall close to the boundary of Piaza De Nardi property. The deep test holes were located at both ends of the proposed underpass structure. The drilling was completed using a track mounted rig operated by Maple Leaf Drilling equipped with 125 mm diameter solid stem augers and HQ wireline for rock coring. The intermediate test hole (TH14-01) was terminated after auger refusal into glacial till at 13.2 m below existing grade. The deep test holes were advanced more than 6 m into bedrock to depths range from 24.4 m to 25.7 m below existing grade.

Phase II
Phase II drilling was completed during the period from October 23 to 26, 2014 and consisted of twenty three shallow test holes (TH14-05 to 14-27) and one intermediate test hole (TH14-28). The shallow test holes were located along the proposed railway and road detours and along the proposed road improvement /widening. The intermediate test hole was located at the southwest corner of CN track/Waverley Street crossing at one of the two locations being considered for the proposed lift station. The drilling was completed using a truck mounted rig operated by Maple Leaf Drilling equipped with 125 mm diameter solid stem augers. The shallow test holes were advanced to depths range from 2.5 to 4 m below existing grade. The intermediate test hole (TH14-28) was terminated after auger refusal into glacial till at 13.9 m below existing grade.

Phase III
Phase III drilling was completed during the period from December 01 to 02, 2014 and consisted of one intermediate test hole (TH14-29) and three shallow test holes (TH14-30 to 14-32). The intermediate test hole was located at the northwest corner of CN track/Waverley Street crossing at one of the two locations being considered for the proposed lift station. The shallow test holes were located at the northeast corner of CN track/Waverley Street crossing along the proposed railway detour where soft dig using hydrovac excavation was required to protect shallow underground utilities. The drilling was completed using a track mounted rig operated by Maple Leaf Drilling equipped with 125 mm diameter solid stem augers. The intermediate test hole (TH14-29) was terminated after auger refusal into glacial till at 15.8 m below existing grade. The shallow test holes were advanced to maximum depth of 4.5 m below existing grade.

During the course of the investigation, Standard Penetration Test (SPT) was completed at regular intervals in the till. Disturbed and relatively undisturbed soil samples and rock cores were collected for further visual classification and testing. Five standpipe piezometers were installed within the project area to monitor the groundwater conditions. These included two standpipe piezometers (SP14-02 and 14-04) installed in the bedrock unit, two standpipe piezometers (SP14-01 and 14-28) installed in the clay unit and one standpipe piezometer (SP14-29) installed in the till unit. Laboratory testing were completed on selected samples and included moisture content, unit weight, gradation, Atterberg limits, undrained shear strength, consolidation test and uniaxial compressive strength for rock cores.

Drilling supervision was provided by AECOM personnel, who visually classified and logged soils, retrieved samples for laboratory testing, and supervised in-situ soil testing and standpipe piezometers installation. The approximate location of the test holes are shown on the Test Hole Location Plan in Appendix A. Test hole logs have been prepared for each test hole to record the description and the relative position of the soil strata, location of samples obtained, seepage and sloughing conditions, field and laboratory test results, and other pertinent information. The test hole logs are attached in Appendix B. The laboratory test results are recorded on the test hole logs and are attached in Appendix C.

2.2 Subsurface Conditions

In descending order the soil profile consists of:

- Fill;
- Glacio-Lacustrine Clay;
- Glacial Till; and
- Limestone Bedrock.

Each of these units is described below. Schematics of soil stratigraphy based on conditions encountered during the investigation are presented on Schematic 01 and 02 in Appendix A. Soil properties from field and laboratory test results are presented on Figure 01.

Fill

Fill was encountered at the ground surface or beneath a thin layer of topsoil in most of the test holes and extended up to 1.5 m below ground surface. Two distinctive zones of fill were observed: an upper granular fill and lower clay fill.

The granular fill was 0.2 to 1.1 m thick, mainly encountered in test holes drilled along existing roads and railway track. The granular fill predominantly consisted of sand and gravel sizes, and contained variable amounts of silt, some clay and trace organic. Cobbles and concrete debris was observed within the granular fill. The fill was light grey to light brown and dry to moist. Moisture contents measured on two samples from the granular fill were 6 and 20 percent.

The clay fill, where encountered, was 0.2 to 1.5 m thick and contained variable amounts of silt, sand, organics, some to trace amounts of gravel and trace oxidation. The clay fill was dark grey to dark brown, moist, firm and was visually classified as of high to intermediate plasticity. Measured moisture contents range from 22 to 41 percent.

Glacio-Lacustrine Clay

In all test holes advanced past the fill zone, the fill was underlain by 10 to 11 m thick galcio-lacustrine silty clay. Generally, the clay was brown changing to grey with increasing depth, firm to stiff becoming soft with increasing depth, moist and of high plasticity. Silt layer(s) about 1.0 m thick, firm to very soft and moist was observed in the upper portion of the clay unit or beneath the fill.

Moisture contents ranged from 21 to 59 percent. The average bulk unit weight of the clay was 17 kN/m^3. Undrained shear strength values measured from unconfined compression test were 30 to 41 kPa.

Glacial Till (Silt)

In all test holes advanced past the clay, the clay was underlain by glacial till that typically contains variable amounts of clay, sand and gravel in silt matrix. Boulders and cobbles are known to be present within the till unit and were encountered during the drilling. Where the drilling advanced below the till unit, the thickness of the till layer varies from 4.8 to 5.2 m. The till was light grey, soft/loose in the upper zone but became dense to very dense with increasing depth. Coring was necessary to advance the drilling through very dense and boulders/cobbles dominated lower zone of the till. The till was moist to wet, and of low plasticity. Measured moisture contents range from 6 to 15 percent.

Limestone Bedrock

The drilling of TH14-02, 14-03 and 14-04 were advanced past the till into the underlaying limestone bedrock, which forms an artesian aquifer. The bedrock formation is a Paleozoic Carbonate rock formation known as the Upper Carbonate Aquifer. The depth to bedrock surface was about 18 m below existing grade or approximately at elevation 215.5 m. A layer of hard clay (shale) infill was encountered within the bedrock at elevation 211.9 and 212.6 m in TH14-03 and 14-04, respectively. The clay infill zone was 0.3 m thick at 3.6 m below bedrock surface in TH14-03 while in TH14-04 it was 0.8 m thick at 2.6 m below bedrock surface. The top 5 m of the bedrock formation was observed as highly decomposed and based on the calculated RQD values for the recovered rock cores, the rock quality was very poor to fair. Low RQD values were calculated over the entire length of rock cores (i.e., 7.8 m) recovered from TH14-04 indicating very poor to poor rock quality. Uniaxial compressive strength tests completed on three samples of rock cores recovered from TH11-02, 14-03 and 14-04 indicate compressive strength of 114, 121 and 194 MPa, respectively. Photographs of the recovered rock cores are presented on Figure 02.

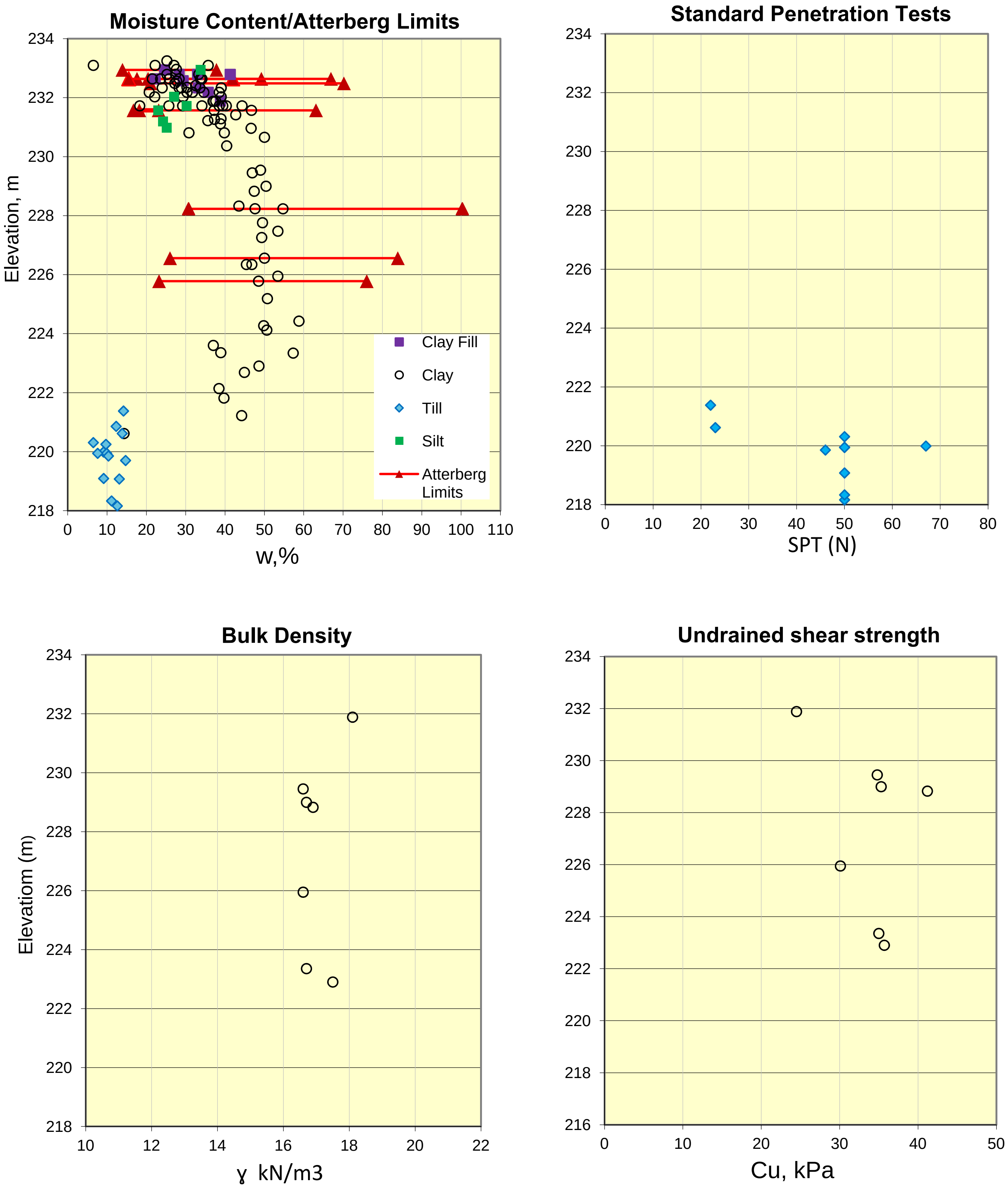


Figure 1 – Field and Laboratory Test Results

Rock Core (Run)	Material	Run Depth below Ground level (m)	RQD (%)
C1	TILL	15.4-16.7	NA
C2	TILL	16.7-17	NA
C3a	TILL	17-17.9	NA
C3b	LIMESTONE	17.9-18.4	65
C4	LIMESTONE	18.4-19.8	25
C5	LIMESTONE	19.8-21.3	43
C6	LIMESTONE	21.3-22.9	29
C7	LIMESTONE	22.9-24.4	93

Rock Core (Run)	Material	Run Depth below Ground level (m)	RQD (%)
C1	TILL	14.3-16.8	NA
C2A	TILL	16.8-18	NA
C2B	LIMESTONE	18-18.3	88
C3	LIMESTONE	18.3-19.8	16
C4	LIMESTONE	19.8-20.4	0
C5	LIMESTONE	20.4-21.8	19
C6	LIMESTONE	21.8-22.8	76
C7	LIMESTONE	22.8-24.4	80

Rock Core (Run)	Material	Run Depth below Ground level (m)	RQD (%)
C1	TILL	13.9-15.2	NA
C2	TILL	15.2-16.8	NA
C3a	TILL	16.8-18	NA
C3B	LIMESTONE	18-18.3	0
C4	LIMESTONE	18.3-19.1	23
C5	LIMESTONE	19.1-20.6	22
C6	LIMESTONE	20.6-21.7	0
C7	LIMESTONE	21.7-22.6	23
C8	LIMESTONE	22.6-24.2	60
C9	LIMESTONE	24.2-25.8	26

Figure 2 – Rock Cores from TH 14-02, 14-03 and 14-04

2.3 Soil Corrosivity

Winnipeg soils are known to contain high contents of sulphates, which can be corrosive when in contact with concrete or cast-iron structures. Table 1 presents a summary of sulphate content, resistivity and pH tests for clay samples. The results indicate high to extremely corrosive condition and at least one test result indicate moderate sulphate attack potential.

All concrete in contact with the soil should be made in accordance with CSA Standard A23.1 and A23.2, sulphate resistant cement is recommended to be used in all concrete structures in contact with the soil.

Table 1 – Summary of Sulphate Content, Resistivity and pH Tests

Soil Unit	Sample Depth (m)	Test hole	Sulphate Content in Soil Sample %	Potential for Sulphate Attack	Resistivity (ohm cm)	pH	Corrosivity Rating
Clay	1.2	TH14-01	0.0187	Negligible	2970	7.93	Highly corrosive
Clay	10	TH14-01	0.1160	Moderate	890	7.99	Extremely corrosive
Clay	3.8	TH14-02	0.0369	Negligible	2870	7.84	Highly corrosive
Clay	6.8	TH14-02	0.1020	Moderate	1430	7.99	Highly corrosive
Clay	2.4	TH14-04	0.0089	Negligible	2340	7.86	Highly corrosive

2.4 Seasonal Frost Penetration

The mean freezing index in the Winnipeg area is estimated at 1900 °C-days, accordingly the seasonal frost penetration depth is approximately 2.4 m. Factors such as snow cover, vegetation at surface, soil type, and groundwater conditions can all significantly impact the depth of frost penetration.

2.5 Groundwater Conditions

Monitoring results of the groundwater level (GWL) from the five standpipe piezometers installed at the site are presented in Table 2 and on Figure 3. Groundwater levels will vary seasonally and from year to year or due to construction activities.

Based on the available short term monitoring results, a GWL between elevation 225 and 225.8 m was recorded in the bedrock piezometers installed in TH14-02 and 14-04. The till is considered to be hydraulically connected to the bedrock aquifer, only two monitoring events recorded for the till piezometer installed in TH14-29, the monitoring will be continued to record additional readings. Monitoring of the clay piezometers installed in TH14-01 and 14-28 recorded a maximum GWL of 226.8 m (i.e., about 6 m below existing grade). This readings need to be confirmed as stabilized GWL in the clay may not have been reached, the monitoring will be continued to record additional readings.

Monitoring results of two Provincial wells for bedrock aquifer GWL over the period from 2005 to 2014 are presented on Figure 4. The short term monitoring results from AECOM installation are in good agreement with the data from well G05oc053 and are close to upper bound data from well G05oc008.

Table 2 – Summary of GWL Monitoring Results

Standpipe ID	Soil Unit	Ground Surface Elevation (m)	Monitoring Date	GWL Elevation (m)
SP14-01	Clay	232.5	12-Aug-14	225.10
			3-Sep-14	224.90
			19-Sep-14	255.55
			17-Oct-14	226.43
			6-Nov-14	226.55
			20-Nov-14	226.53
			6-Dec-14	226.40
			18-Dec-14	226.40
SP14-28	Clay	233.6	6-Nov-14	226.30
			20-Nov-14	226.58
			6-Dec-14	226.60
			18-Dec-14	226.67
SP14-02	Bedrock	233.4	12-Aug-14	225.20
			3-Sep-14	225.07
			19-Sep-14	225.5
			17-Oct-14	225.78
			6-Nov-14	225.65
			20-Nov-14	225.59
			6-Dec-14	224.90
			18-Dec-14	225.40
SP14-04	Bedrock	233.2	12-Aug-14	225.20
			3-Sep-14	225.08
			19-Sep-14	225.55
			17-Oct-14	225.50
			6-Nov-14	225.40
			20-Nov-14	225.36
			6-Dec-14	225.23
			18-Dec-14	225.27
SP14-29	Glacial Till	233.4	6-Dec-14	225.27
			18-Dec-14	225.61

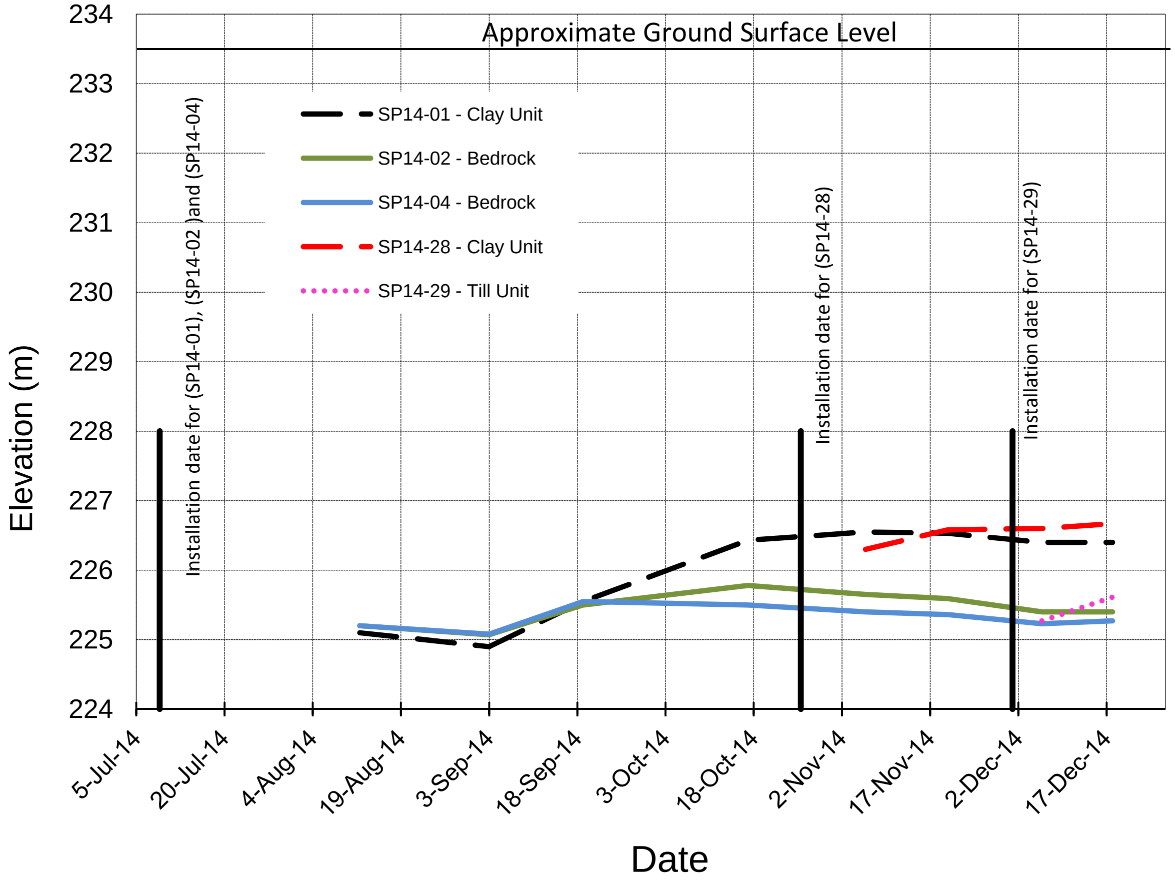
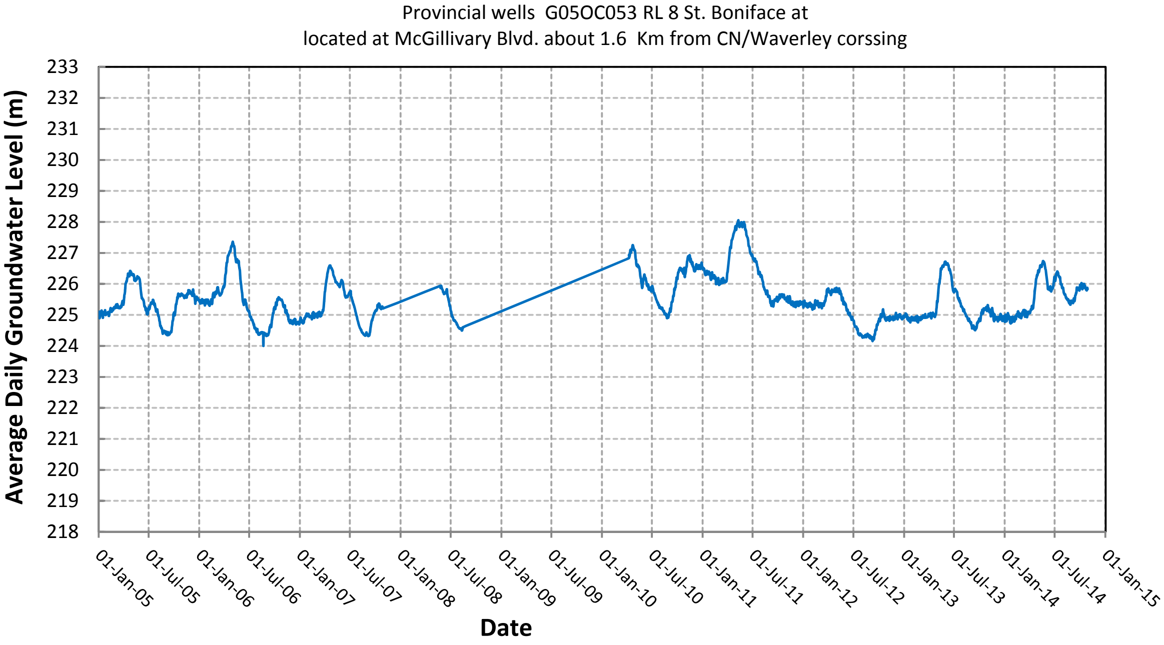


Figure 3 – Groundwater Monitoring Results



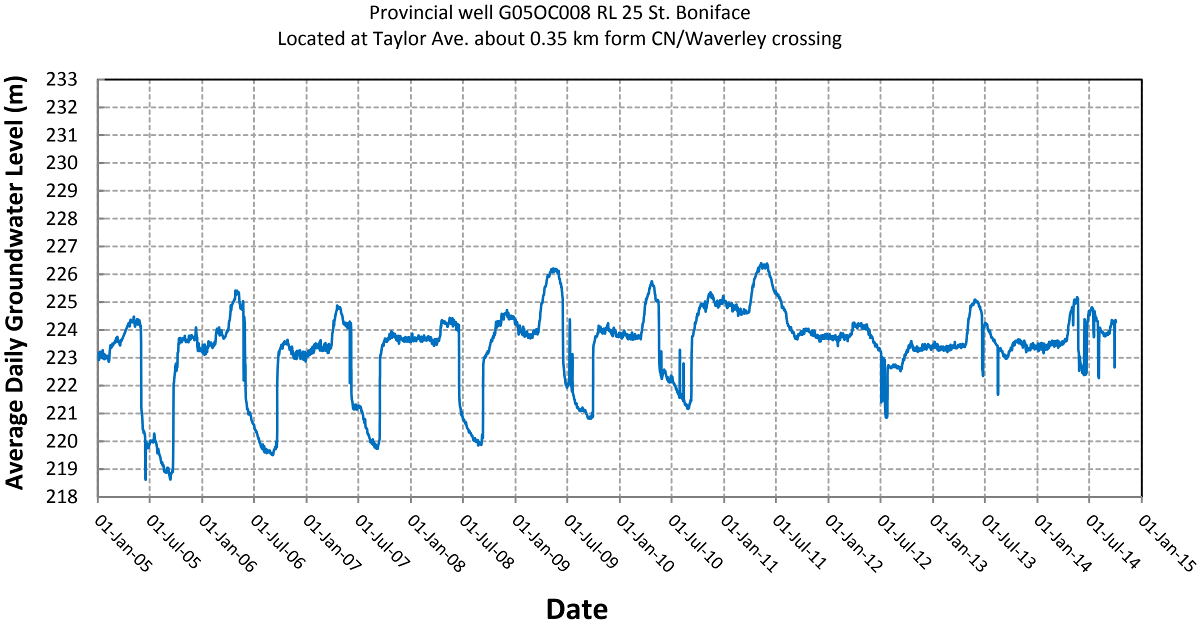


Figure 4 – Aquifer Groundwater Monitoring Results - Provincial Wells

3. Foundations

3.1 Underpass Structure Foundations

Shallow foundations are not considered suitable to support heavy loaded structures. Deep foundations bearing on competent very dense till or bedrock will be required to support these structures. Available deep foundation alternatives include:

- Driven Pre-cast Pre-stressed Concrete Piles;
- Driven Steel Piles;
- Cast-in-Place Belled Caissons; and
- Cast-in-Place Rock Socketed Caissons.

AREMA Manual 2012 is referenced as the design code for the Underpass Structure.

3.1.1 Driven Pre-Cast Pre-Stressed Concrete (PPC) Piles

Driven PPC piles can be designed to support the heavy loads of the proposed underpass structure, however our experience with CN indicate that PPC piles are not a preferred foundation system for railway structures support. If used, PPC piles should be driven to practical refusal into very dense glacial till or on the underlying bedrock. Provided that a well maintained hammer with a rated energy of at least 40 kJ per blow is utilized, the piles may be assigned the conventional capacities shown in Table 3. These traditional pile capacities are based on a series of studies and load tests and have been successfully used in the Winnipeg area for several decades.

Table 3 – Allowable Capacity for Driven PPC Piles

Pile Size (mm)	Maximum Allowable Capacity (kN)	Final Refusal (blows/25 mm)
300	450	5
350	625	8
400	800	12

Final refusal for driven PPC piles shall be taken as three consecutive sets of the refusal criteria as defined in the Table 3. PPC piles driven to practical refusal will develop the majority of their capacity from toe resistance, and therefore, no reduction in pile capacity is necessary for reasons related to group action. The design capacity of a pile group can be taken as the sum of the capacity for the number of piles in the group.

Pre-construction Wave Equation analysis and dynamic monitoring using Pile Driving Analyzer (PDA) during construction should be utilized to assess the suitability of the pile driving equipment, verify the set criteria, evaluate the mobilized capacity and protect against pile damage.

Further design and construction recommendations for driven PPC piles are summarized below:

1. The weight of the embedded portion of the pile may be neglected in the design;
2. The above allowable capacities pertain to soil resistance only, the pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation;
3. Pile spacing should not be less than 3 pile diameters, measured center to center;

4. Pre-boring can be used to enhance pile alignment and to reduce the effects of pile heave during driving of adjacent piles;
5. All piles should be driven continuously to the required refusal criteria, once driving is initiated;
6. All piles located within 5 pile diameters of another pile location should be monitored for heave during pile installation. Where pile heave is observed, the piles should be re-driven to the refusal criteria outlined above;
7. Any piles that are damaged, excessively out of alignment or refuse prematurely may need to be replaced, pending a review by the structural designer to assess pile load carrying capacity and any consequences of expected settlement on performance;
8. Where a steel follower is required to install piles below the ground surface, the refusal criteria should be increased by up to 50 percent, or as determined from PDA monitoring, to account for additional energy losses through the use of the follower;
9. The driving of all piles should be documented by experienced geotechnical personnel to confirm and record acceptable piling installation.

3.1.2 Driven Steel Piles

Driven steel H piles are commonly used to support heavily loaded structures. Steel piles can be designed on the basis of the structural capacity of the pile section provided the piles are driven to practical refusal into/onto bedrock. As per AREMA Manual 2011, the structural capacity of the pile can be determined from the steel sectional area and the maximum allowable stresses of 86 MPa (12,600 psi). All H-pile section shall conform to the current ASTM Designation A36. Practical refusal can be defined as 10 to 15 blows/25 mm pile penetration using a well maintained hammer with rated energy of not less than 50 kJ. The actual refusal criteria and load capacity for specific steel section and pile driving system should be established based on pre-construction Wave Equation analysis and PDA testing.

Steel piles driven to practical refusal will develop the majority of their capacity from toe resistance, and therefore, no reduction in pile capacity is necessary for reasons related to group action. The design capacity of a pile group can be taken as sum of the capacity of the number of piles in the group.

The following additional recommendations regarding steel piles are provided:

- The pile cross sections must be designed to withstand the design loads, handling stresses and driving forces during installation;
- The minimum depth of a steel H-section shall be 200 mm (8 inches). The minimum thickness of metal in the flange or web shall be 9.5 mm (3/8 inch). The flange width shall be not less than 85 percent of the depth of the section;
- Piles should be fitted with an appropriate toe or shoe to protect the pile tip during installation;
- Piles should be protected against corrosion using additional steel thickness;
- Pile spacing should be a minimum of 3 pile diameters measured centre to centre; and
- All piles driven within 5 pile diameters of one another should be monitored for heave and where observed, the piles should be re-driven to the specified refusal criteria.

3.1.3 Cast-in-Place Belled Caissons

Typically in the Winnipeg area, the till is considered loose to medium dense when the moisture content is greater than 10 percent. When the moisture contents are between 7 and 9 percent, the till is considered dense, and when the moisture content is between 4 and 6 percent, the till is usually very dense. Cast-in-place belled caissons has been designed on the basis of an allowable end bearing pressure of about 700 kPa provided they are founded in very dense till. The caissons might be mechanically or manually belled but the caisson bottom must be hand cleaned so that no loosened or disturbed soils are left in the base of the bore. Safety concerns related to man entry into the boring (e.g., high level of gas) may preclude undertaking the cleaning and inspection and should be considered if this alternative is contemplated.

Caisson's installation difficulties with respect to groundwater seepage, bell stability of roofs of the bells or caving and bore advance through boulder/cobble zone should be carefully evaluated; these types of construction challenges are common in the Winnipeg area and should be anticipated in this project. The foundation contractor must expect to encounter boulders within the glacial till and at elevations above the required founding level. Chopping of boulders may be necessary to advance the borings into till. The minimum shaft diameter should be 760 mm to permit the entry of personnel for base cleaning and inspection. Temporary steel sleeves must be used to permit the safe entry of personnel. The maximum bell/shaft diameter ratio should be in the order of 2.7. All caisson bases should be inspected by experienced geotechnical personnel to verify that the base conditions are consistent with the design parameters.

On the basis of in-situ testing, pile load testing, and analytical studies that have been undertaken at other locations in Winnipeg for caissons in comparable glacial silt till, the caisson settlements can be expected to be less than 20 mm for bell diameters that are commonly employed.

This foundation alternative is expected to present challenges and it may not be feasible for this project. If this design alternative is contemplated, a test caisson(s) is highly recommended to verify design assumptions, examine the feasibility of construction and assist in the selection of adequate equipment and proper construction practices.

3.1.4 Cast-in-Place Rock Socketed Caissons

Drilled caissons socketed into sound bedrock can be designed to support the proposed heavy structures. Local practice is to design the drilled shafts based on values of maximum allowable end bearing and/or shaft adhesion of 3.0 and 1.0 MPa, respectively, provided that downhole inspection and assessment of the rock competency are undertaken. The assessment of the rock competency consists of small diameter proof drilling to 2 m below the socket base to detect the presence of voids or clay/silt layers of any significance and determine if deeper socket boring is required. In the event that the socket cannot be visually inspected, inspection of the recovered rock core and/or downhole video monitoring can confirm the competency of the bedrock. In this situation, caissons founded in sound bedrock should be designed on the basis of a reduced allowable shaft adhesion of 0.60 MPa with no contribution from end bearing. Safety concerns related to man entry into the boring (e.g., high level of gas) may preclude undertaking the visual inspection.

To our knowledge, settlements of rock socketed caissons have never been measured in the Winnipeg area. However, it is anticipated that the settlements would be less than 20 mm.

Based on the finding of the three test holes advanced into the bedrock (TH14-02 to 14-04), the top 5 m of the bedrock is dominated by very poor to poor quality rock. A layer of clay infill 0.3 to 0.8 m thick was encountered within the bedrock between elevation 211.7 and 212.6 m. The thickness of the fractured and heavily jointed bedrock is variable and could be in excess of 5 m and the clay infill may vary in thickness and could be encountered at different elevations. Socket length, at least at the location of these test holes, should be expected to be developed below elevation 211.0 m and measures to maintain socket wall stability and groundwater control should be anticipated.

Inspection of the recovered rock cores by qualified and experienced geotechnical personnel and downhole video inspection will be required to aid in assessing the competency of the bedrock and determining if longer socket lengths are required. The depth to sound bedrock should be expected to vary across the site and it should be recognized that the presence of the heavily fractured rock and infill material above the socket length may require that a permanent steel casing be left in the ground so that the integrity of the shaft is maintained. In this regard, the basis for measurement and payment for the rock socket installation should be established in the contract preparation stage to recognize that the bedrock conditions at some rock socket locations may require unanticipated extra effort and materials for their completion.

The socket length should be a minimum of three socket diameter within competent bedrock. The minimum shaft diameter of the rock socket should not be less than 760 mm and the maximum diameter should be selected to suit the locally available coring equipment. The rock sockets should not be spaced closer than 3 socket diameters, centre to centre. Tremie placement of concrete is likely to be required.

Should this type of foundation is contemplated, a test caisson(s) is highly recommended to verify design assumption, examine the feasibility of construction and assist in the selection of adequate equipment and proper construction practices.

3.1.5 Foundation Alternatives Assessment

Four deep foundation alternatives are identified to support the proposed underpass structure including:

- Driven precast prestressed concrete piles;
- Driven steel H piles;
- Belled caissons; and
- Rock socketed caissons.

Numerous structures in the Winnipeg area are supported on foundation systems consisting of one or a combination of the above types. The factors governing the design and performance of these pile types are well understood by the engineering community and the construction industry. Local contractors are familiar with related construction practices and the necessary equipment for installation is available.

Driven steel H piles can be driven to practical refusal into/onto bedrock surface and designed on the basis of steel section structural capacity. Pile axial capacity up to 1200 kN can be mobilized for common pile sections. These piles offer easy splicing and can be made in variable lengths. Larger sections can be selected if greater design loads are desired. Adequate driving equipment, good installation experience and reliable testing methods are locally available. Pile caps are anticipated to be of reasonable size. Also, steel H piles are the preferred pile type by CN Rail.

Driven PPC piles are common in the Winnipeg area but are limited in manufactured length and the design capacity ranges between 400 to 800 kN. Pile cap size is expected to be larger than the size required using steel piles to support similar load. Precast piles do not lend themselves to certain structural applications such as integral abutment design. Driven PPC piles are not preferred by CN Rail.

Belled caissons bearing on competent till can be designed to mobilize loads comparable to steel H piles. However pile cap size would be significantly larger to support a similar load. Based on the findings of the deep test holes drilled at the vicinity of the proposed underpass, the encountered till is not anticipated to mobilize bearing capacity that would make this pile type cost effective. The installation requires base cleaning and downhole inspection. Construction difficulties related to groundwater control, roof stability and boulder removal are not uncommon and may impact project cost and schedule or require design review. We are not aware if such pile foundation had been used recently to support CN Rail structures.

Rock socketed caissons bearing in competent bedrock can be designed to support significant design load. The rock condition encountered at the proposed underpass structure indicated the top 5 m of the bedrock is generally dominated by poor quality and extensively jointed/broken rock mass. Accordingly, rock socketed caissons need to develop their capacity based on adhesion mobilized below this weak zone. Rock socketed caissons lend themselves for top down construction being currently contemplated for the proposed underpass structure and it has been successfully used in Kenaston underpass.

Based on the available information and above discussion, it seems that driven steel H piles are the preferred foundation system to support the abutments of the proposed underpass structures while rock socketed caissons seems suitable to support the intermediate piers. Further investigation and assessment should be undertaken to confirm subsurface conditions and review the suitability of the selected foundation type(s).

3.2 Retaining Walls Foundations

Loads from retaining walls could range from light to heavy depends on the type and dimensions of the walls. Foundation requirements could be governed by lateral resistance and/or construction aspects rather than axial resistance. Heavy loads from retaining wall can be supported using deep foundation elements including driven PPC and steel piles. The ease of installing battered driven piles to resist lateral forces makes these piles preferable for wall foundation. Lightly loaded walls could be supported on shallow foundation or cast-in place friction piles.

AASHTO LRFD Bridge Design Specification 2014 and CAN/CSA Canadian Highway Bridge Design Code 2006 are referenced as the design code for the retaining walls.

3.2.1 Shallow Foundations

Shallow footings can be used to support and transfer light loads to the underlying soil at a pressure consistent with the loading requirements and the bearing capacity of the soil. The footings should bear on native clay below the frost penetration depth. The nominal and factored bearing resistance at ultimate limit state (ULS) for a range of footing dimensions bearing at 2.4 m below ground has been evaluated. A nominal bearing resistance of 225 kPa and a resistance factor of 0.5 should be used to derive the factored bearing resistance at ULS. The bearing capacity of a footing is highly influenced by the load inclination, an inclined load H/V = 0.1 would result in reduction of the bearing resistance to 90 percent of the value above (i.e., 225 x 0.90). As part of the deign development, structure specific assessment and further analysis should be completed to verify and confirm these preliminary recommendations. Different configurations of spread footings may result in a potential for load superposition and overstressing of the bearing stratum. Under these circumstances modification to the footings configuration or a review of the bearing capacity may be required. Total and differential settlement magnitude and rate under spread footings can be estimated using a one dimensional consolidation theory, Footings load, configuration and subsoil compressibility characteristics are necessary input in settlement analysis and will need to be conducted as part of the detailed design phase.

Shallow footings should be located below the frost penetration depth which is estimated at 2.4 m below ground surface. This depth can be reduced if thermal insulation is used to protect against frost penetration provided the footing is bearing on competent soil. The potential for movement caused by volumetric changes of the high plasticity clay due to changes in moisture content should be reviewed for its impact on future performance.

Nominal unit resistance to sliding at ULS conditions can be calculated as the sum of normal sliding resistance and passive sliding resistance. A resistance factor of 0.85 should be applied to the nominal normal sliding resistance which can be taken as the smaller of:

- Clay undrained shear strength = 30 kPa; or
- Provided the footing is supported on at least 150 mm compacted granular, one half the normal stress at the footing/clay interface.

If passive sliding resistance accounted for in the design it should be carefully evaluated for the possibility of future removal of the soil from the front of the wall and the associated displacement to mobilize the maximum passive soil resistance.

Soil within the depth of frost penetration can freeze to the foundation developing an uplift force. An adfreeze bond of 65 kPa can be used to estimate the uplift forces. These forces can be resisted by the sustained vertical loads on the footing. A frost non-susceptible material or bond breaker/thermal insulation between the footing and the adjacent soil can be used to protect against adfreeze bond development.

Footings should not be placed on uncontrolled fill, organic or other deleterious soils. The bearing stratum should be cleaned to remove all disturbed or otherwise affected soil and protected from frost, desiccation and the ingress of free water.

3.2.2 Cast-in-Place Friction Piles

Cast-in-place concrete friction piles can be used to support lightly loaded structures. The nominal and factored unit friction resistance are summarized in Table 4. The frictional resistance for the top 2 m along the pile shaft should be ignored from the design calculations to accommodate for moisture change and freeze/thaw effects. The piles should not extend into the soft clay above the till layer to protect against seepage and instability of the bore hole. In this regard, friction piles should not extend deeper than elevation 223.0, this elevation can be reviewed once further investigation is completed as part of the detailed design. The bearing resistance at service limit state (SLS) presented in Table 4 is associated with a settlement of 5-10 mm excluding elastic shortening of the pile.

Table 4 – Limit State Bearing Resistance for Cast-in-Place Friction piles

Nominal Unit skin Friction (kPa)	ULS Condition				SLS Condition Bearing Resistance (kPa)
	AASHTO LRFD		CAN/CSA-S6-06		
	Resistance Factor	Factored Bearing Resistance (kPa)	Resistance Factor	Factored Bearing Resistance (kPa)	
20	0.45	9	0.4	8	Equal to ULS

Additional design and construction recommendations are provided below:

1. Pile diameter should not be less than 0.45 m.
2. Piles should be adequately reinforced to resist possible tension from clay swelling or frost heave.
3. Pile spacing should be a minimum of 3 pile diameters measured centre to centre.
4. Temporary casing to facilitate cleaning, inspection and protect against seepage and sloughing during construction should be available on site.
5. All piles must be taken to completion once they have been initiated.

3.2.3 Driven Pre-cast Pre-Stressed Concrete Piles

Static analysis was carried out using DRIVEN 1.2 software to estimate the axial capacity for driven PPC piles. In estimating the pile capacities, the design SPT (N) value profile presented on Figure 1 and results from laboratory tests were used to estimate the angle of internal friction for glacial till. In determining pile capacity, no contribution was considered from the clay layer. Detailed results of the analysis and the nominal resistance versus pile penetration are attached in Appendix D. Regardless of the geotechnical capacity, the load applied to the pile should not exceed the structural capacity of the pile section. Estimated nominal bearing resistance at ULS for piles driven at least 2 m into dense/very dense till are summarized and presented in Table 5. The factored bearing resistance at ULS will depend on the level of construction control adopted at site during pile installation to verify that piles are installed to mobilize the desired nominal bearing resistance. In this regard, a resistance factor was determined for two conditions:

- Using PDA testing and dynamic monitoring on at least 2 percent of the piles number to determine the driving criteria. The associated resistance factor = 0.65.
- Using FHWA modified Gates dynamic pile formula to determine the driving criteria. The associated resistance factor = 0.40.

The bearing resistance at SLS, associated with a settlement of 5 -10 mm excluding elastic shortening of the pile, is also provided in Table 5. PPC piles driven into dense/very dense will develop the majority of their capacity from toe resistance, and therefore no reduction in pile capacity is necessary for reasons related to group action. The design capacity of a pile group can be taken as sum of the capacity of the number of piles in the group.

A pile driving analyzer (PDA) test program is recommended to confirm pile capacity and verify safe installation of the piles. The PDA testing services can be provided by AECOM upon request.

Table 5 – Limit State Bearing Resistance for Driven PPC Piles

Pile Size	Nominal Bearing Resistance, (kN)			ULS Condition				SLS Condition Bearing Resist. (kN)	Driving Criteria Basis and Field Control
				AASHTO LRFD 2014		CAN/CSA-S6-06			
	Total	Shaft	Toe	Resist. Factor	Factored Bearing Resist. (kN)	Resist. Factor	Factored Bearing Resist. (kN)		
HEX 300 mm	1365	239	1127	0.65	887	0.5	683	350	PDA Test
				0.40	546	0.4	546		Modified Gates Formula
HEX 350 mm	1885	346	1539	0.65	1225	0.5	943	450	PDA Test
				0.40	745	0.4	745		Modified Gates Formula
HEX 400 mm	2456	452	2004	0.65	1596	0.5	1228	550	PDA Test
				0.40	982	0.4	982		Modified Gates Formula

Further design and construction recommendations for driven PPC piles are summarized below:

- The weight of the embedded portion of the pile may be neglected in the design;
- The above allowable capacities pertain to soil resistance only, the pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation;
- Pile spacing should not be less than 3 pile diameters, measured center to center;
- Pre-boring can be used to enhance pile alignment and to reduce the effects of pile heave during driving of adjacent piles;
- All piles should be driven continuously to the required driving criteria, once driving is initiated;
- All piles located within 5 pile diameters of another pile location should be monitored for heave during pile installation. Where pile heave is observed, the piles should be re-driven to the refusal criteria outlined above;
- Any piles that are damaged, excessively out of alignment or refuse prematurely may need to be replaced, pending a review by the structural designer to assess pile load carrying capacity and any consequences of expected settlement on performance;
- Where a steel follower is required to install piles below the ground surface, the driving criteria should be adjusted by up to 50 percent, or as determined from PDA monitoring, to account for additional energy losses through the use of the follower;
- The driving of all piles should be documented by experienced geotechnical personnel to confirm and record acceptable piling installation

3.2.4 Driven Steel H Piles

Static analysis was carried out using DRIVEN 1.2 software to estimate the axial capacity for driven steel H piles. Similar to Section 3.2.3, the design SPT (N) value profile presented on Figure 1 and results from laboratory tests were used to estimate the angle of internal friction for glacial till. In determining pile capacity, no contribution was considered from the clay layer. Detailed results of the analysis and the nominal resistance versus pile penetration are attached in Appendix D. Regardless of the geotechnical capacity, the load applied to the pile should not exceed the structural capacity of the pile section. Estimated nominal bearing resistance at ULS for piles driven at least 3 m into dense/very dense till are summarized and presented in Table 6. The factored bearing resistance at ULS will depend on the level of construction control adopted at site during pile installation to verify that piles are installed to mobilize the desired nominal bearing resistance. In this regard, a resistance factor was determined for two conditions:

- Using PDA testing and dynamic monitoring on at least 2 percent of the piles number to determine the driving criteria. The associated resistance factor = 0.65.
- Using FHWA modified Gates dynamic pile formula to determine the driving criteria. The associated resistance factor = 0.40.

The bearing resistance at service limit state (SLS), associated with a settlement of 5 -10 mm excluding elastic shortening of the pile, is also provided in Table 6. Steel H piles driven into dense/very dense will develop the majority of their capacity from toe resistance, and therefore no reduction in pile capacity is necessary for reasons related to group action. The design capacity of a pile group can be taken as the sum of pile capacities in the group.

A pile driving analyzer (PDA) test program is recommended to confirm pile capacity and verify safe installation of the piles. The PDA testing services can be provided by AECOM upon request.

Table 6 – Limit State Bearing Resistance for Driven Steel H Piles

Pile Size	Nominal Bearing Resistance, (kN)			ULS Condition				SLS Condition Bearing Resist. (kN)	Driving Criteria Basis
	Total	Shaft	Toe	AASHTO LRFD 2014		CAN/CSA-S6-06			
				Resist. Factor	Factored Bearing Resist. (kN)	Resist. Factor	Factored Bearing Resist. (kN)		
H 310 x110	2238	388	1850	0.65	1455	0.5	1119	560	PDA Test
				0.40	895	0.4	895		Modified Gates Formula

The following additional recommendations regarding steel piles are provided:

- The pile cross sections must be designed to withstand the design loads, handling stresses and driving forces during installation;
- Piles should be fitted with an appropriate toe or shoe to protect the pile tip during installation;
- Piles should be protected against corrosion using additional steel thickness;
- Pile spacing should be a minimum of 3 pile diameters measured centre to centre; and
- All piles driven within 5 pile diameters of one another should be monitored for heave and where observed, the piles should be re-driven to the specified refusal criteria.

3.3 Lift Station Foundations

The lift station structure will be configured into two main parts: deep and shallow. It is important to support these two parts on common competent soil stratum and protect against differential movement. The deep part can be supported on raft foundation bearing on the dense/very dense till. The loads from the shallow part should be transferred through piles bearing into the till layer at elevation of the raft or deeper. Driven steel H-piles or PPC piles can be designed to support the shallow part of the structure. The sequence of the pile installation, excavation and raft construction should be carefully assessed to protect against any adverse impact.

Limit state design in accordance to the principles of AASHTO 2014 and CAN/CSA 2006 are referenced as the design code for the lift station.

3.3.1 Raft Foundation

Raft foundation can be designed to provide suitable support to the deep part of the lift station. Foundations placed at depths where the structural load equals the weight of the excavated soil usually have adequate bearing capacity and only recompression settlement.

The elevation of the underside of the proposed pump station, as determined by the civil designer, is about 219.5 m. Preliminary recommendations for the bearing resistance at both ULS and SLS are provided on Figure 5. SLS bearing resistance has been calculated corresponding to settlement of 25 mm. Once the raft dimensions and exerted loading are finalized, the foundation response should be evaluated as part of the detailed design stage. If the pressure from the structure is in excess of the in-situ, the anticipated settlement should be estimated. If the structure loading is less than the in-situ overburden pressure then an upward displacement/ rebound at the foundation level is expected to be a result of the stress relief due to excavation unloading. The rebound movement is expected to be restrained by the weight of the structure and the side friction along the walls/backfill interface. Theoretically, the rebound will continue to a point where the stress at the foundation level is equal to the in-situ overburden pressure before the excavation. In this regard, the base of the structure should also be designed to resist an upward pressure equal to the in-situ overburden pressure.

A preliminary estimate of the modulus of subgrade reaction for the undisturbed till at elevation of 219.5 is 10 MN/m^3.

A foundation preparation should include removal of all loose/disturbed soil and placement of at least 100 mm lean concrete (mud slab) after inspection by qualified geotechnical engineer. Raft should not bear on uncontrolled /undocumented fill. Dewatering system will be required to control groundwater and allow construction in the dry. Care should be taken during excavation so that the final bearing surface is not disturbed or subjected to freezing, water inundation or excessive drying. Once the bearing surface has been suitably prepared, it should be evaluated by qualified geotechnical personnel to verify the suitability of the bearing soils, confirm that the soils are uniform, not affected by frost or disturbance and to confirm that the soils encountered are consistent with the conditions noted in this report. As soon as possible, a 100 mm thick lean concrete (mud slab) should be placed and followed by the reinforcing steel and concrete.

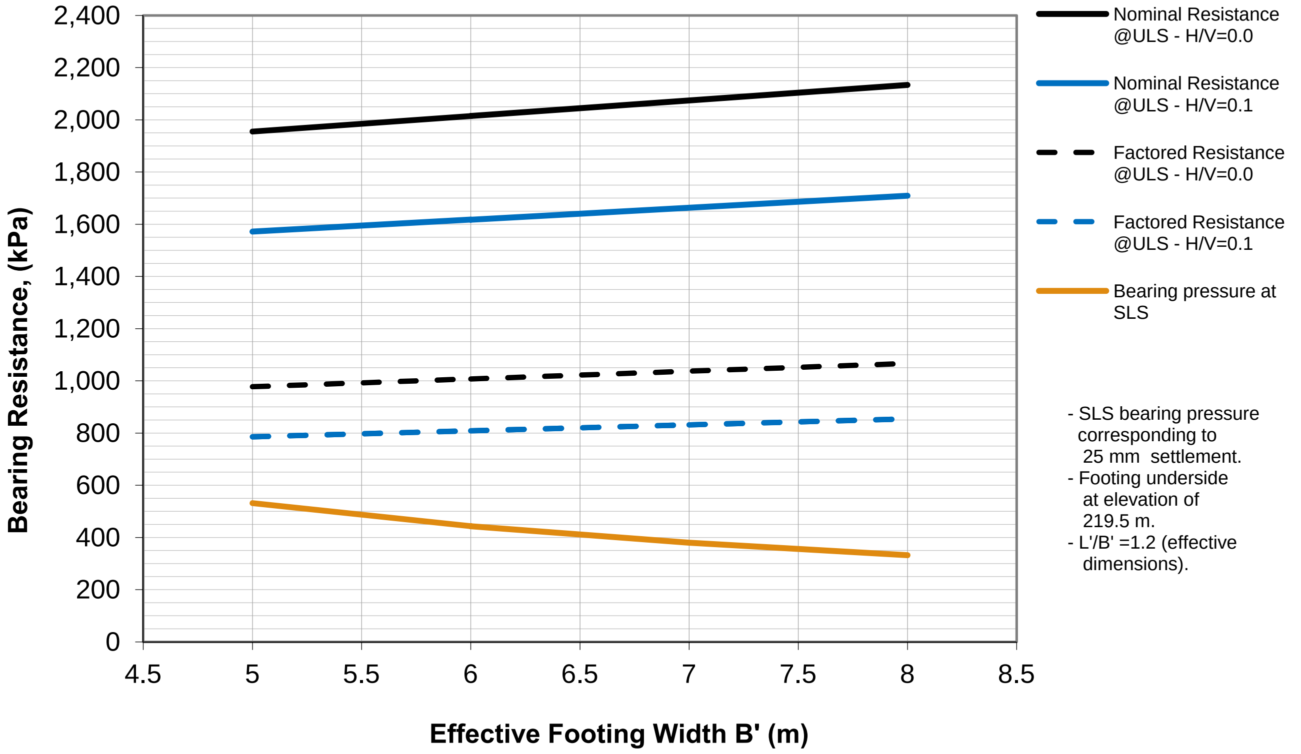


Figure 5 – Limit State Bearing Resistance for Raft Foundations (Lift Station)

3.3.2 Driven PPC Piles

The upper portion of the pumping station can be supported on driven PPC piles. The discussion and recommendations provided in Section 3.2.3 are applicable.

3.3.3 Driven Steel H Piles

The upper portion of the pumping station can be supported on driven steel H piles. The discussion and recommendations provided in Section 3.2.4 are applicable.

4. Temporary Excavations and Shoring

Temporary excavations range from 3 to about 13 m deep will be required to facilitate the construction of the proposed work (i.e., lift station, abutments and retaining wall foundations). These excavations will be in close proximity to CN tracks and existing utilities and infrastructures along Waverley Street.

Temporary works are the responsibilities of the Contractor and all necessary measures should be undertaken to protect against adverse impact or undermining the foundation or stability of existing infrastructure. All excavations must comply with Manitoba’s Workplace Safety and Health Act and Regulations.

This section discusses geotechnical concerns including shoring and lateral forces, anticipated ground movement around excavations, and base stability.

Additional stability analysis and excavation plan development, related to the stability of the temporary railway detour and temporary road detour during the construction period should to be investigated as part of the detailed design stage.

4.1 Unsupported Excavations

Open cut excavations could be used where the available space allows, however the maximum open cut height should not exceed 6 m. The location and height of the cut slopes may be further dictated by other considerations such as access, proximity to existing infrastructure, anticipated construction approach and staging. A design objective FS of 1.30 against slope instability is considered acceptable design practice for short term temporary work (i.e., not exceeding two months period). The Contractor shall provide stability assessment prepared by professional engineer demonstrating the proposed excavations satisfy the design objective. Railway and construction surcharges should be accounted for in the stability model where applicable. The stability model shall adopt soil strength parameters and groundwater conditions representative of the Winnipeg area and acceptable to the project geotechnical engineer.

4.2 Supported Excavations

In addition to open cut excavations, supported and partially supported excavations will be necessary for the proposed construction. Cantilever and braced shoring can be used to support the excavations. A partially supported excavation utilizes a combination of cut slopes and shoring. The design is expected to include a soldier pile system and sheet piling. Recommendations for design earth pressures are provided in Figure 6. The shoring should be designed to resist lateral earth pressure and lateral forces from live load surcharges including railway loading and anticipated construction activities. Lateral pressure from railway loading should be determined as per the latest CN Guidelines and AREMA Manual using Cooper E90 loading.

The active pressure should be extended to the base of the wall system (i.e., the bottom of the piles). The wall must be embedded deeply enough to provide adequate resistance for the portion of the wall below the excavation. Passive resistance below the excavation level should include a factor of safety of 1.5. Passive resistance from the soil located in the upper 0.5 m below the excavation level should be ignored.

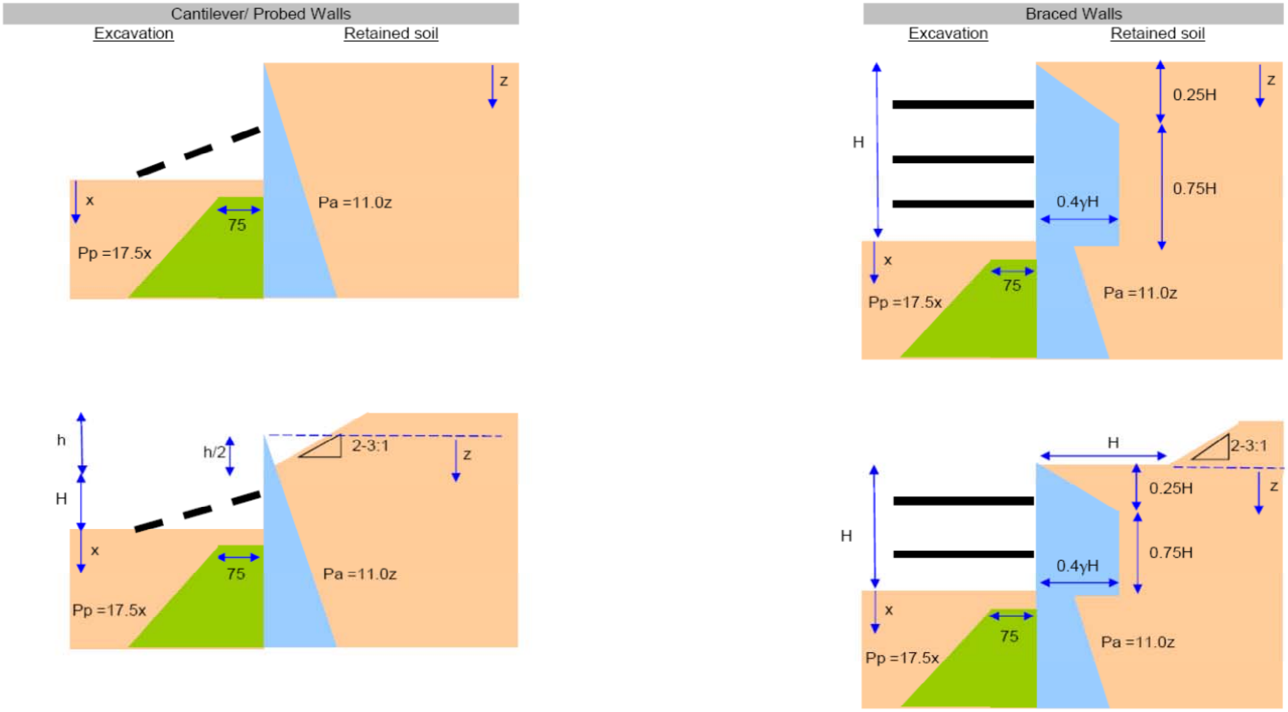


Figure 6 –Lateral Earth Pressure for Temporary Shoring

4.3 Ground Movement

Excavation support systems are usually designed to keep movements around the perimeter of the excavation within acceptable limits. Avoidance of ground movements entirely is not possible. The amount of movement that will occur cannot be accurately predicted mainly because the movements are more a function of excavation procedures and workmanship than they are of theoretical considerations. Settlements of the ground surface adjacent to braced excavation can be estimated using the chart developed by Clough and O'Rourke (1990) as shown in Figure 7. It is recommended that the boundary between Zone III and IV be used to estimate vertical ground movements at the site. It should be recognized that the predicted ground movements are associated with standard soldier piles and lagging or sheet piles with cross bracing or tie back anchors, assuming they are installed with a normal quality of workmanship. Good contact between the lagging and retained soil should be maintained throughout the construction period. Free draining sand should be used to fill the voids behind the lagging or sheet piles.

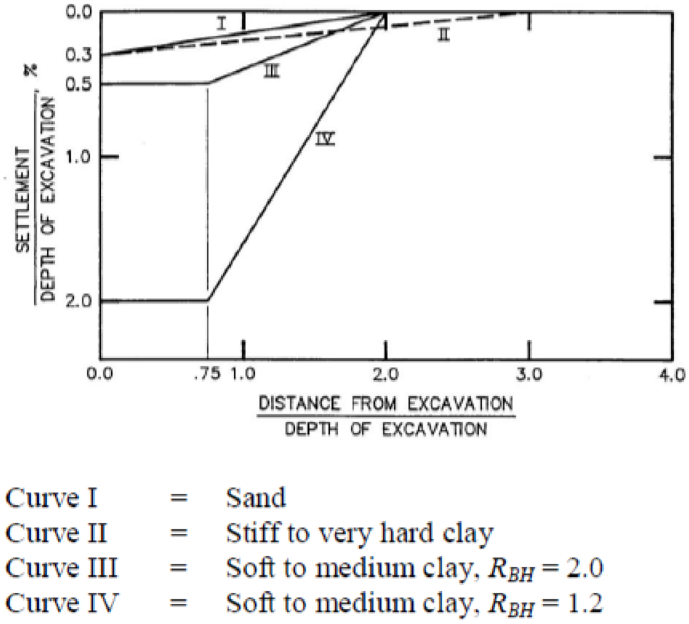


Figure 7 – Ground Settlement Estimate adjacent to Excavations

4.4 Base Heave

When impervious layer is underlain by pervious layer subject to artesian condition, the potential for base heave should be evaluated. The upward pressure exerted by the artesian groundwater on the underside of the impervious layer should be controlled so not to exceed the downward overburden pressure at the interface between the two layers and protect against development of critical condition. The factor of safety (FS) against base heave is expressed as the ratio of the total stress at the base of the impervious layer to the groundwater pressure acting on the base of the impervious layer, with no account for any shearing resistance. A minimum FS of 1.3 is recommended against base heave for short term condition.

Temporary excavations for the abutments, the retaining walls and the lift station are considered in base heave assessment. Where the excavation is expected to advance into the till (i.e., lift station) the till was modelled as impervious soil and the artesian pressure to act on bedrock/till interface. The results are presented on Figure 8. The range of the Aquifer GWL observed during the monitoring period from June to December 2014 and the historical peak GWL from the nearby Provincial wells are shown on Figure 8. The results indicate the following:

- Temporary excavations up to elevation 224.5 m would attain acceptable short term FS under the GWL range observed in the aquifer during the monitoring period (i.e., GWL < 225.8 m);
- Groundwater control and aquifer depressurization will be required for temporary excavations deeper than elevation 224.5 m or for shallower excavations if GWL higher than observed is encountered in the aquifer during the excavation period;

- Temporary excavations for abutments and retaining walls are anticipated to be shallower than elevation 225 m and therefore aquifer depressurization will likely not be required;
- Temporary excavations for the lift station is anticipated to be advanced into till up to elevation 219 m. Groundwater control, aquifer depressurization and construction dewatering to facilitate construction will be required to lower the GWL to at least 0.5 m below the excavation bottom. Other concerns such as piping may call for additional GWL control (i.e., more than 0.5 m below excavation level);
- GWL monitoring is necessary during construction; and
- Base heave potential and protection measures increases with increasing excavation depth.

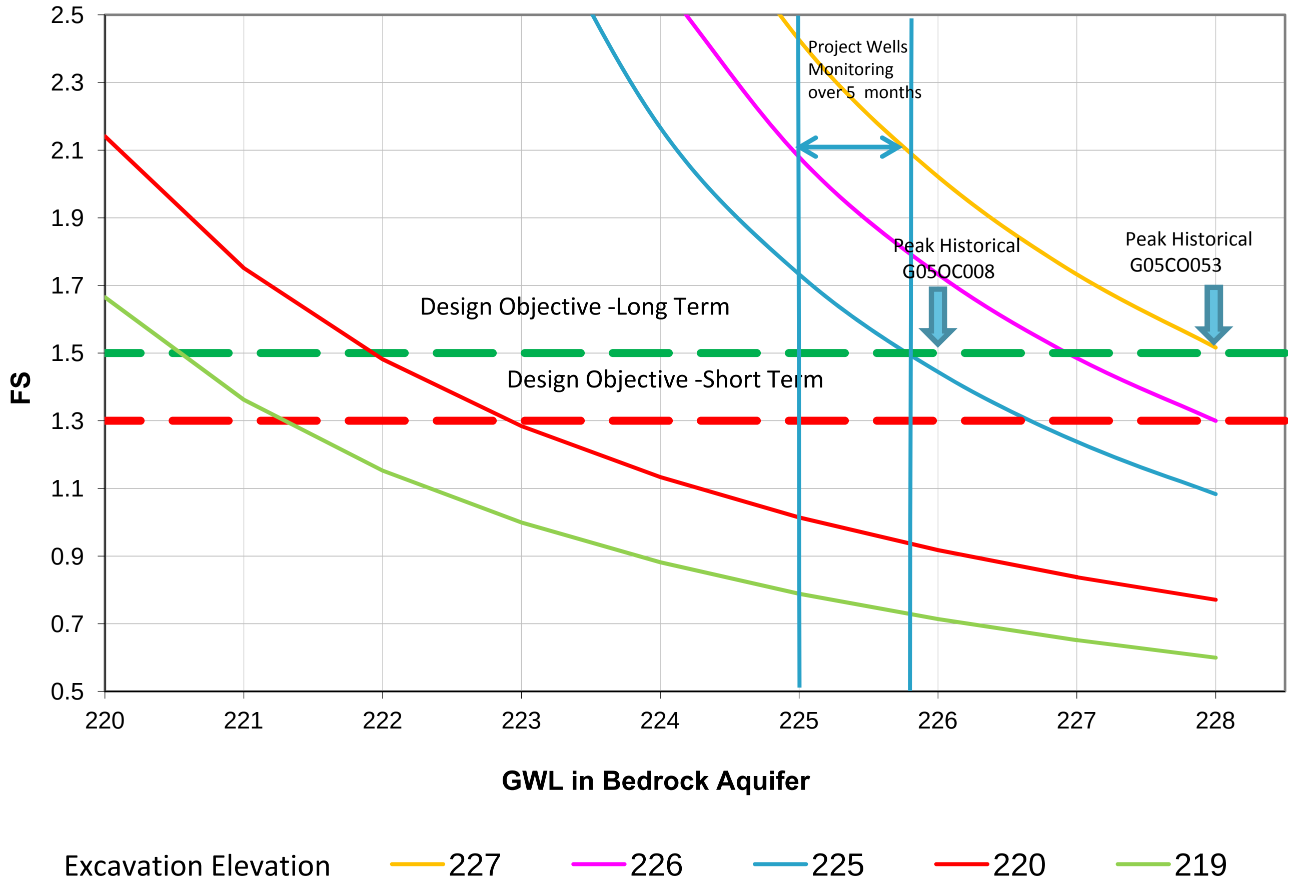


Figure 8 – Calculated Factor of Safety against Base Heave

4.5 Piping

The silt till or part of could behave similar to a cohesionless soil and the potential for piping under artesian condition may arise. Piping is the phenomenon where upward seepage through soil introduces the condition at which the exit hydraulic gradient approaches the critical hydraulic gradient. The critical hydraulic gradient is the gradient that would reduce effective stress to zero and its average value for most soil is equal to 1. The FS against piping is the ratio between critical and exit hydraulic gradient.

Assuming water will not be allowed to accumulate in the excavation and the hydraulic gradient across the till is equal to the exit gradient, a preliminary assessment has been completed to estimate the FS against piping. The results are presented on Figure 9. Based on this preliminary assessment aquifer depressurization to 1 m below excavation elevation is recommended to attain design objective FS of 1.5.

Hydrogeological assessment, instrumentation installation and further review and evaluation as part of the detailed design stage will be required to determine and confirm measures necessary to protect against piping.

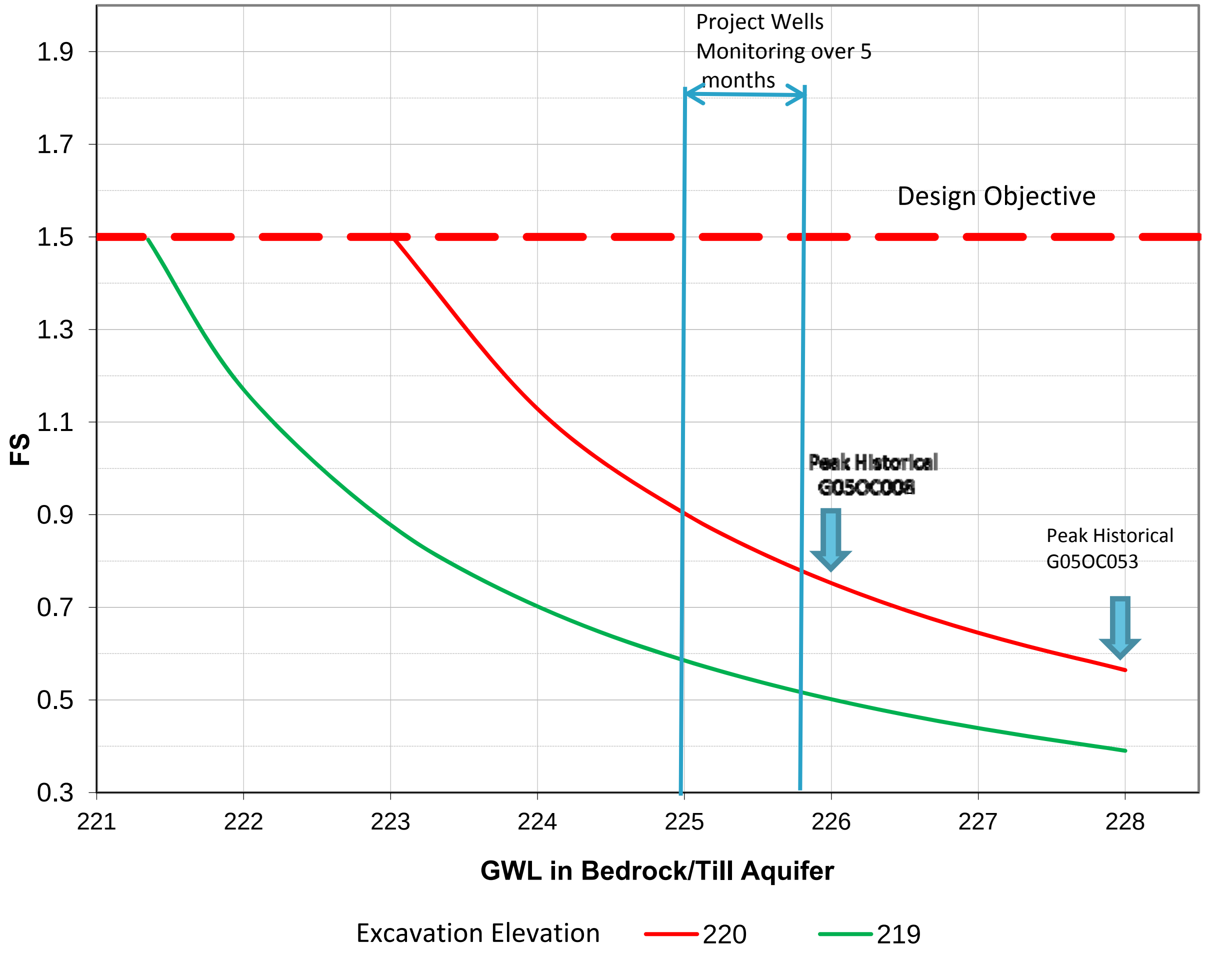


Figure 9 – Calculated Factor of Safety against Piping

4.6 Base Instability

Base instability is a concern for excavations in soft to firm clays. It is analogous to a bearing capacity failure, the difference being that stresses in the ground are relieved instead of increased. Two types of analysis are available for calculating the FS against base instability: the Terzaghi method and the Bjerrum & Eide method. The Terzaghi method is applicable for shallow and wide excavations, whereas the Bejrrum method is suitable for deep and narrow excavations.

The Bjerrum & Eide method has been used to complete an analysis for the FS against base instability for the lift station, retaining wall and abutment excavations. A live load surcharge due to railway loading and construction equipment was considered in this assessment. The analysis considered a range of excavation lengths, widths and depths. The analysis results indicate that the calculated FS against base instability for the scenarios considered in the analysis was greater than 1.50 which satisfy the design objective.

5. Permanent Excavations

The depressed road section of the proposed Waverley underpass will be a permanent excavation with cut slopes as deep as 7 m below the existing grade. The following sections discuss the geotechnical concerns related to permanent excavations including base heave, swell and rebound and slope stability.

5.1 Base Heave

As previously discussed in Section 4.4, base heave potential for the permanent excavations was also evaluated. A design objective FS of 1.5 is commonly adopted for long term condition. The deepest road section will be at about elevation 225.5 m. Figure 8 indicates that excavations at elevation 225.5 or shallower will attain FS satisfying or exceeding the design objective for the observed range of aquifer GWL. In the event the aquifer GWL exceeded this range a lower FS will be experienced however it will be higher than 1.3 and it is considered to be acceptable over short duration.

5.2 Heave

Heave in excavations is comprised of elastic rebound and swelling due to removal of overburden or change in moisture. Elastic rebound will take place immediately while swelling is time dependent, more swelling will be realized the longer the period the excavation is open. An estimate of the anticipated rebound and swell can be provided once additional investigation completed and the profile design is finalized as part of the detailed design phase. Recompression of the elastic rebound will take place immediately after construction while recompression of swell, if any, is time dependent. Once recompression has occurred settlement will start to take place due to imposed loading, if it is in excess of in-situ effective stress.

The swell can be reduced if staged and sequenced construction approaches are utilized. An optimum time lag between stages and phases of construction can be used to protect against differential heave/recompression. Further assessment should be provided as part of the detailed design stage.

5.3 Slope Stability

An adequate FS against slope instabilities must be achieved for the proposed cut slopes along Waverley Street. In this regard, a design objective FS of 1.5 for long term and 1.3 for short term end of construction has been selected. These objectives are consistent with acceptable design practice and commonly selected in the Winnipeg area.

A preliminary stability analysis was completed to investigate feasibility of cut slope design and determine if additional design measures are required to attain design objective FS. Long term condition was analyzed for selected configurations of cut slopes and head slopes. Short term condition was not considered in this preliminary assessment, it should be carefully evaluated in the detailed design phase as part of the design development.

The soil strength parameters used in the analysis are summarized in Table 7. These parameters were selected based on available geotechnical information and related experience from similar projects. The parameters are within the range of locally accepted values for Winnipeg clay and till. The assumed groundwater and piezometric conditions modelled in the analyses were based on short term GWL monitoring of site specific installation and our knowledge of local conditions.

Table 7 – Strength Parameters for Stability Assessment

Material	Unit Weight (γ),	Effective Stress Analysis		Groundwater Level
		Cohesion (C`)	Friction Angle (Φ`)	
	kN/m^3	kPa	degree	m
Fill	17	5	16	1 – 3 below grade
Clay				
Till	20	10	28	226
Bedrock	Impenetrable			

The initial results of the preliminary stability assessment are illustrated on Stability Figures 01 and 02 in Appendix E. The results indicate the following:

- Cut slopes not exceeding 6 m deep can be designed at configuration consists of two slopes and intermediate bench. The upper and lower slopes should be at 4H:1V inclination or flatter. The intermediate bench should be 4.5 m wide at level between 40 - 50 percent of the total slope height measured from the toe of the cut slope.
- Cut slopes between 6 and 7 m deep can be designed at configuration consists of two slopes and intermediate bench. The upper slope should not be steeper than 5H:1V inclination and the lower slope should be at 4H:1V inclination or flatter. The intermediate bench should be 4.5 m wide at level between 40 - 50 percent of the total slope height measured from the toe of the cut slope.
- Subdrains system about 1m deep below the intermediate bench and 0.5m below road subgrade were modeled to control groundwater and should be incorporated in the design.

The 4.5 m wide intermediate bench will be used as Active Transportation Path (ATP). Stability improvement can be attained by optimizing the level/ position of this bench. Crest offloading by permanent subcut or replacement of in-situ soil with light weight fill could also be considered to attain stability improvement. A design optimization should be completed as part of the detailed design stage.

6. Buoyancy and Uplift

Structures located below groundwater level should either be designed to resist buoyant forces from hydrostatic pressures or have an integrated pressure relief system. It is prudent to investigate buoyancy effect for the completed structures and for conditions during construction assuming credible scenarios for groundwater condition. In this regard, an input from a hydrogeologist may be required. Structures resisting the buoyant uplift forces will require restraining devices or uplift resistance measures. Forces that can be considered in providing the uplift resistance include: the dead weight of the structure and the weight of the soil above. The footing can also be extended symmetrically beyond the walls of the structures at least 1.0 m so that the weight of the soil above the footing can be accounted for in buoyancy resistance. A design objective of 1.5 and 1.3 should be adopted against the buoyant uplift forces for long term and short term conditions, respectively. It is recommended to assess resistance to buoyancy assuming design groundwater level at elevation 230.0 m. The bulk soil unit weight should be used above the design groundwater level and buoyant soil unit weight should be used below the design groundwater level.

The potential to account for side friction along the structure and values at soil/wall and soil/soil interface should be determined based on the nature and method of placement and compaction of the backfill material. Further recommendations can be provided in conjunction with detailed design phase.

7. Retaining Walls

The proposed project includes construction retaining walls at the southeast corner of Waverley Street/Taylor Avenue and at the abutments of the proposed underpass structure. Design considerations for walls supporting cuts and fills are presented in the following sections.

All retaining walls should be designed to support earth lateral pressure, hydrostatic pressure, if applicable, and lateral forces from live load surcharge including railway traffic as per AREMA Manual and CN guidelines and other potential use of the site. Retaining walls should include a suitable drainage system to protect against buildup of hydrostatic pressures behind the wall. Wall drainage typically consists of a layer of free-draining sand/gravel mixture in conjunction with a perforated drainage pipe connected to a suitable discharge point. Geo-composites products can be used behind the walls to facilitate drainage. Retaining walls may also be equipped with weep holes to protect against buildup of hydrostatic pressure. A provision for drainage should be provided to protect against the development of hydrostatic water pressure behind sheet pile and secant pile walls, if used. Wall movement depends on design factors, including type of wall being used; stiff wall is more stable than flexible wall by providing more restriction against lateral movement, however high cost may be associated with a rigid retaining wall system.

7.1 Wall Alternatives

Reinforced concrete retaining walls are the common type locally used in the Winnipeg area. Other wall types including MSE walls, sheet pile and secant pile walls were used on limited basis. Soldier pile walls are mostly used for temporary work to provide excavations support.

The availability of construction space and the proximity to and potential impact on existing buildings/installations are among the governing factors that define the wall types. Traditional gravity type walls (i.e., reinforced concrete and MSE wall) are constructed in bottom-up fashion and require considerable space behind the wall. Temporary shoring is often necessary in conjunction with the construction of a gravity wall for cut applications in urban environment. In sites of limited space or when the new cut wall is in close proximity to existing buildings, gravity type walls may not be feasible and embedded type walls are considered more viable alternatives. Embedded walls include sheet pile walls or secant pile walls with/without tie backs. These walls are constructed in top-down fashion and are installed prior to excavation in front of the wall. The construction of embedded walls lends itself for stage construction and can be designed efficiently to reduce temporary shoring requirements.

Based on the developed design concepts, an embedded wall could be considered at the southeast corner of Waverley Street/Taylor Avenue intersection to protect/retain the existing PIAZZA DE NARDI monument. The available space, construction sequence and the potential for interaction and impact with the existing monument foundations should be reviewed as part of the detailed design stage. .

7.2 Lateral Loads

Lateral earth pressures transferred to abutments or to retaining walls will be a function of the backfill/retained material, the method of placing and compacting the backfill, and the amount of horizontal deflection allowed by the abutment or the wall after the backfill is placed. It is recommended that abutments and walls be backfilled with a free draining granular material containing a maximum of 5 percent fines (maximum of 5 percent finer than #200 sieve). Cohesive soils are not recommended for backfill behind retaining structures. For free draining coarse granular soils, an active earth pressure coefficient (K_a) of 0.30 can be used in the design of walls that allowed to translate or deflect horizontally by at least 0.2 percent of the retained height. For retaining structures, which are not free to translate, an at-rest earth pressure coefficient (K_o) of 0.5 should be used. Compaction of backfill within about 1.5 m of the wall should be conducted using a light hand operated vibrating plate compactor. Over-compaction of the backfill may result in earth pressures that are considerably higher than those predicted in design. Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

Embedded walls retain predominantly natural ground. The in-situ (at-rest) earth pressure of clay deposit depends on the geological stress history. Over-consolidated clay, as the case for the approximate top 5 m of Winnipeg clay, exhibits an at-rest earth pressure coefficient greater than unity. Wall installation may modify (increase/decrease) the horizontal earth pressure close to the wall from the in-situ values. Walls of driven piles may increase the lateral stresses, bored piles may result in reduction. The lateral pressure distributions on the retained side should be extended to the base of the wall system (i.e., the bottom of the piles). The wall must be embedded deeply enough to provide adequate kick out resistance for the portion of the wall below the excavation.

In addition to earth lateral pressure, the walls should be designed to resist lateral loads from other applicable surcharges including railway and construction loading, traffic loads, and loads that may arise from interference with foundation of existing building.

The nominal passive resistance in front of permanent walls can be assumed as shown on Figure 10. Passive resistance should only be accounted for from soils 2.0 m below the final grade in front of the wall. Resistance factor of 0.50 should be used to determine the factored passive resistance. The associated displacement to mobilize the maximum passive soil resistance should be evaluated against tolerable wall movement.

Further assessment will be required to assess the soil design parameters and impact of tie-back installation, if required, on design loads as part of detailed design phase.

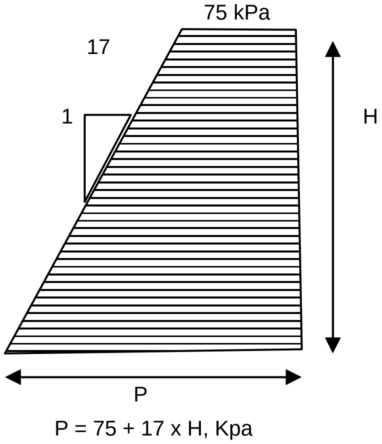


Figure 10 – Nominal Passive Earth Resistance in front of Retaining Wall

7.3 Tieback Anchors

Tieback anchors embedded in the native or compacted fill soils behind the wall can provide outward movement control of the wall. Shallow tieback can be designed to mobilize resistance from passive resistance in front of deadman block/wall. The deadman should be located outside the active wedge in the area defined by a line starting at the ground surface perpendicular to the active wedge boundary under sufficient soil cover as illustrated on Figure 11.

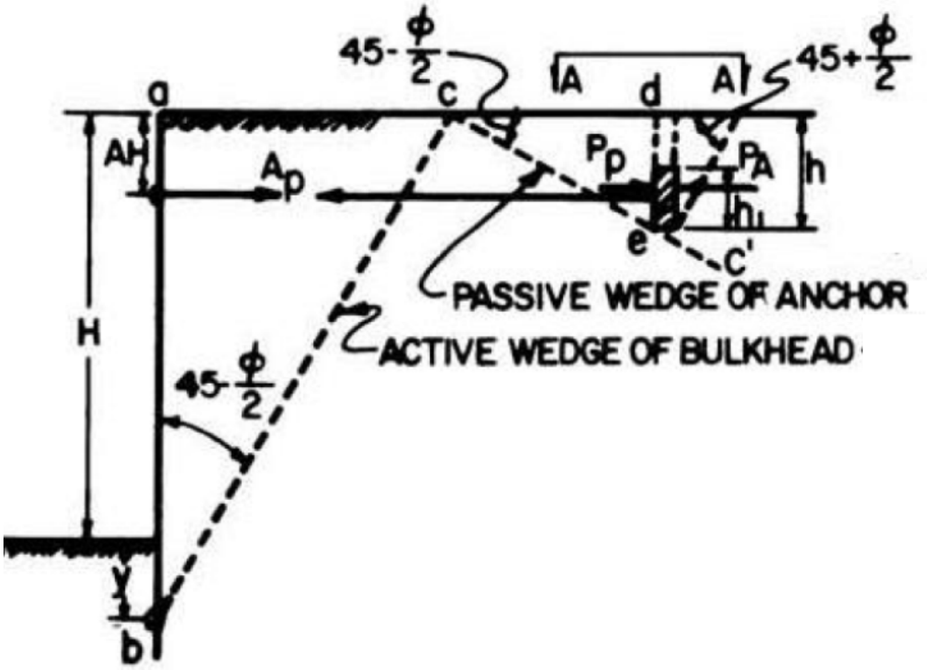


Figure 11 –Deadman Anchorage Location

7.4 External Stability

Walls final configuration should be designed to satisfy design objectives related to bearing capacity, sliding, overturning and overall stability. The external stability review can be completed as part of the detailed design stage.

8. Trenchless Pipe Installation

There are two methods of pipe jacking practiced locally. One utilizes the Akkerman system while the other is a variation of the Atkins coring system. Both methods follow a similar construction approach and result in similar ground response. A brief description for each method is provided herein:

8.1 Akkerman System

The Akkerman installation method requires a jacking shaft from which the pipe installation starts and a receiving shaft at the end of the pipe length to retrieve the Tunnel Boring Machine (TBM) which would be used to excavate underground along the pipe alignment. The TBM has a rotating cutterhead that rotates and excavates the soil which comes inside the cutting head. The spoil is transferred to the rear of the shield through conveyers which dump it into muck carts or conveys it out of the tunnel or the pipe being installed. Thrust power of hydraulic jacks is utilized to force the TBM and the following string of pipes forward. The hydraulic pressures overcome face resistance and friction forces on the exposed surfaces of the shield and installed pipes.

Drive lengths up to 120 m have been successfully achieved in Winnipeg area using this method. However, since the method requires personnel working inside the pipe, the method is limited to man entry size boring. Even though it is theoretically possible for a person to enter a 900 mm diameter bore, it is practically difficult for the person to work in it. Locally, 1050 mm diameter pipes are the minimum size installed using this method.

8.2 Atkins System

The Atkins jacking method is a variation of Atkins traditional coring method. This method requires a shaft on both ends of the pipe length to be installed. Three steel rods are driven through from shaft to shaft along the center of the proposed pipe alignment (one at the centre and one on each side). A push-pull earth coring knife is attached to the center rod and front cutting and a shielding rim is attached to the two outer rods. The first pipe section is placed so that it abuts to the front cutting and shielding rim securely. A pulling and holding rim connected to the outer rods and secured against the back of the pipe section is used to advance the pipe forward. The rods are pulled, or jacked, towards the opposite shaft to move the whole assembly through the soil. The spoil removed from the coring knife as necessary by pushing the knife forward. Once a pipe section is installed, additional section is added and the installation process continued. Drive length between shafts is limited to 30 to 35 m. Pipe diameter up to 1600 mm was installed locally using the Atkins system.

8.3 Face Stability

The Face Stability Index illustrated in Figure 12, frequently referred to as the overload factor (OF), is the ratio of the difference between the vertical pressure at tunnel axis and the pressure applied to the tunnel face, and the undrained shear strength. In cohesive soils, the tunnel face is considered stable when the index is less than six. While the limiting value of OF=6 represents a threshold of serious problems, a value of OF=5 represents a practical limit below which tunnelling may be carried out without unusual difficulties.

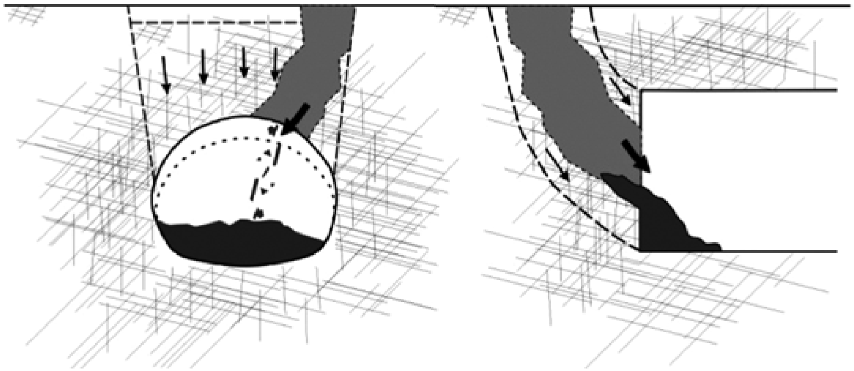


Figure 12 – Concept of Face Stability

A preliminary assessment for face stability was completed assuming a range of typical values for undrained shear strength and bulk unit weight and assuming pipe inverts 4 to 7 m below ground, the estimated OF is between 2.5 and 5 which suggests that tunnel face stability is satisfactory. However, difficulties in face stability should be expected if localized soft clay zones or wet silt layer /seams are encountered along the tunnel/pipe alignment.

Caution should be exercised to monitor the face and minimize the time period associated with the tunnelling operations. A contractual requirement for continuous jacking operations under the railway tracks or other sensitive structures and visual observation of the cuttings to confirm that no silt zone has been encountered will allow remedial action to be undertaken in the unlikely event of experiencing face instabilities.

8.4 Ground Subsidence

Like other tunnelling methods, pipe jacking will result in a change in the state of stress in the ground with the corresponding displacements. Ground subsidence can be caused by several factors such as ground loss at the tunnel face, behind the tail of the shield and through the tunnel support or linings. Based on having a stable tunnelling face, the only significant contribution to ground loss is the closure of the over-cut. The over-cut is the annular space between the tunnel boring walls and the installed pipe.

Some degree of ground surface subsidence can be expected from tunneling although in many instances its effects, from a practical perspective are negligible. Empirical methods of predicting settlement due to tunnelling induced ground movements have been used extensively and successfully over the years. Most methods derived for estimating surface or subsurface subsidence are empirical in nature and based on field observations in the UK although the same computational methods have been successfully applied locally. The most common method is estimating the value of (i), a parameter used to define the distance from the tunnel centre line to the point of inflexion of the settlement trough of a normal probability curve as shown in Figure 13. The distribution of the settlements or settlement trough approximates a normal probability distribution function described as:

$$S_x = S_{max}\, exp\, [-x^2/2i^2] \quad \text{……………………Equation 1}$$

where S_x = surface settlement at a transverse distance (x) from the tunnel centre line
S_{max} = maximum settlement at $x = 0$
i = location of maximum settlement gradient or point of inflexion.

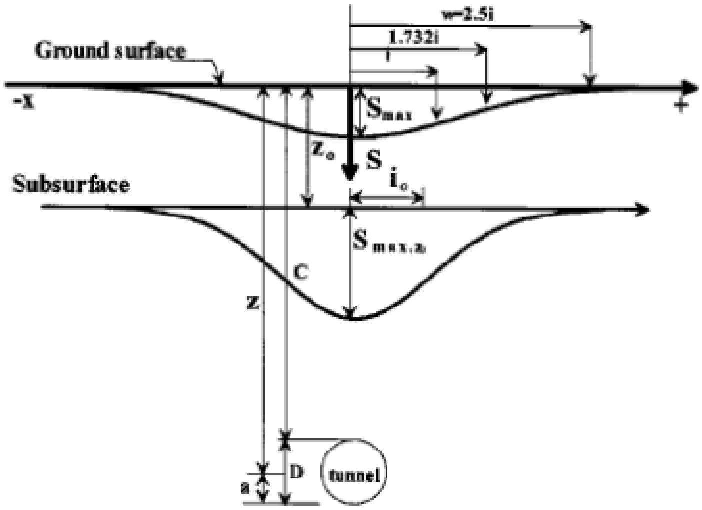


Figure 13 –Surface and Subsurface Settlement Trough

Based on Equation 1, the estimated i parameter, width of settlement trough and maximum settlement at ground surface and selected subsurface depths are presented in Table 8. In estimating these values, the volume of settlement trough, per unit length, was considered equal to the ground loss from the closure of 13mm over-cut between the excavated tunnel bore and the outer pipe wall. The over-cut size used in the above estimation is consistent with the local construction practice. As shown in Table 8 subsurface settlement troughs are narrower with larger settlement as compared to surface settlement.

Table 8 – Estimated Surface and Subsurface Settlement Trough Parameters

Depth (m)	i parameter (m)	Total Trough Width (approx. 5 i) (m)	Max. Settlement (mm)
Ground surface	3.68	19	5*
3.0 m below ground surface	2.34	12	8*
4.0 m below ground surface	1.96	10	10*
5.0 m below ground surface	1.53	8	12*

**Estimates are for 1.2 m diameter pipe installed at 6.5 m below ground surface using trenchless techniques*

To put these maximum anticipated values in perspective they are presented graphically using an exaggerated vertical scale on Figure 14. The maximum estimated subsidence at ground surface is in the order of 5 mm and it diminishes to zero across the width of the settlement trough which is estimated to be about 19 meters. The estimated extent and amount of the ground subsidence is not expected to be of concern and unlikely to impose adverse impact on existing infrastructures or utilities. However, each utility owner should be contacted to define and confirm acceptable surface/subsurface displacement and acceptable mitigation measures if required. Continuous monitoring during construction is recommended to monitor actual ground subsidence and protect against development of unanticipated conditions.

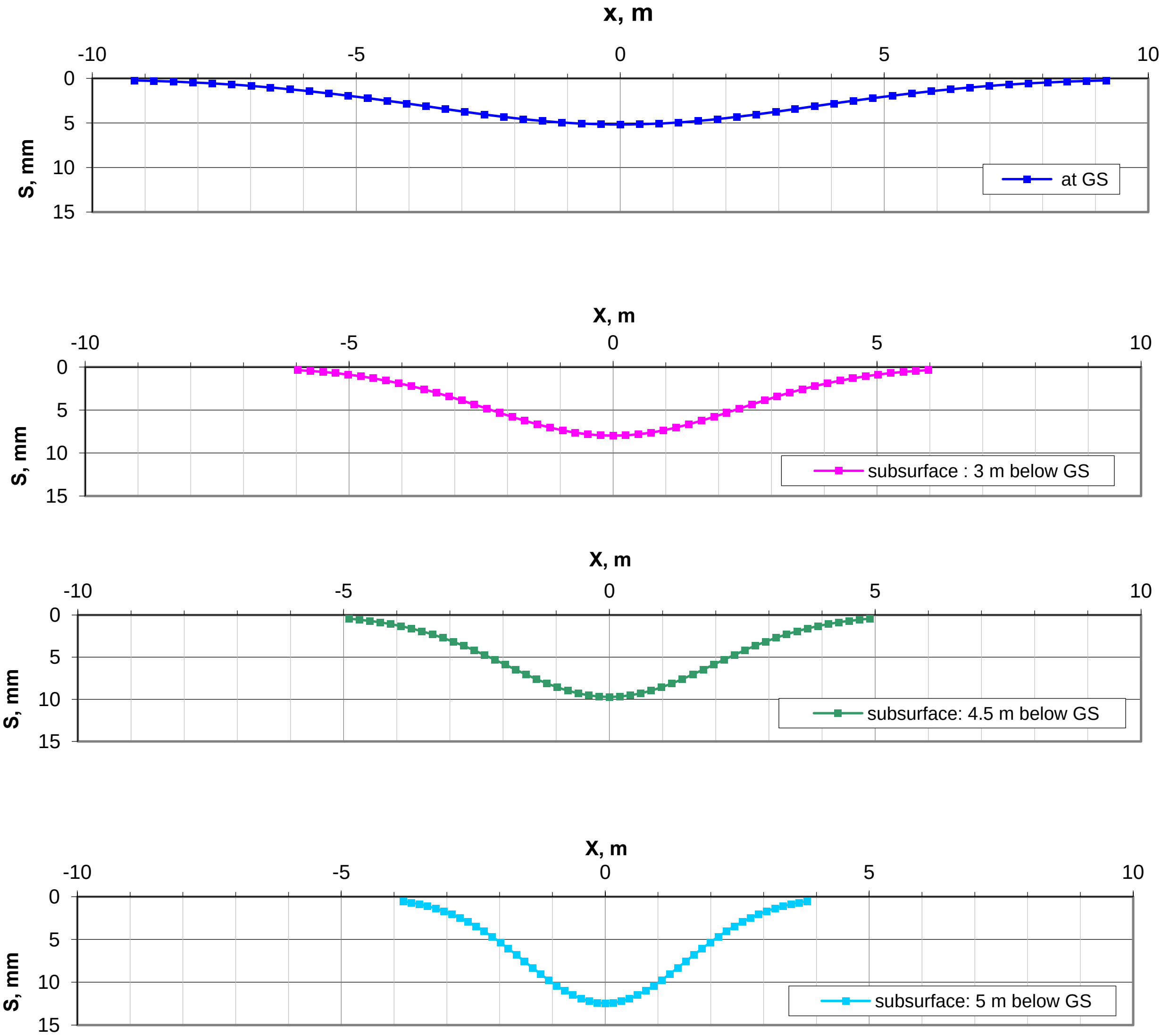


Figure 14 – Estimated Induced Surface and Subsurface Subsidence

9. Road Subgrade

The in situ and fill materials encountered at ground surface or underneath the thin layer of top soil along the proposed widening /improvement are expected to perform satisfactorily as roadway subgrade when compacted, confined and protected against erosion. The surficial clays underlying the topsoil layer is generally firm to stiff and should provide a suitable subgrade for roadway construction.

The Atterberg limit results for selective soil samples within 1 m depth below the proposed road work are presented graphically on Figure 15. Using the AASHTO M-145-91 classification, the soil may be classified as A-7-6 of high plastic clay and A-6 of intermediate plastic. Both A-6 and A-7-6 clay usually have high volume change between wet and dry states. When moisture content is properly controlled, they compact quite readily with a sheep foot roller. They have high dry strength but lose much of this strength upon absorbing water. These types of soil will compress when wet and shrink and swell with changes in moisture content. When placed in the shoulders adjacent to the pavement, they tend to shrink away from the pavement edge upon drying and thereby provide access for surface water to the underside of the pavement. Silt and/or silt predominate soil was identified at shallow depths in TH14-18, and TH14-21 to TH 14-27 along the proposed roadway works. Silt could be classified under AASHTO M-145-91 as A-4 or A-5 and is considered unsuitable material for road construction. It frequently has an affinity for water and can liquefy and lose stability unless properly drained. Silt does not drain readily and may absorb water by capillary action and it is frost susceptible. Also, silt predominate soils are often difficult to compact properly and will required high moisture control and confinement to attain acceptable compaction. All silt should be removed for a depth not less than half the frost penetration depth below the road surface. AASHTO classification for the tested soil samples are presented in Table 9**.**

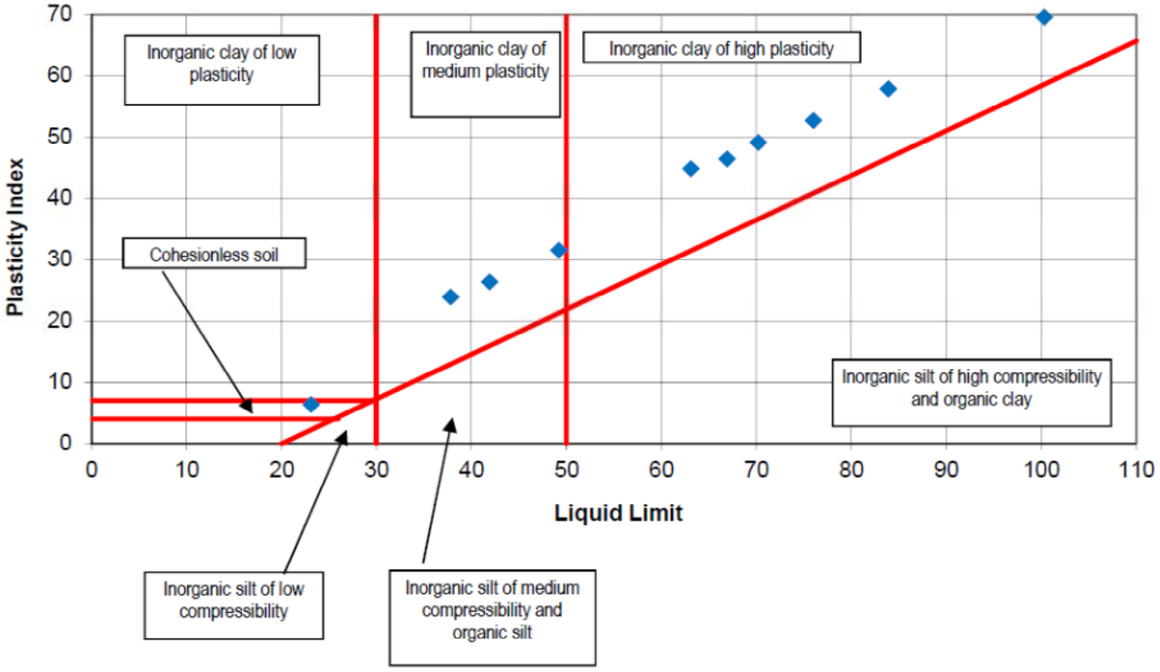


Figure 15 – Atterberg Limits Results

Table 9 – Laboratory Test Results and AASHTO Classification – Road Subgrade

Test hole No.	Location	Depth (m)	Liquid Limit	Plasticity index	Group classification	Group Index
TH14-07	South east shoofly	0.75	49.2	31.6	A-7-6	29
TH14-16	Taylor Av./west	0.75	66.9	46.5	A-7-6	50
TH14-17	Taylor Av./west	1.0	70.2	49.2	A-7-6	54
TH14-18	Road detour	Approximately 1.0 below proposed detour	23.2	6.5	A-4	4
TH14-21	Hurst Way	0.75	41.9	26.4	A-7-6	25
TH14-25	Taylor Av./east	0.75	37.8	23.9	A-6	22
TH14-28	Underpass	7.1	83.9	57.9	A-7-6	68

The subgrade surface should be scarified to a minimum depth of 150 mm and compacted to a minimum of 95 % of Standard Proctor maximum dry density (SPMDD). Silt rich soil, random fill, topsoil or organic should be treated as unsuitable subgrade and should be excavated and replaced with compacted suitable fill. In cases where the depth of excavation exceeds 750 mm below subgrade surface, the unsuitable material may be bridged with geotextile and granular fill. This approach is considered sufficient for bridging since removal of this material for the entire layer thickness will not be practical in some cases (i.e. the base of the silt layer is deep and excavation depths well in excess of 750 mm would be otherwise required). A woven geotextile should be placed between the native soil and the granular fill to provide separation and reinforcement. The geotextile should meet or exceed the following physical properties:

- Grab Tensile Strength of 1,400 Newtons (N);
- Puncture Strength of 530 N;
- Trapezoidal tear of 500 N; and
- Mullen Burst Value of 3,500 kPa.

The granular fill should consist of a 100 to 150 mm down crushed material and/or a 50 mm down crushed material. The 100 to 150 mm down material is suitable when fill depths greater than about 300 mm are required. A 150 mm thick layer (minimum) of the 50 mm down crushed fill should be provided between the 100 to 150 mm granular fill material and the granular base material for the pavement. The crushed granular fill should be compacted in uniform layers followed by proof rolling to attain compaction and verify that no soft or weak areas exist. If significant deformation (squeezing and bulking) of the subgrade occurs, compaction should be halted and an investigation undertaken to determine the cause of the deformation. For example, a wet silt layer at a shallow depth below the subgrade may require over-excavation or bridging. The subgrade should be proof rolled with a loaded tandem truck, or approved equivalent, having a gross vehicle weight of at least 20 tonnes to identify any soft areas before the granular base and pavement layers are placed. Each successive pass of the equipment used for proof rolling should be offset by not greater than one tire width to provide adequate coverage. The rolling pattern should be completed in a systematic fashion and the results recorded. Best results are generally obtained using ground speeds ranging from 4 to 8 km/h.

Areas identified as being weak or soft during proof rolling should be stabilized by additional re-working and compaction or removal and replacement with suitable material. Any softened or weak areas should be bladed aside and the underlying material scarified and re-compacted. The excavated material, if suitable, should then be bladed back and compacted to a minimum of 95 percent of SPMDD. Cuts across the roadway alignment should be sloped at a maximum (i.e. no steeper than) of 5H:1V to minimize the potential for differential movement beneath the pavement. Once filled to subgrade elevation, proof rolling of these areas should be completed.

10. Railway Detour

Subgrade characterization and preparation discussion provide in Section 9 is applicable to subgrade along the proposed railway detour. Railway grade could be constructed using clay or granular fill. It is understood that the proposed detour grade will be about 1 m above existing grade (top of ballast to toe of fill). For fill not exceeding 1 m in heights a 2H:1V side slopes can be used for fill placed in layers not exceeding 200 mm in loose thickness and compacted to 95 percent of SPMDD.

11. Key Recommendations and Future Works

- Geotechnical Investigation: Additional test hole drilling particularly at the exact locations of the structure support units should be completed during the detailed design phase.
- Geotechnical Investigation: Additional test hole drilling along the proposed pipe route to identify soil units through which the proposed pipe will be installed.
- Hydro-geological Exploration: Assessment of existing groundwater users and potential impact form construction activities should be completed as part of the detailed design phase. This assessment may include well installation and pump test.
- Groundwater Monitoring: Continue groundwater monitoring to verify and confirm related design assumptions.
- Overpass Structure Foundation: Steel H piles are recommended at the abutments and rock socketed caissons are recommended at the intermediate piers. Test caisson installation is recommended.
- Retaining Wall Foundation: It is recommended to support gravity retaining wall on deep foundation system.
- Lift Station Foundation: It is recommended to support the deep portion of the lift station on raft foundation and support the shallow portion on driven piles bearing into the till at the level of the raft or deeper.
- Temporary Excavations: Complete hydro-geological assessment as Aquifer depressurization and groundwater control will be required to facilitate the construction of the lift station.
- Slope Stability: Slope configuration of two slopes and intermediate bench will be required to attain the design objective factor of safety. The cut slopes will be 4H:1V for excavation shallower than 6 m and a combination of 4H:1V and 5H:1V for excavation between 6 and 7 m.
- Buoyancy and Uplift: The structural design for all buried structures under groundwater should consider the buoyant forces. A design groundwater elevation of 230 m is recommended.
- Trenchless Pipe Installation: Trenchless installation is feasible, settlement monitoring is recommended during construction at railway crossing and other sensitive installation.
- Geotechnical and Hydro-geological assessment will be required during the detailed design to confirm and supplement the finding of the preliminary design phase.

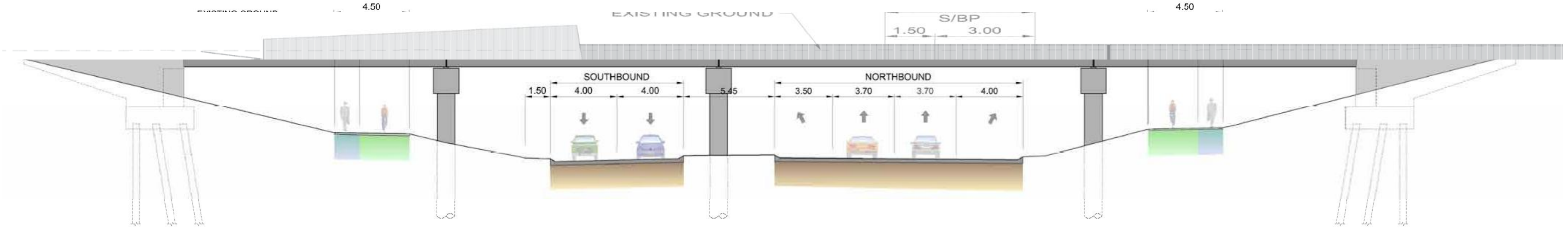
Appendix A

- General Arrangement Figures
- Test Hole Location Plan
- Schematics Soil Stratigraphy

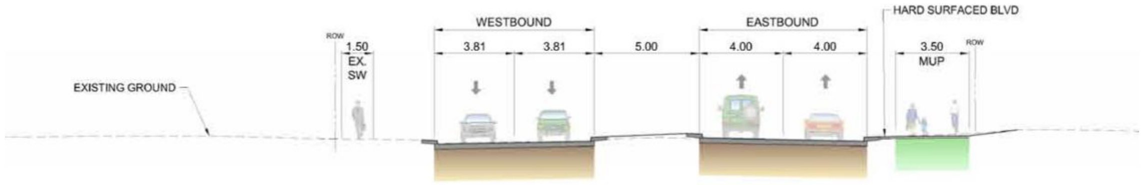
Waverley Underpass (CN Rail Line) - Preliminary Design Study

Draft Concept - Overall Plan

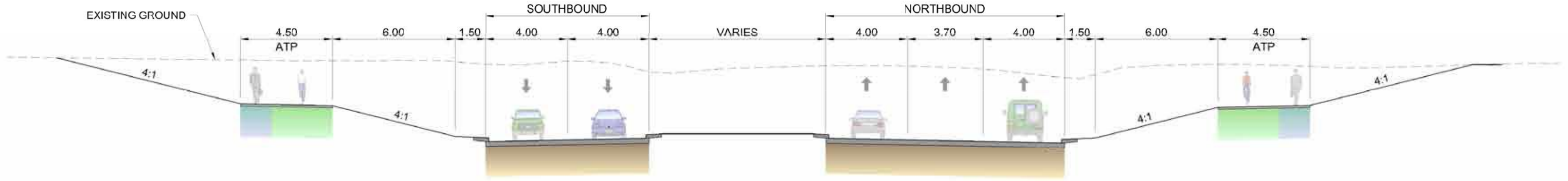
Draft Concept - Cross Sections



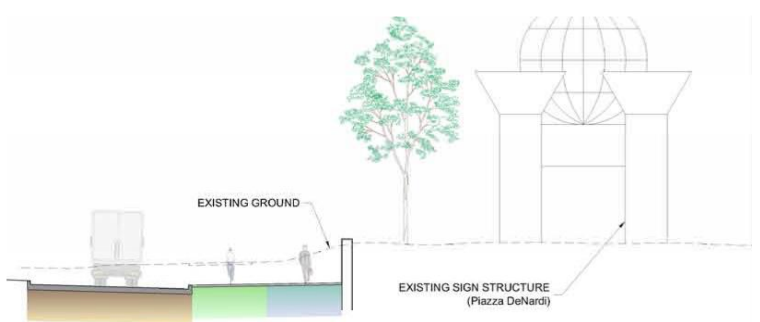
A The Waverley Underpass



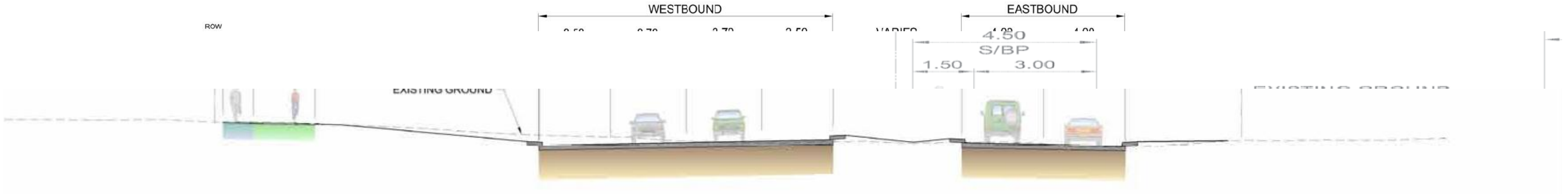
D Taylor Avenue - West of Waverley (Looking East)



B Waverley Street (Looking North)



E Right Turn Lane Waverley Northbound at Taylor



C Hurst Avenue - East of Waverley (Looking East)

ANSI B 279.4mm x 431.8mm

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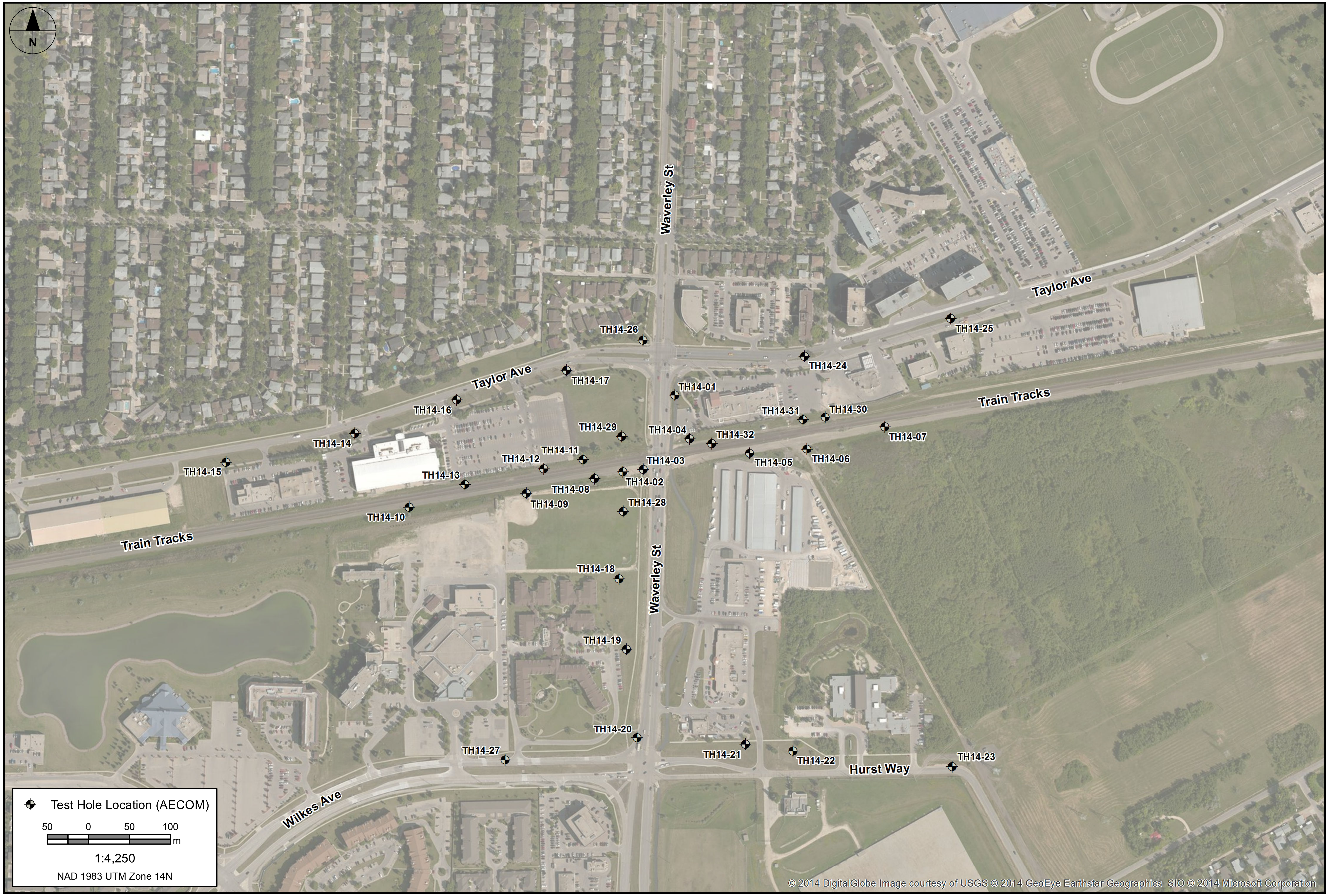

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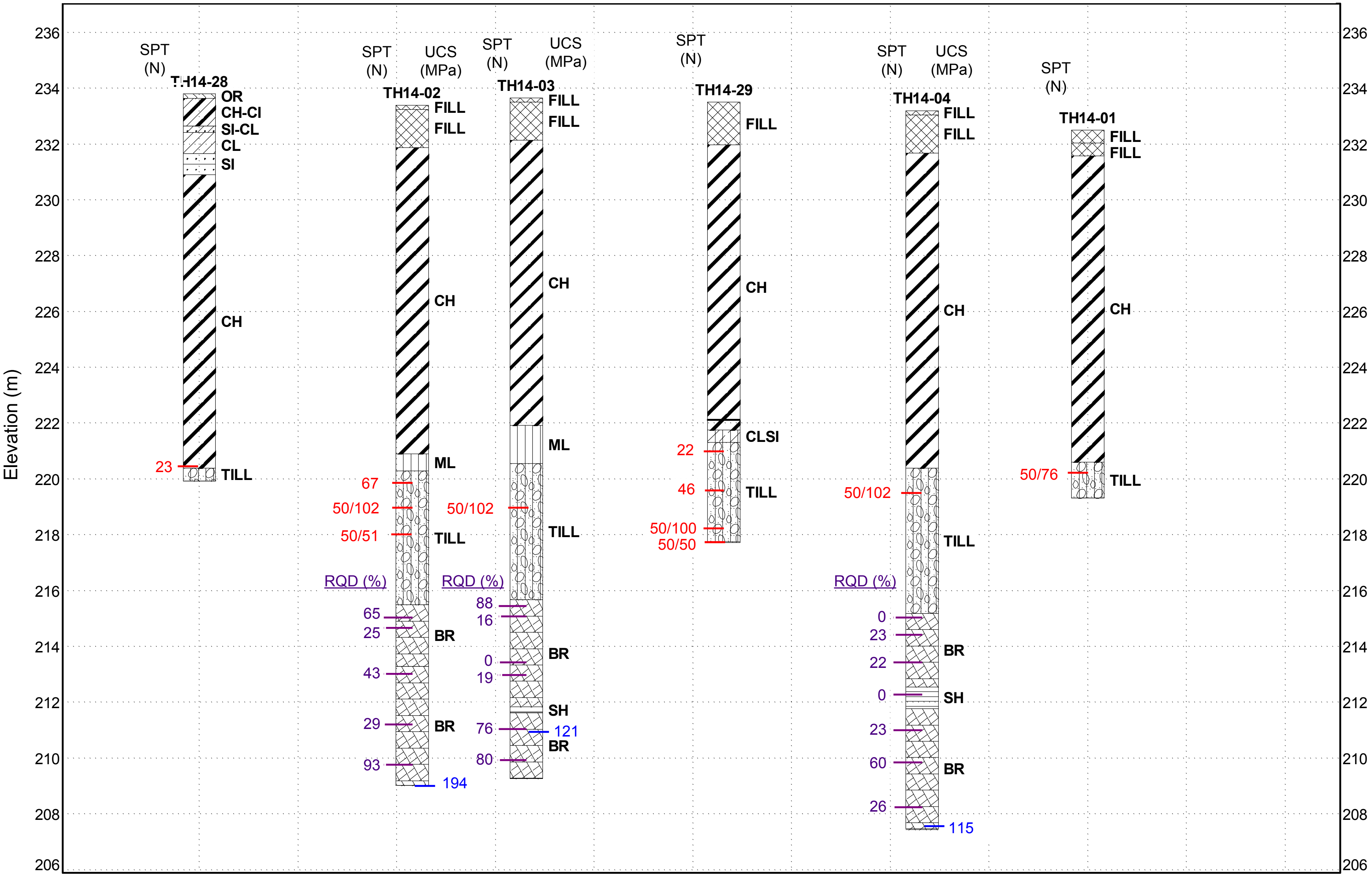
Drawing: 01

Test Hole Location Plan

Waverly Underpass Pre-Design

Dillon Consulting Limited
Project No.: 60321148

CLIENT: City of Winnipeg | PROJECT: Waverley underpass | PROJECT NO: 60321148



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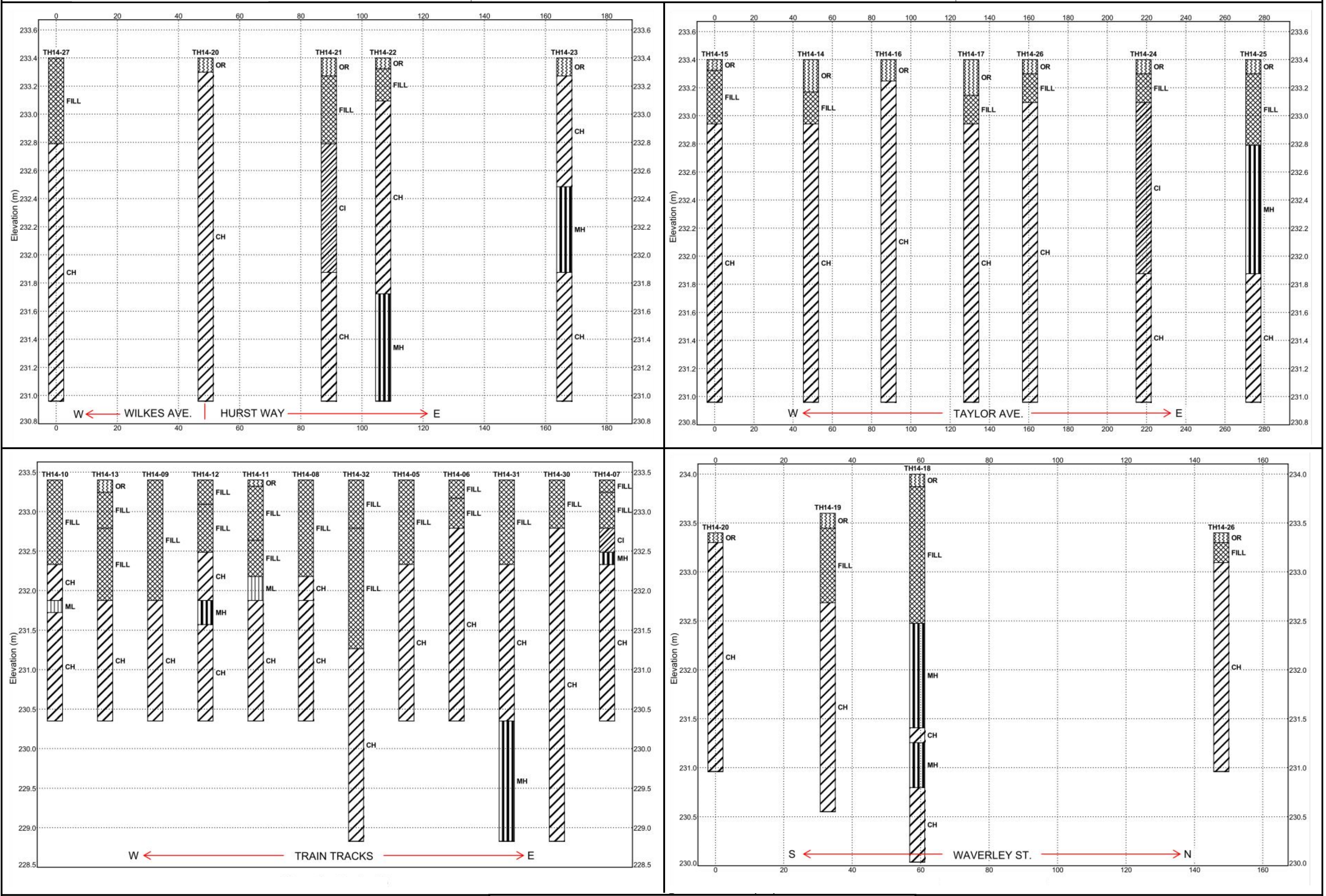
Note: Horizontal distance not to scale

Schematic 01: Soil stratigraphy for deep test-holes

STRATIGRAPHY & GW ELEV WAVERLEY UP- TEST HOLE LOGS - REVISION 5 (RH).GPJ UMA.GDT 12/5/14

CLIENT: City of Winnipeg | PROJECT: Waverley underpass | PROJECT NO: 60321148



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Notes: Horizontal distance not to scale
Elevations are estimated

Schematic 02: Soil stratigraphy for shallow test-holes

STRATIGRAPHY & GW ELEV WAVERLEY UP- TEST HOLE LOGS - REVISION 5 (RH).GPJ UMA.GDT 12/5/14

Appendix B

Test Hole Logs

AECOM Canada Ltd.

GENERAL STATEMENT

NORMAL VARIABILITY OF SUBSURFACE CONDITIONS

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability for the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of earth work, foundations and similar. In the event of any changes in the basic design or location of the structures as outlined in this report or plan, we should be given the opportunity to review the changes and to modify or reaffirm in writing the conclusions and recommendations of this report.

The analysis and recommendations presented in this report are based on the data obtained from the borings and test pit excavations made at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere are not significantly different from those disclosed by the borings and excavations. However, variations in soil conditions may exist between the excavations and, also, general groundwater levels and conditions may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions differ from those encountered in the exploratory borings and excavations, are observed or encountered during construction, or appear to be present beneath or beyond excavations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Since it is possible for conditions to vary from those assumed in the analysis and upon which our conclusions and recommendations are based, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction procedures.

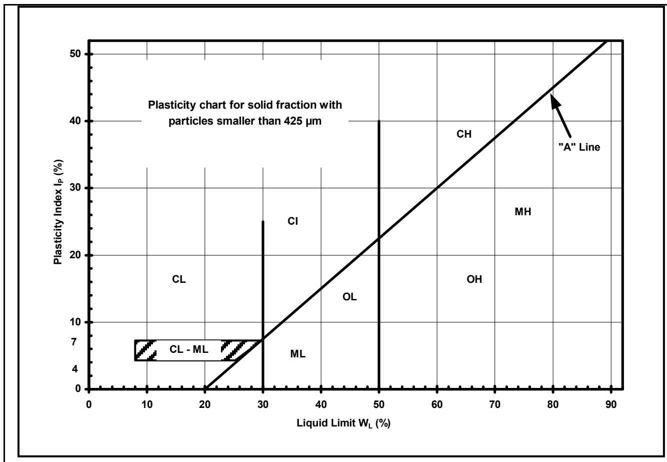
In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced soils engineer. We can be retained to provide these services for you during construction. In addition, we can be retained to review the plans and specifications that have been prepared to check for substantial conformance with the conclusions and recommendations contained in our report.

EXPLANATION OF FIELD & LABORATORY TEST DATA

Description				UMA Log Symbols	USCS Classification	Laboratory Classification Criteria			
						Fines (%)	Grading	Plasticity	Notes
COARSE GRAINED SOILS	GRAVELS (More than 50% of coarse fraction of gravel size)	CLEAN GRAVELS (Little or no fines)	Well graded gravels, sandy gravels, with little or no fines	△	GW	0-5	$C_U > 4$ $1 < C_C < 3$		Dual symbols if 5-12% fines. Dual symbols if above “A” line and $4<W_P<7$ $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10} x D_{60}}$
			Poorly graded gravels, sandy gravels, with little or no fines	▲	GP	0-5	Not satisfying GW requirements		
		DIRTY GRAVELS (With some fines)	Silty gravels, silty sandy gravels	▥	GM	> 12		Atterberg limits below “A” line or $W_P<4$	
			Clayey gravels, clayey sandy gravels	▨	GC	> 12		Atterberg limits above “A” line or $W_P<7$	
	SANDS (More than 50% of coarse fraction of sand size)	CLEAN SANDS (Little or no fines)	Well graded sands, gravelly sands, with little or no fines	⁘	SW	0-5	$C_U > 6$ $1 < C_C < 3$		
			Poorly graded sands, gravelly sands, with little or no fines	○	SP	0-5	Not satisfying SW requirements		
		DIRTY SANDS (With some fines)	Silty sands, sand-silt mixtures	○	SM	> 12		Atterberg limits below “A” line or $W_P<4$	
			Clayey sands, sand-clay mixtures	▨	SC	> 12		Atterberg limits above “A” line or $W_P<7$	
FINE GRAINED SOILS	SILTS (Below ‘A’ line negligible organic content)	$W_L<50$	Inorganic silts, silty or clayey fine sands, with slight plasticity	▥	ML			Classification is Based upon Plasticity Chart	
		$W_L>50$	Inorganic silts of high plasticity	▥	MH				
	CLAYS (Above ‘A’ line negligible organic content)	$W_L<30$	Inorganic clays, silty clays, sandy clays of low plasticity, lean clays	▨	CL				
		$30<W_L<50$	Inorganic clays and silty clays of medium plasticity	▨	CI				
		$W_L>50$	Inorganic clays of high plasticity, fat clays	▨	CH				
	ORGANIC SILTS & CLAYS (Below ‘A’ line)	$W_L<50$	Organic silts and organic silty clays of low plasticity	┆	OL				
		$W_L>50$	Organic clays of high plasticity	▨	OH				
HIGHLY ORGAINIC SOILS			Peat and other highly organic soils	〰	Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

■	Asphalt	○	Till		
⁘	Concrete	▨	Bedrock (Undifferentiated)		
▩	Fill	▦	Bedrock (Limestone)		

When the above classification terms are used in this report or test hole logs, the designated fractions may be visually estimated and not measured.



FRACTION		SEIVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		Passing	Retained	Percent	Identifier
Gravel	Coarse	76	19	35-50	and
	Fine	19	4.75		
Sand	Coarse	4.75	2.00	20-35	“y” or “ey” *
	Medium	2.00	0.425		
	Fine	0.425	0.075	10-20	some
Silt (non-plastic) or Clay (plastic)		< 0.075 mm		1-10	trace

* for example: gravelly, sandy clayey, silty

Definition of Oversize Material

COBBLES: 76mm to 300mm diameter
BOULDERS: >300mm diameter

LEGEND OF SYMBOLS

Laboratory and field tests are identified as follows:

q_u - undrained shear strength (kPa) derived from unconfined compression testing.

T_v - undrained shear strength (kPa) measured using a torvane

pp - undrained shear strength (kPa) measured using a pocket penetrometer.

L_v - undrained shear strength (kPa) measured using a lab vane.

F_v - undrained shear strength (kPa) measured using a field vane.

γ - bulk unit weight (kN/m^3).

SPT - Standard Penetration Test. Recorded as number of blows (N) from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 51 mm O.D. Raymond type sampler 0.30 m into the soil.

DPPT - Drive Point Pentrometer Test. Recorded as number of blows from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 50 mm drive point 0.30 m into the soil.

w - moisture content (W_L, W_P)

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Su (kPa)	**CONSISTENCY**
<12	very soft
12 – 25	soft
25 – 50	medium or firm
50 – 100	stiff
100 – 200	very stiff
200	hard

The resistance (N) of a non-cohesive soil can be related to compactness condition as follows

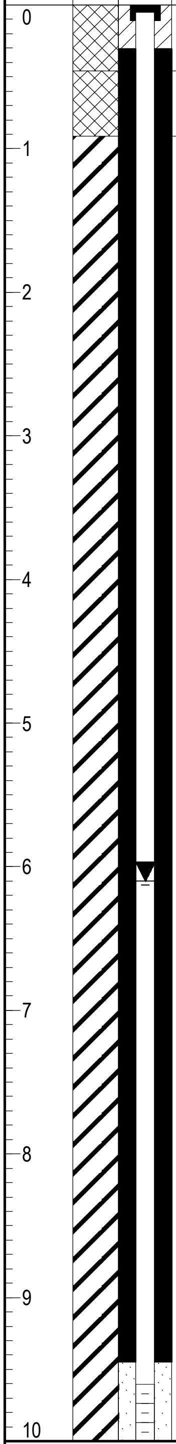
N – BLOWS/0.30 m	**COMPACTNESS**
0 - 4	very loose
4 - 10	loose
10 - 30	compact
30 - 50	dense
50	very dense

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-01**
LOCATION: UTM: 14U, 5523653 m N, 630934 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 232.50

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▧ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▧ CUTTINGS ⁘ SAND

DEPTH (m)	SOIL SYMBOL	SLOTTED PIEZOMETER	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS	UNDRAINED SHEAR STRENGTH	COMMENTS	ELEVATION



GRAVEL (FILL) - some sand, some limestones
- light grey, moist
- well graded,

CLAY (FILL) - trace to some silt, trace rootless, trace organic, trace oxidation
- black, soft to firm, moist

CLAY - silty, trace sand
- light brown, firm, moist
- high plasticity

- some to trace silt, silt inclusions < 6 mm in dia., mottled grey and brown below 2 m

- trace oxidation

- dark brown below 4 m

- soft to firm below 4.5 m

- grey

- some silt below 6 m

- soft, silt inclusion (12 mm in dia.) below 6.4 m

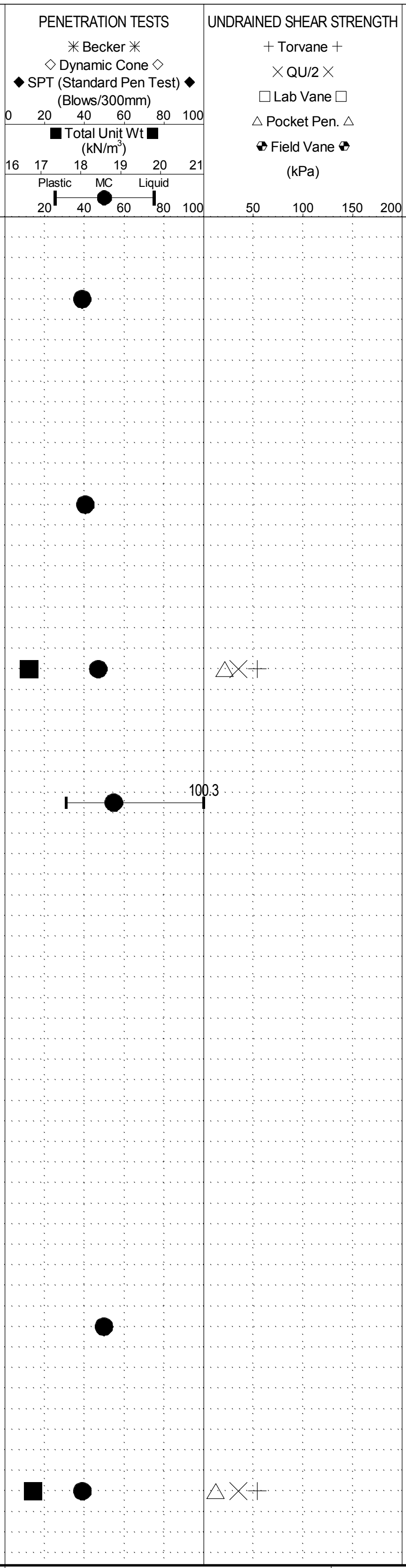
- trace gravel below 6.7 m

- silt inclusion (20 mm in dia.), trace gravel (angular 25 mm in dia)

- very soft below 8.3 m

- moist to wet below 9.7 m

SAMPLE TYPE	SAMPLE #
■	G1
■	G2
■	G3
▥	T4
■	G5
■	G6
▥	T7
■	G8
■	G9
■	G10
▥	T11



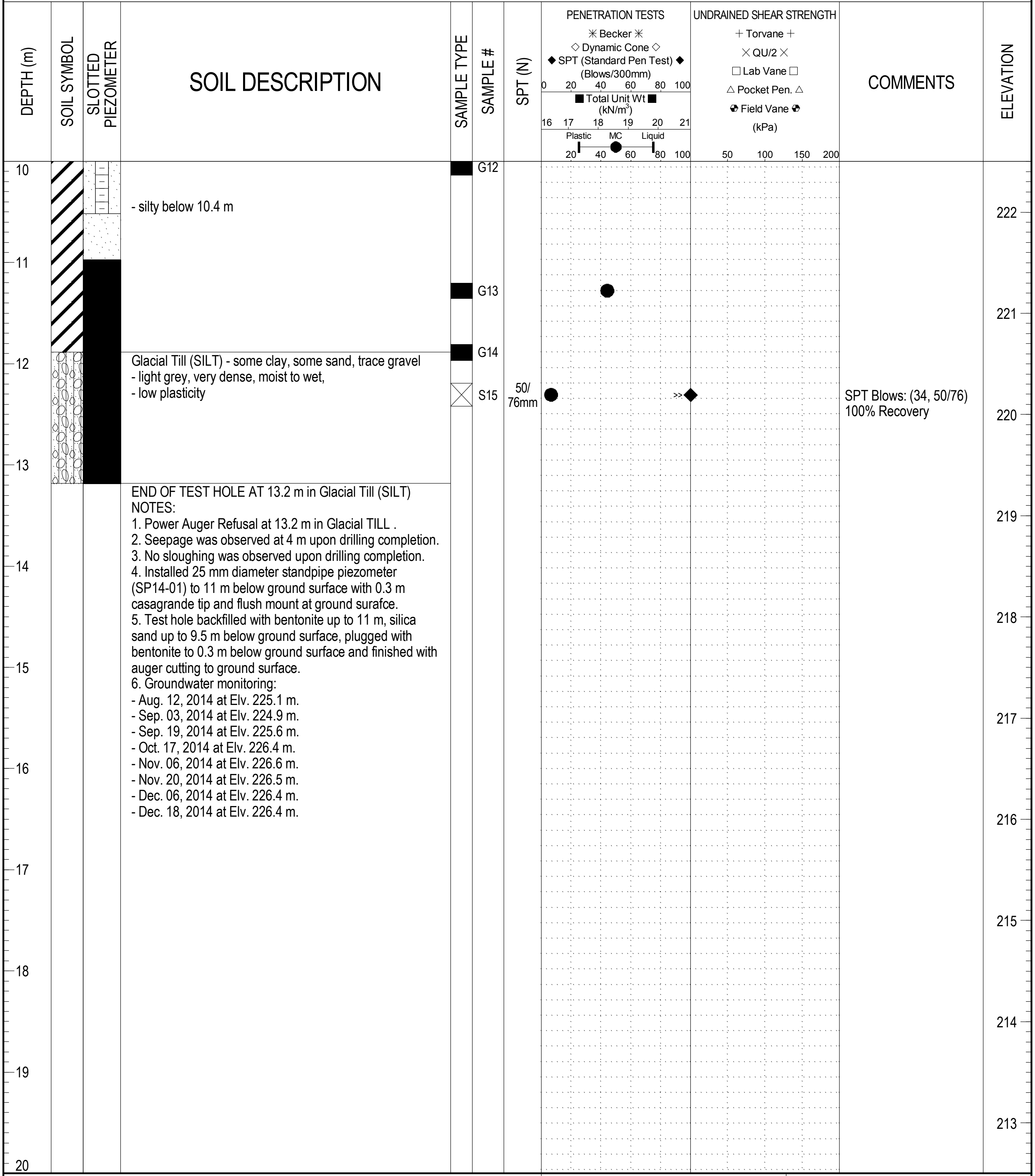
ELEVATION: 232, 231, 230, 229, 228, 227, 226, 225, 224, 223

AECOM	LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 13.18 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/9/14
	PROJECT ENGINEER: Faris Khalil	Page 1 of 2

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-01**
LOCATION: UTM: 14U, 5523653 m N, 630934 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m): 232.50

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



- silty below 10.4 m

Glacial Till (SILT) - some clay, some sand, trace gravel
- light grey, very dense, moist to wet,
- low plasticity

SPT Blows: (34, 50/76)
100% Recovery

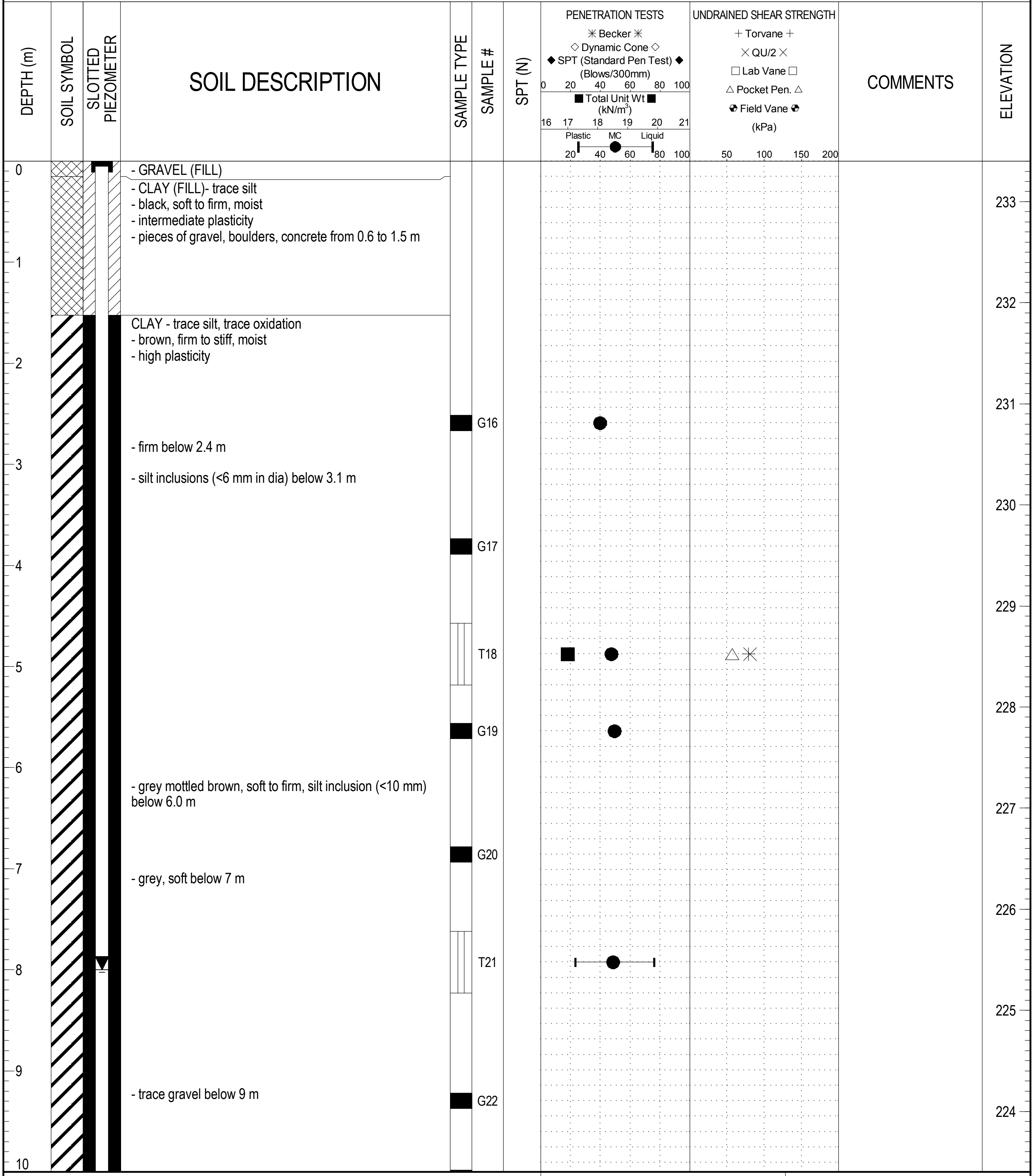
END OF TEST HOLE AT 13.2 m in Glacial Till (SILT)
NOTES:
1. Power Auger Refusal at 13.2 m in Glacial TILL .
2. Seepage was observed at 4 m upon drilling completion.
3. No sloughing was observed upon drilling completion.
4. Installed 25 mm diameter standpipe piezometer (SP14-01) to 11 m below ground surface with 0.3 m casagrande tip and flush mount at ground surafce.
5. Test hole backfilled with bentonite up to 11 m, silica sand up to 9.5 m below ground surface, plugged with bentonite to 0.3 m below ground surface and finished with auger cutting to ground surface.
6. Groundwater monitoring:
- Aug. 12, 2014 at Elv. 225.1 m.
- Sep. 03, 2014 at Elv. 224.9 m.
- Sep. 19, 2014 at Elv. 225.6 m.
- Oct. 17, 2014 at Elv. 226.4 m.
- Nov. 06, 2014 at Elv. 226.6 m.
- Nov. 20, 2014 at Elv. 226.5 m.
- Dec. 06, 2014 at Elv. 226.4 m.
- Dec. 18, 2014 at Elv. 226.4 m.

LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 13.18 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 7/9/14
PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-02**
LOCATION: UTM: 14U, 5523559 m N, 630870 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.40

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌CORE

BACKFILL TYPE ■ BENTONITE ⁖ GRAVEL ▥ SLOUGH ⁖ GROUT ▨ CUTTINGS ⁖ SAND



- GRAVEL (FILL)

- CLAY (FILL)- trace silt
- black, soft to firm, moist
- intermediate plasticity
- pieces of gravel, boulders, concrete from 0.6 to 1.5 m

CLAY - trace silt, trace oxidation
- brown, firm to stiff, moist
- high plasticity

- firm below 2.4 m

- silt inclusions (<6 mm in dia) below 3.1 m

- grey mottled brown, soft to firm, silt inclusion (<10 mm) below 6.0 m

- grey, soft below 7 m

- trace gravel below 9 m

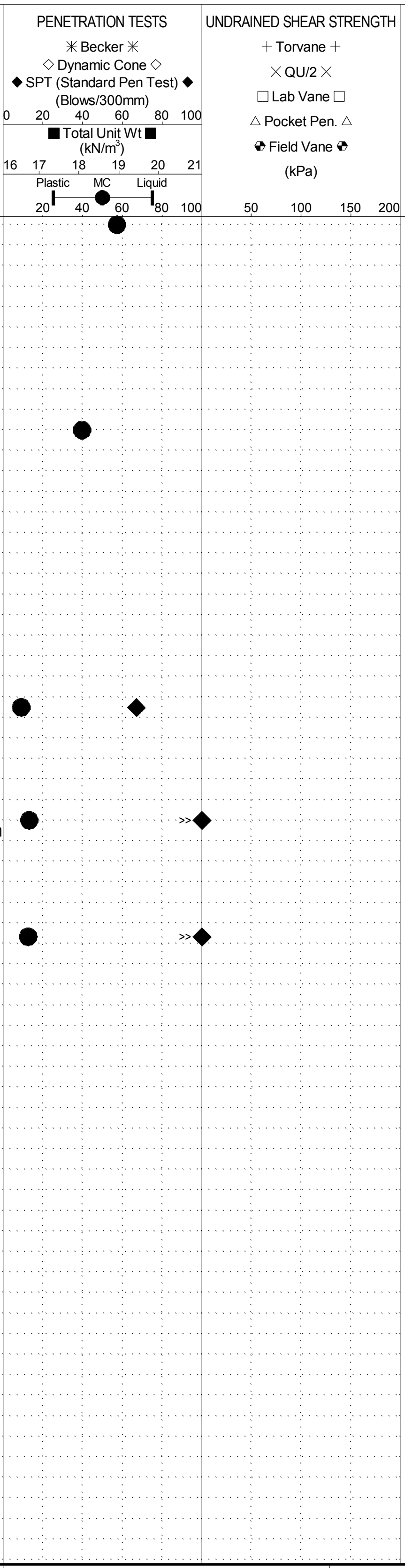
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/11/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 3

PROJECT: Waverley Underpass | CLIENT: City of Winnipeg | TESTHOLE NO: **TH14-02**
LOCATION: UTM: 14U, 5523559 m N, 630870 m E | PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd. | METHOD: 125 mm SSA/ HQ Coring | ELEVATION (m): 233.40

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10		■	G23			
	- silt inclusion (<30 mm in dia.) below 10.3 m					223
11		▥	T24			
	- some silt from 11.2 to 11.5 m					222
	- silty, light brown, soft , wet, low plasticity below 11.5 m	■	G25 G26			
12						
						221
	SILT -some gravel - light grey, very dense, moist to wet - low plasticity					
13		■	G27			
	Glacial Till (SILT)- some sand, some to trace gravel, trace clay - light grey, compact, moist to wet - low plasticity					220
		☒	S28	67	SPT Blows: (32, 43, 24) 61 % Recovery	
14						
	- ligth brown, some gravel below 14.4 m	☒	S29	50/ 102mm	SPT Blows: (35, 50/102) 89 % Recovery	219
	- trace gypsum					
15						
	- some gravel, some cobbles below 15.5 m	☒	S30	50/ 51mm	SPT Blows: (50/51) 100 % Recovery	218
16		▌	C1		C1 RQD: 0% C1 Recovery: 28 %	
						217
	- sandy below 16.7 m					
17		▌	C2		C2 RQD: 0% C2 Recovery: 100%	
		▌	C3A		C3A RQD: 0% C3A Recovery: 67%	216
18	LIMESTONE - fine grained, no foliation - creamish white - R3 - medium strong - close to moderately closed spacing,smooth, undulation, planar fractures, - no evidence of water flow (class 2) - fossiliferous - vuggy	▌	C3B		C3B RQD: 65% C3B Recovery: 100%	215
19		▌	C4		C4 RQD: 25% C4 Recovery: 90%	214
20						

AECOM

LOGGED BY: Saba Ibrahim | COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri | COMPLETION DATE: 7/11/14
PROJECT ENGINEER: Faris Khalil

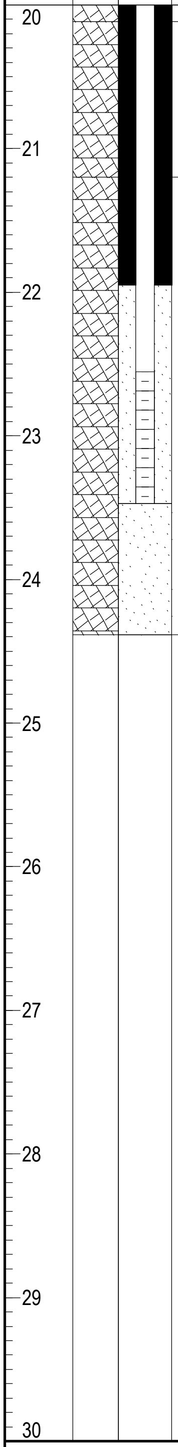
PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-02**
LOCATION: UTM: 14U, 5523559 m N, 630870 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.40

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▱ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND

DEPTH (m) | SOIL SYMBOL | SLOTTED PIEZOMETER | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE # | SPT (N) | COMMENTS | ELEVATION

PENETRATION TESTS: ✳ Becker ✳; ◇ Dynamic Cone ◇; ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane +; × QU/2 ×; ☐ Lab Vane ☐; △ Pocket Pen. △; ● Field Vane ● (kPa) 50 100 150 200



DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	COMMENTS	ELEVATION
20	- altered yellow and red below 20 m - extremely close to moderately closed spaced, smooth planar fractures - evidence of water flow (class 3)	C5	C5 RQD: 43% C5 Recovery: 98%	213
21	- laminated below 21.2 m - close spaced to moderately closed spaced, smooth planner fractures, - no evidence of water flow (class 2)	C6	C6 RQD: 29% C6 Recovery: 75 %	212 211
23	- R5- very strong	C7	C7 RQD: 93% C7 Recovery: 100 %, qu = 194.4 MPa	210
24.4	END OF TEST HOLE AT 24.4 m IN BEDROCK			209

Notes:
1. Power Auger Refusal at 15.4 m in Glacial TILL.
2. HQ coring below 15.4 m.
3. Seepage observed at 3.0 m upon drilling completion.
4. Installed 25 mm diameter standpipe piezometer (SP14-02) to 23.5 m below ground surface with 0.3 m casagrande tip and flush mount at ground surface.
5. Test hole backfilled with silica sand up to 22 m below ground surface, bentonite up to 1.5 m and plugged with auger cutting to ground surface.
6. Prominent sub-vertical fracture (180 degrees to core axis), closed to gapped, smooth undulating, evidence of water flow (class 3) between 17.9 to 18.4 m.
7. Groundwater monitoring:
- Aug. 12, 2014 at Elv. 225.29 m.
- Sep. 03, 2014 at Elv. 225.0 m.
- Sep. 19, 2014 at Elv. 225.5 m.
- Oct. 17, 2014 at Elv. 225.8 m.
- Nov. 06, 2014 at Elv. 225.7 m
- Nov. 20, 2014 at Elv. 225.6 m
- Dec. 06, 2014 at Elv. 225.4 m
- Dec. 18, 2014 at Elv. 225.4 m

208
207
206
205
204

AECOM

LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 24.38 m
COMPLETION DATE: 7/11/14

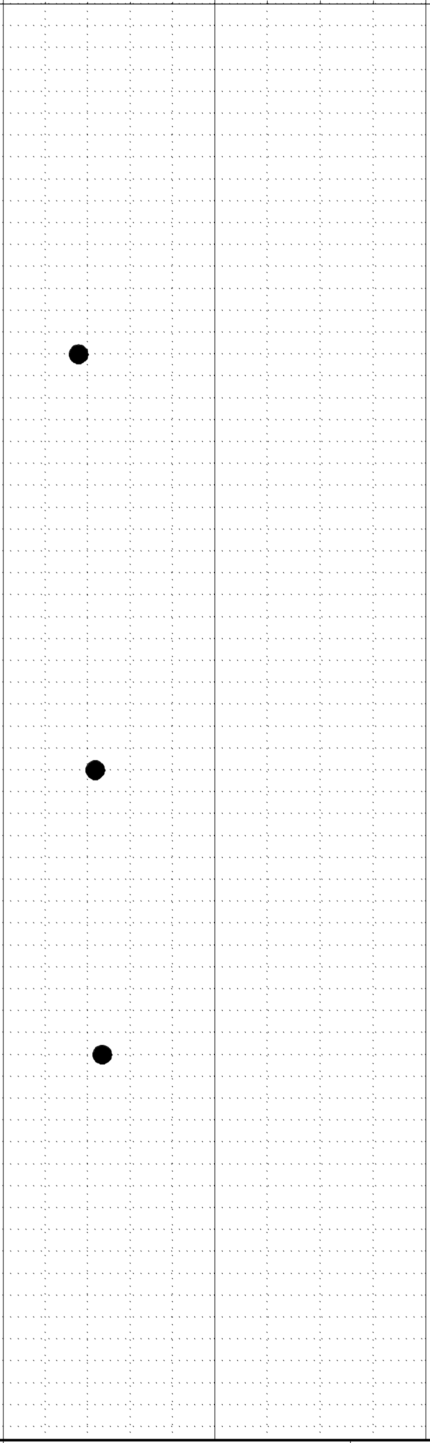
PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-03**
LOCATION: UTM: 14U, 5523562 m N, 630895 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.66

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ⧄ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
0	☒	-GRAVEL (FILL)					
	☒	- CLAY (FILL)-trace silt - black, soft to firm, moist - intermediate plasticity - pieces of gravel, boulders, concrete from 0.6 to 1.5 m					233
1							
	▨	CLAY - some silt, trace oxidation - dark brown, firm to stiff, moist - intermediate to high plasticity - silt inclusion (<12 mm in dia.) - brown mottled grey below 2.1 m					232
2			■	G31			231
3			▥	T32			
		- brown, high plasticity, firm below 3.7 m					230
4							
		- dark brown below 4.6 m					229
5		- firm , trace gypsum below 5.2 m	■	G33			228
6			▥	T34			227
7		- soft to firm, dark brown, trace gravel below 7 m	■	G35			
		- grey, soft, silt inclusion (6-30 mm in dia.) below 7.6 m					226
8			■	G36			225
9			▥	T37			224
10							

PENETRATION TESTS
✳ Becker ✳
◇ Dynamic Cone ◇
◆ SPT (Standard Pen Test) ◆
(Blows/300mm)
0 20 40 60 80 100
■ Total Unit Wt ■
(kN/m³)
16 17 18 19 20 21
Plastic MC Liquid
20 40 60 80 100

UNDRAINED SHEAR STRENGTH
+ Torvane +
× QU/2 ×
□ Lab Vane □
△ Pocket Pen. △
◉ Field Vane ◉
(kPa)
50 100 150 200



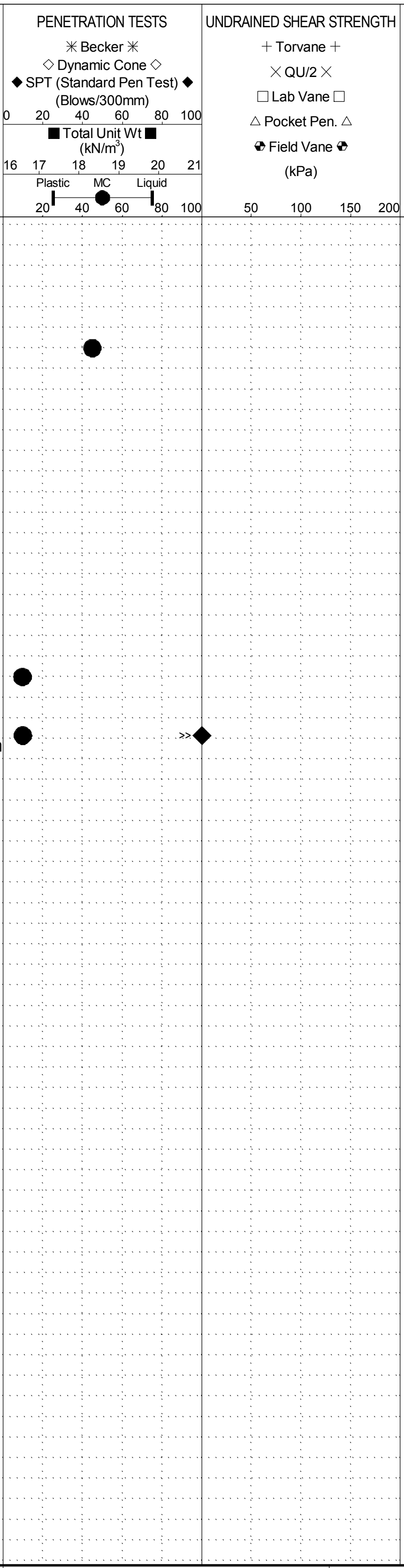
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/14/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 3

AECOM

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-03**
LOCATION: UTM: 14U, 5523562 m N, 630895 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.66

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10	- silt pocket , trace gravel below 10 m					
						223
11		■	G38			
	- very soft, moist to wet, light grey mottled gery below 11.3 m					
		■	G39			222
12	SILT - clayey, trace gravel - light brown, soft, moist to wet - intermediate to low plasticity					
		▥	T40			221
13						
	Glacial Till (SILT)- some sand, some gravel, some clay - light grey, very dense, moist - low plasticity	■	G41			220
		☒	S42	50/ 102mm	SPT Blows: (48, 50/102) 100 % Recovery	
14						
						219
15						
		▌	C1		C1 RQD: 0% C1 Recovery: 63 %	218
16						
	- ligth brown, gravelly below 16.3 m					217
17	- boulders form 16.9 to 17.5 m					
		▌	C2A		C2A RQD: 0% C2A Recovery: 74 %	216
18	LIMESTONE - fine grained - cremish white and grey - no foliation, vuggy - R3- medium strong - very closed to moderately spaced, rough undulating fractures, closed to gapped - no evidence of water flow (class 2)	▌	C2B		C2B RQD: 88% C2B Recovery: 95 %	
						215
19		▌	C3		C3 RQD: 16 % C3 Recovery: 88%	
						214
20						



LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/14/14
PROJECT ENGINEER: Faris Khalil	Page 2 of 3

PROJECT: Waverley Underpass | CLIENT: City of Winnipeg | TESTHOLE NO: **TH14-03**

LOCATION: UTM: 14U, 5523562 m N, 630895 m E | PROJECT NO.: 60321148

CONTRACTOR: Maple Leaf Drilling Ltd. | METHOD: 125 mm SSA/ HQ Coring | ELEVATION (m): 233.66

SAMPLE TYPE ■ GRAB ▯ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

DEPTH (m) | SOIL SYMBOL | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE # | SPT (N) | PENETRATION TESTS | UNDRAINED SHEAR STRENGTH | COMMENTS | ELEVATION

PENETRATION TESTS: ✳ Becker ✳; ◇ Dynamic Cone ◇; ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane +; × QU/2 ×; □ Lab Vane □; △ Pocket Pen. △; Field Vane (kPa) 50 100 150 200

Depth (m)	Soil Description	Sample #	Comments	Elevation
20	- recovered as coarse, sub angular to sub rounded light grey gravel between 20.3 to 21.9 m	C4	C4 RQD: 0% C4 Recovery: 100%	213
21		C5	C5 RQD: 19% C5 Recovery: 68 %	212
22	SHALE - very fine grained - blue, green - no foliation - R1- very weak - extremely close spaced, rough undulating fractures	C6	C6 RQD: 76% C6 Recovery: 100 %	211
23	LIMESTONE - white - fine grained - no foliation - R3- medium strong - close to moderately spaced, smooth fractures, closed, no evidence of water flow (class 2) - laminated below 22 m	C7	C7 RQD: 80% C7 Recovery: 100 %	210
24	- R5- very strong		qu =120.9 MPa	209

END OF TEST HOLE AT 24.4 m IN BEDROCK

Notes:

1. Power Auger Refusal at 14.3 m in Glacial TILL.
2. HQ coring below 14.3 m.
3. No sloughing was observed upon drilling completion.
4. No seepage was observed upon drilling completion.
5.Test hole backfilled with bentonite up to 3 m below ground level and with auger cutting to the ground surafce.

25 26 27 28 29 30 | 208 207 206 205 204

AECOM

LOGGED BY: Saba Ibrahim | COMPLETION DEPTH: 24.38 m

REVIEWED BY: Zeyad Shukri | COMPLETION DATE: 7/14/14

PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-04**
LOCATION: UTM: 14U, 5523599 m N, 630952 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.20

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ◿ CUTTINGS ⁘ SAND

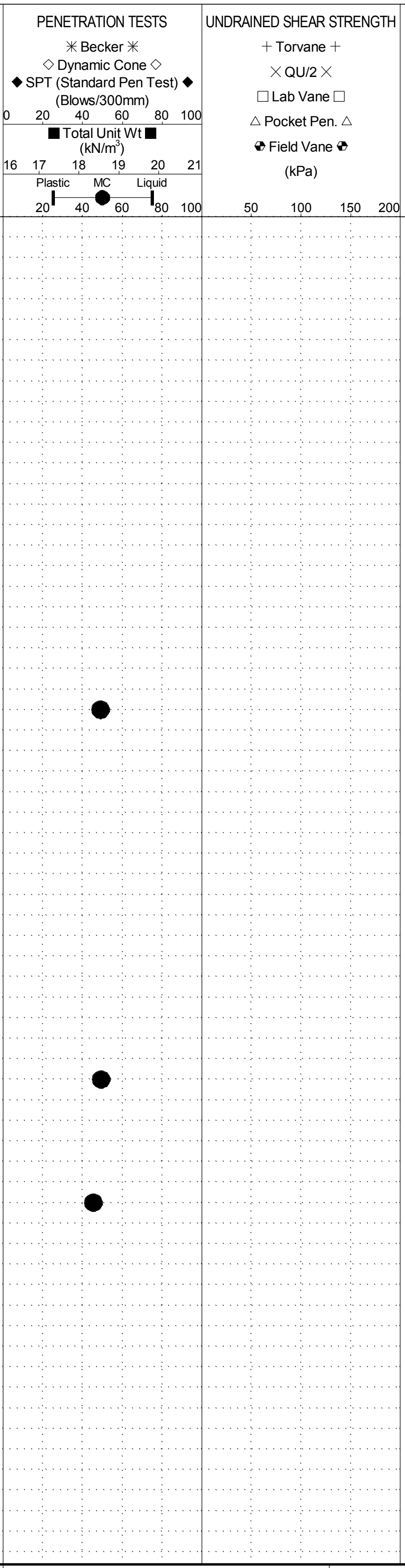
DEPTH (m) | SOIL SYMBOL | SLOTTED PIEZOMETER | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE # | SPT (N) | PENETRATION TESTS | UNDRAINED SHEAR STRENGTH | COMMENTS | ELEVATION

-GRAVEL (FILL)

- CLAY (FILL)-trace silt
- black, soft to firm, moist
- intermediate plasticity
- pieces of gravel, boulders, concrete from 0.6 to 1.5 m

CLAY - trace oxidation
- brown, firm, moist
- high plasticity
- soft to firm between 2.4 to 3 m
- brown mottled light brown, silt inclusion (< 6 mm in dia.) below 3 m
- dark brown, silt inclusion (<10 mm in dia.) below 4.5 m
- grey mottled brown below 6 m
- soft below 7.3 m
- silt pocket at 8.3 m
- trace gravel below 8.8 m
- some silt to silty, light grey to grey below 9.1 m

SAMPLE TYPE	SAMPLE #
■	G43
■	G44
▥	T45
■	G46
■	G47
▥	T48
■	G49



DEPTH (m)	ELEVATION
0	233
1	232
2	231
3	230
4	229
5	228
6	227
7	226
8	225
9	224
10	

AECOM

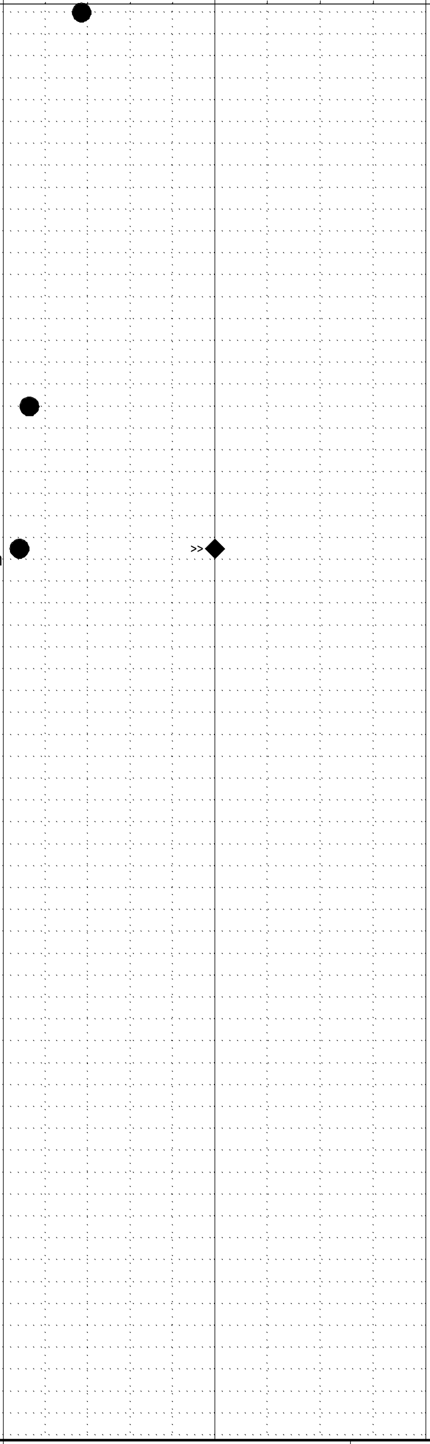
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.73 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/15/14
PROJECT ENGINEER: Faris Khalil	

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-04**
LOCATION: UTM: 14U, 5523599 m N, 630952 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.20

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE
BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND

PENETRATION TESTS: ✳ Becker ✳; ◇ Dynamic Cone ◇; ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100
UNDRAINED SHEAR STRENGTH: + Torvane +; × QU/2 ×; □ Lab Vane □; △ Pocket Pen. △; Field Vane (kPa) 50 100 150 200

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10		GRAB	G50			223
	- grey below 10.6 m					
11		SHELBY TUBE	T51			222
	- silty, silt inclusion (<40 mm in dia.) below 11.3 m					
12	- light grey to grey, some to trace gravel, low to intermediate plasticity below 12.1 m					221
	Glacial Till (SILT)- some to trace gravel, trace sand, trace clay - light grey, very dense, moist - low plasticity - loose, wet from 13.1 to 13.6 m	GRAB	G52			
13						220
		SPLIT SPOON	S53	50/ 152mm	SPT Blows: (50/152) 100 % Recovery	
14	- some sand, some boulders ,some cobbles below 14 m					219
		CORE	C1		C1 RQD: 0% C1 Recovery: 78 %	
15						218
16		CORE	C2		C2 RQD: 0% C2 Recovery: 95 %	217
17		CORE	C3A		C3A RQD: 0% C3A Recovery: 57%	216
18	LIMESTONE - fine grained - light grey, yellow staining - no foliation - R3- medium strong - closed to moderately closed, rough undulating fractures, closed to gapped, clean to filled with coarse cemented gravel, evidence of water flow (class 3), red staining, oxidized between 19 to 20.6 m	CORE	C3B		C3B RQD: 0 % C3B Recovery: 75%	215
		CORE	C4		C4 RQD: 23% C4 Recovery: 86%	
19						214
20		CORE	C5		C5 RQD: 21.6%	



LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 25.73 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 7/15/14
PROJECT ENGINEER: Faris Khalil

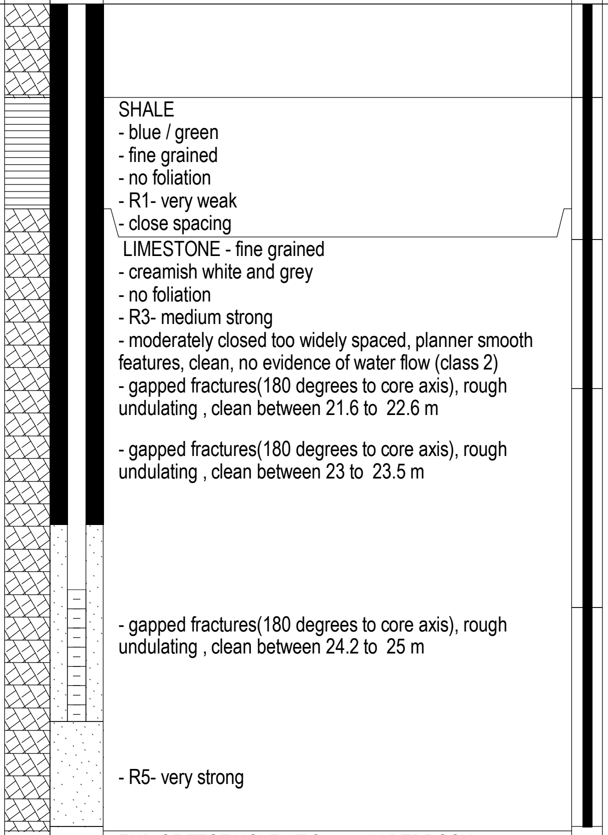
PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-04**
LOCATION: UTM: 14U, 5523599 m N, 630952 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.20

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◩ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁚ GRAVEL ▥ SLOUGH ▩ GROUT ▨ CUTTINGS ⁚ SAND

PENETRATION TESTS: ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200



DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	COMMENTS	ELEVATION
20			C5 Recovery: 71 %	213
21	SHALE - blue / green - fine grained - no foliation - R1- very weak - close spacing	C6	C6 RQD: 0% C6 Recovery: 56 %	212
22	LIMESTONE - fine grained - creamish white and grey - no foliation - R3- medium strong - moderately closed too widely spaced, planner smooth features, clean, no evidence of water flow (class 2) - gapped fractures(180 degrees to core axis), rough undulating , clean between 21.6 to 22.6 m	C7	C7 RQD: 23% C7 Recovery: 81 %	211
23	- gapped fractures(180 degrees to core axis), rough undulating , clean between 23 to 23.5 m	C8	C8 RQD: 60% C7 Recovery: 100 %	210
24	- gapped fractures(180 degrees to core axis), rough undulating , clean between 24.2 to 25 m			209
25	- R5- very strong	C9	C9 RQD: 26% C7 Recovery: 100 % qu= 114.9 MPa	208
26	END OF TEST HOLE AT 25.7 m IN BEDROCK			207
27				206
28				205
29				204
30				

NOTES:
1. Power Auger Refusal at 13.8 m in Glacial TILL.
2. HQ coring below 13.8 m.
3. Seepage observed at 3.0 m upon drilling completion.
4. Installed 25 mm diameter standpipe piezometer (SP14-04) to 23.5 m below ground surface with 0.3 m casagrande tip and flush mount at ground surface.
5. Test hole backfilled with silica sand up to 23.6 m below ground surface, bentonite up to 1 m and plugged with auger cutting to ground surface.
6. Groundwater monitoring:
- Aug. 12, 2014 at Elv. 225.2 m.
- Sep. 03, 2014 at Elv. 225.0 m.
- Sep. 19, 2014 at Elv. 225.6 m.
- Oct. 17, 2014 at Elv. 225.5 m.
- Nov. 06, 2014 at Elv. 225.4 m.
- Nov. 20, 2014 at Elv. 225.4 m.
- Dec. 06, 2014 at Elv. 225.2 m.
- Dec. 18, 2014 at Elv. 225.2 m.

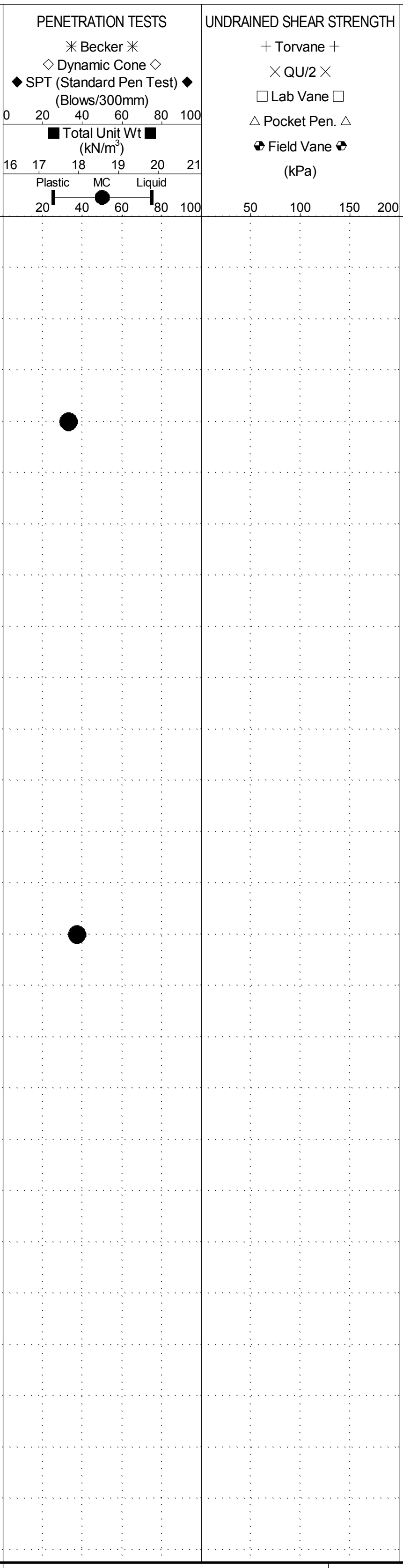
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.73 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/15/14
PROJECT ENGINEER: Faris Khalil	

AECOM

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-05**
LOCATION: UTM: 14U, 5523582 m N, 631025 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	CLAY (FILL) - silty, some sand - brown to dark brown, firm, moist - intermediate to high plasticity					
		■	G54		Gravel: 0.0%, Sand: 12.9%, Silt: 23.4%, Clay: 63.7%	
	- black below 0.9 m					
1	CLAY - trace silt, trace oxidation - dark brown, firm, moist - high plasticity	■	G55			1
	- silty, brown mottled light brown, soft from 1.5 m to 1.7 m					
	- some silt, stiff to firm below 1.7 m					
2		■	G56			2
		■	G57			
3	END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 1.4 m immediately following drilling. 2. Seepage was observed at 1.2 m and from 2.4 m to 2.7 m. 3. Test hole backfilled with auger cuttings upon drilling completion.					3
4						



LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 10/23/14
PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-06**
LOCATION: UTM: 14U, 5523587 m N, 631095 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ‖ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▌ CORE

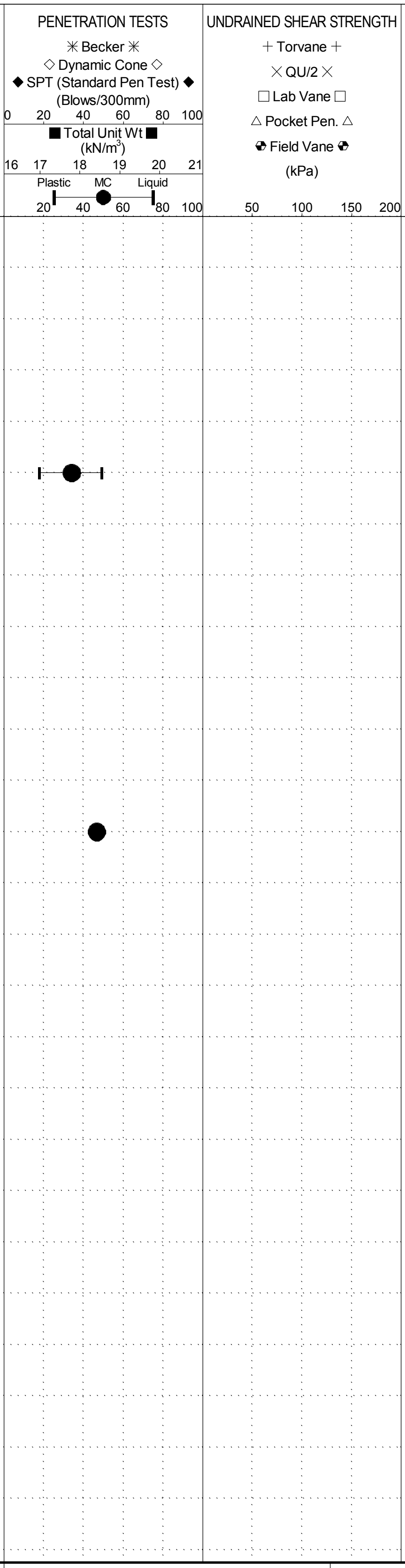
DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × ☐ Lab Vane ☐ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	DEPTH
0	▩	SAND and GRAVEL (FILL) - light brown, dry to moist							
	▩	CLAY (FILL)- silty - light grey to grey, firm, moist - high plasticity	■	G58					
	▨	CLAY- some silt - brown, firm to stiff, moist - high plasticity							
1	▨		■	G59		●			1
	▨	- silt pocket, soft to firm, trace oxidation below 1.5 m							
2	▨		■	G60					2
	▨	- silty, soft below 2.4 m	■	G61		●			
3		END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 2.9 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Water level measured at 1.8 m below ground surface immediately following drilling. 4. Test hole backfilled with auger cuttings upon drilling completion.							3
4									

AECOM	LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.05 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/23/14
	PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-07**
LOCATION: UTM: 14U, 5523614 m N, 631190 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	☒	GRAVEL and SAND (FILL) - some clay					
	☒	CLAY (FILL) - silty - light grey and grey-black, firm, moist - intermediate plasticity					
	▨	CLAY AND SILT- organic, silty, some sand - black, firm, moist - intermediate plasticity	■	G62		Gravel: 0.0%, Sand: 19.6%, Silt: 36.1%, Clay: 44.2%, AASHTO classification (A-7-6)	
1	▥	SILT - clayey - light grey, firm, moist, - low plasticity					1
	▨	CLAY - some silt - grey, firm, moist, - high plasticity	■	G63			
	▨	- trace silt inclusions (< 6 mm in dia.), brown, firm to stiff below 1.5 m					
	▨		■	G64			
2	▨						2
	▨		■	G65			
3	▨						3
		END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 3.05 m immediately following drilling. 2. No sloughing was observed upon drilling completion. 3. Seepage was observed at 0.9 m and 1.5 m below ground surface. 4. Test hole backfilled with auger cuttings upon drilling completion.					
4							



AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/23/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

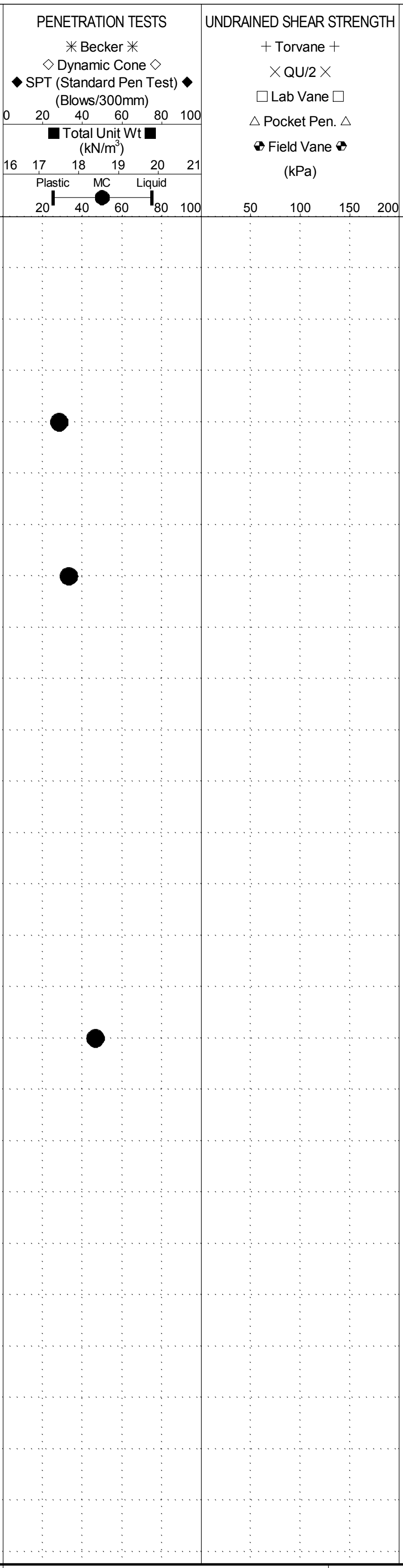
PROJECT: Waverley Underpass | CLIENT: City of Winnipeg | TESTHOLE NO: **TH14-08**

LOCATION: UTM: 14U, 5523551 m N, 630836 m E | PROJECT NO.: 60321148

CONTRACTOR: Maple Leaf Drilling Ltd. | METHOD: 125 mm SSA | ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▨	CLAY (FILL) - silty, sandy - grey, moist					
			■	G66		Gravel: 0.0%, Sand: 23.4%, Silt: 27.5%, Clay: 49.1%	
1			■	G67			1
	▧	CLAY - silty - light grey, soft, moist - low to intermediate plasticity - brown, firm to stiff, moist, high plasticity below 1.5 m					
			■	G68			
2		- silty from 2 m to 2.2 m - trace oxidation below 2 m - silt inclusion (<12 mm in dia.) below 2.1 m					2
			■	G69			
3		END OF TEST HOLE AT 3.05 m IN CLAY.					3
4							



NOTES:
1. Hole open to 3.05 m immediately following drilling.
2. No sloughing was observed upon drilling completion.
3. Seepage observed at 2.4 during drilling.
4. Water level measured at 2.9 m immediately following drilling.
5. Test hole backfilled with auger cuttings upon drilling completion.

AECOM

LOGGED BY: Saba Ibrahim | COMPLETION DEPTH: 3.05 m

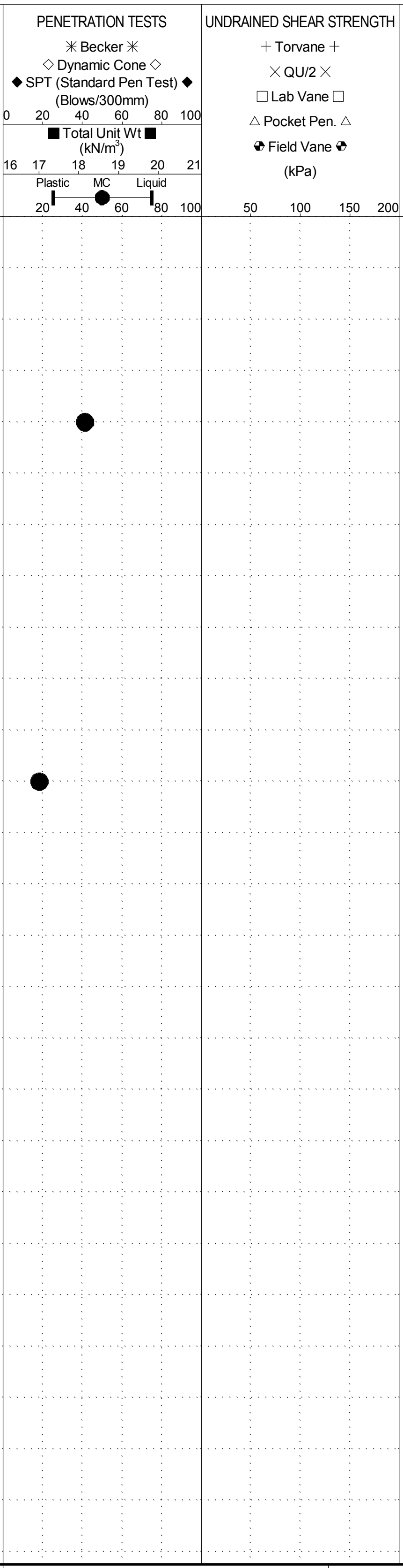
REVIEWED BY: Zeyad Shukri | COMPLETION DATE: 10/23/14

PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-09**
LOCATION: UTM: 14U, 5523533 m N, 630753 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▨	CLAY (FILL) - silty, some organic - black and light grey, firm, moist					
			■	G70			
1		- some sand below 0.9 m					1
			■	G71			
	▨	CLAY - brown mottled grey, firm to stiff, moist - high plasticity - silty, soft from 1.7 m to 1.9 m	■	G72			
2							2
		- trace oxidation below 2.13 m - silt inclusion (< 6 mm in dia.) from 2.1 m 2.3 m					
			■	G73			
3							3
		END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. No sloughing was observedupon drilling completion. 2. Seepage was observed at 1.2 m and 1.52 m during drilling. 3. Test hole backfilled with auger cuttings upon drilling completion.					
4							



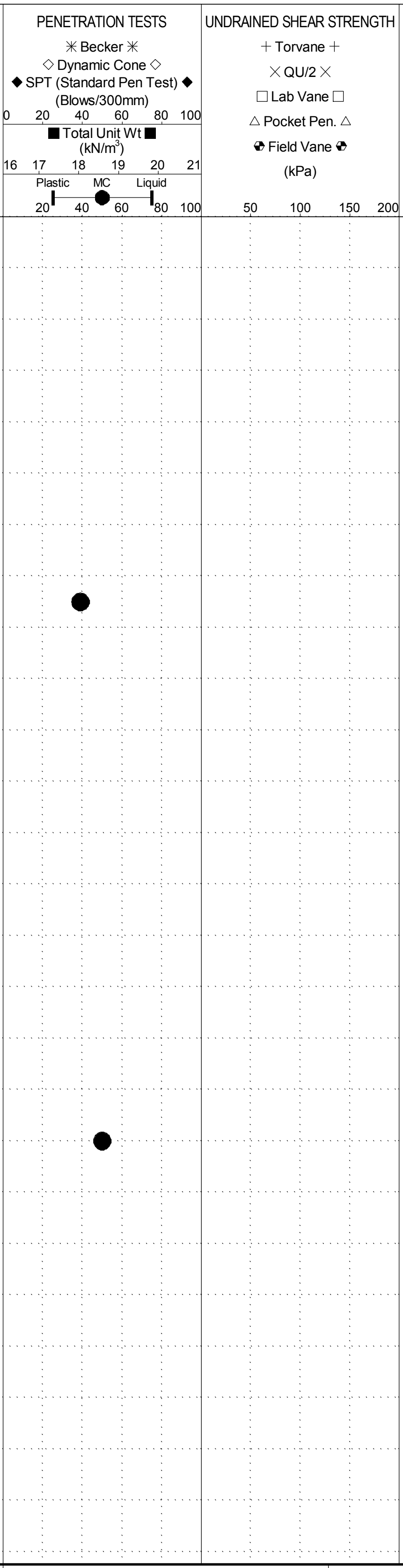
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/23/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-10**
LOCATION: UTM: 14U, 5523516 m N, 630610 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▨	CLAY (FILL) - silty - black to light grey, firm, moist - low to intermediate plasticity	■	G74			
1	▨	CLAY - organic, silty to some silt, some sand - black, soft to firm, moist to wet, - low plasticity - grey, firm below 1.4 m	■	G75		Gravel: 0.0%, Sand: 14.1%, Silt: 33.5%, Clay: 52.4%	1
	▥	SILT - some clay - brown, soft, moist to wet - low plasticity	■	G76			
2	▨	CLAY- some to trace silt - grey mottled brown, firm to stiff, moist, - high plasticity - silt inclusions (<6 mm in dia.) below 2.4 m	■	G77			2
3		END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 3.05 m immediately following drilling. 2. Sloughing was observed at 1.8 m. 3. Seepage was observed at 1.1 m and below 1.5 m. 4. Test hole backfilled with auger cuttings upon drilling completion.					3
4							



AECOM

LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 10/23/14
PROJECT ENGINEER: Faris Khalil

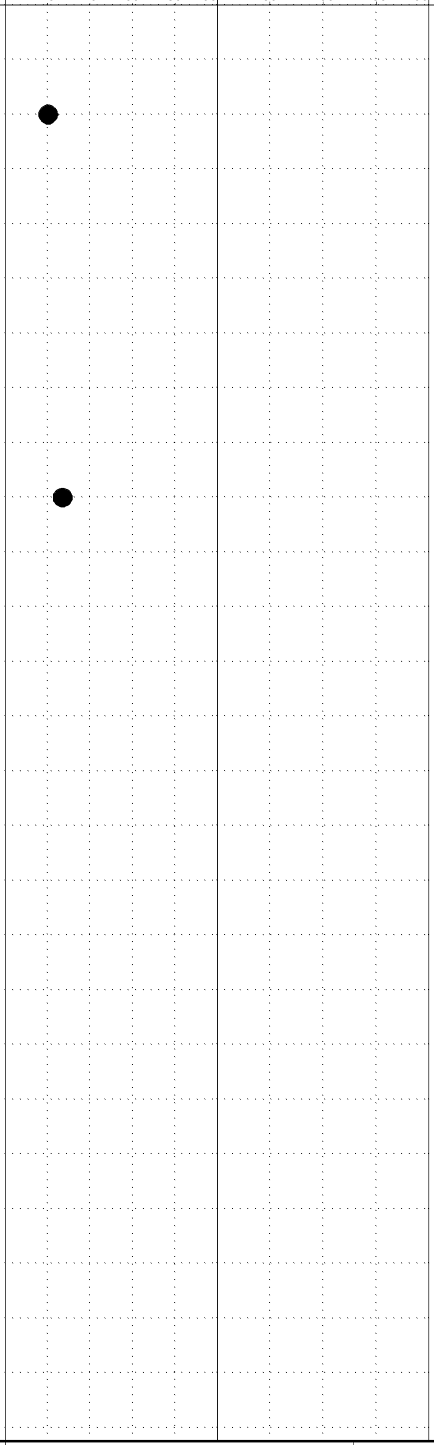
PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-11**
LOCATION: UTM: 14U, 5523574 m N, 630822 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
0	TOPSOIL				
	SAND and GRAVEL (FILL) - light brown, moist to wet	■	G78		
	CLAY (FILL) - organic, sandy, trace wood - black, firm, moist to wet	■	G79		
1	SILT - some clay - light grey, soft, moist - low plasticity	■	G80		
	CLAY - trace silt - brown mottled grey, firm to stiff, moist, - high plasticity				
2	- trace silt inclusions (< 12 mm in dia.) below 2.3 m	■	G81		
3	END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 3.05 m immediately following drilling. 2. No sloughing was observed upon drilling completion. 3. Seepage was observed at 1.1 m. 4. Test hole backfilled with auger cuttings upon drilling completion.				
4					

PENETRATION TESTS
✳ Becker ✳
◇ Dynamic Cone ◇
◆ SPT (Standard Pen Test) ◆
(Blows/300mm)
0 20 40 60 80 100
■ Total Unit Wt ■
(kN/m^3)
16 17 18 19 20 21
Plastic MC Liquid
20 40 60 80 100

UNDRAINED SHEAR STRENGTH
+ Torvane +
× QU/2 ×
□ Lab Vane □
△ Pocket Pen. △
● Field Vane ●
(kPa)
50 100 150 200

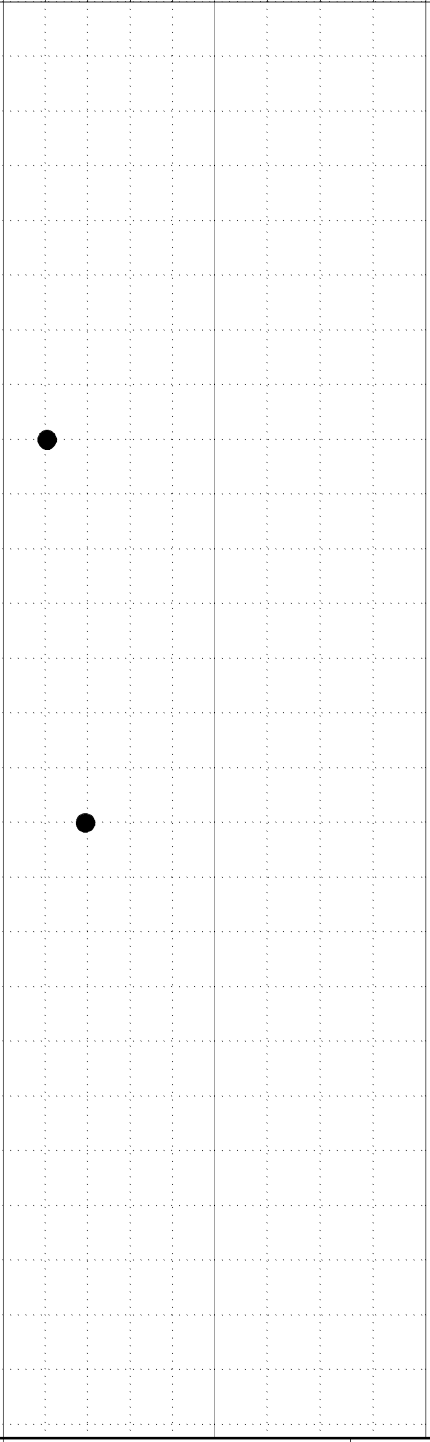


LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 3.05 m
COMPLETION DATE: 10/23/14

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-12**
LOCATION: UTM: 14U, 5523563 m N, 630774 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◪ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × ☐ Lab Vane ☐ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	DEPTH
0		GRAVEL (FILL) - light brown, moist							
		CLAY (FILL) - some gravel, trace to some silt, trace oxidation - grey, firm, moist	■	G82					
1		CLAY - organic, some silt, trace gravel, trace oxidation - black, firm, moist, - pieces of wood from 0.9 m to 1.2 m	■	G83					1
		SILT - light brown, soft, moist, - low plasticity	■	G84					
2		CLAY - trace to some silt - brown mottled grey, soft to stiff, moist, - high plasticity - silt pocket from 1.8 m to 2 m - trace oxidation below 2.5 m - silty, soft to firm below 2.75 m	■	G85					2
3		END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 1.74 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cuttings upon completion.							3
4									



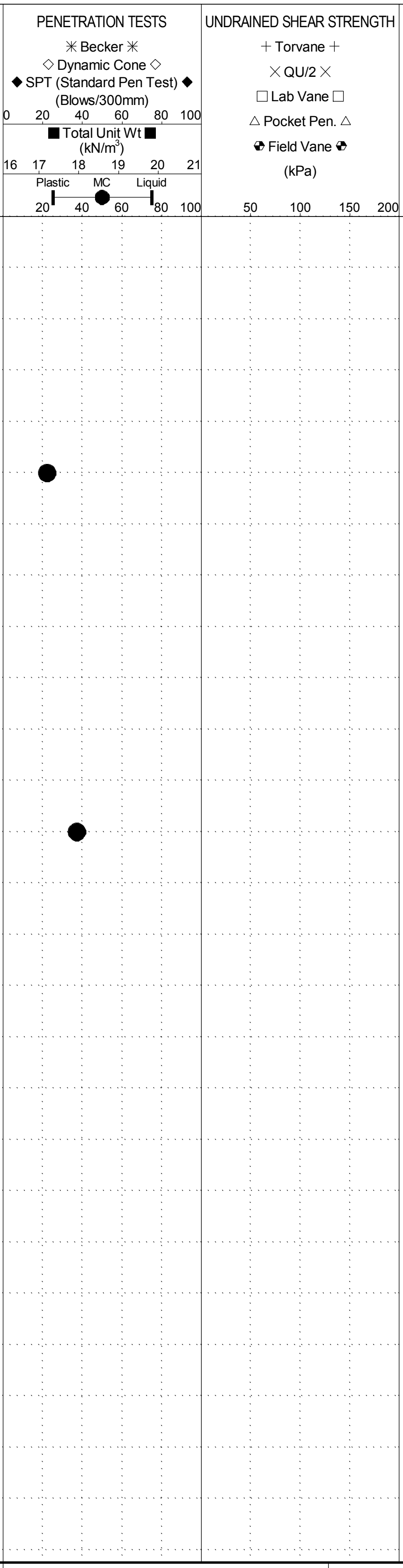
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/23/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

AECOM

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-13**
LOCATION: UTM: 14U, 5523544 m N, 630678 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▐ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
0	TOPSOIL				
	SAND and GRAVEL (FILL) - light brown, moist				
	CLAY (FILL) - silty, trace organics - black to brown, firm, moist, - intermediate to low plasticity	■	G86		
1					
		■	G87		
	CLAY - trace silt - brown to dark brown, firm to stiff, moist, - high plasticity	■	G88		
2	- silty, soft to firm, trace oxidation from 2 m to 2.3 m				
	- silt inclusion (< 6 mm in dia.) below 2.3 m	■	G89		
3	END OF TEST HOLE AT 3.05 m IN CLAY. NOTES: 1. Hole open to 2.90 m immediately after drilling. 2. No seepage observed upon drilling completion. 3. Test hole backfilled with auger cuttings upon drilling completion.				
4					

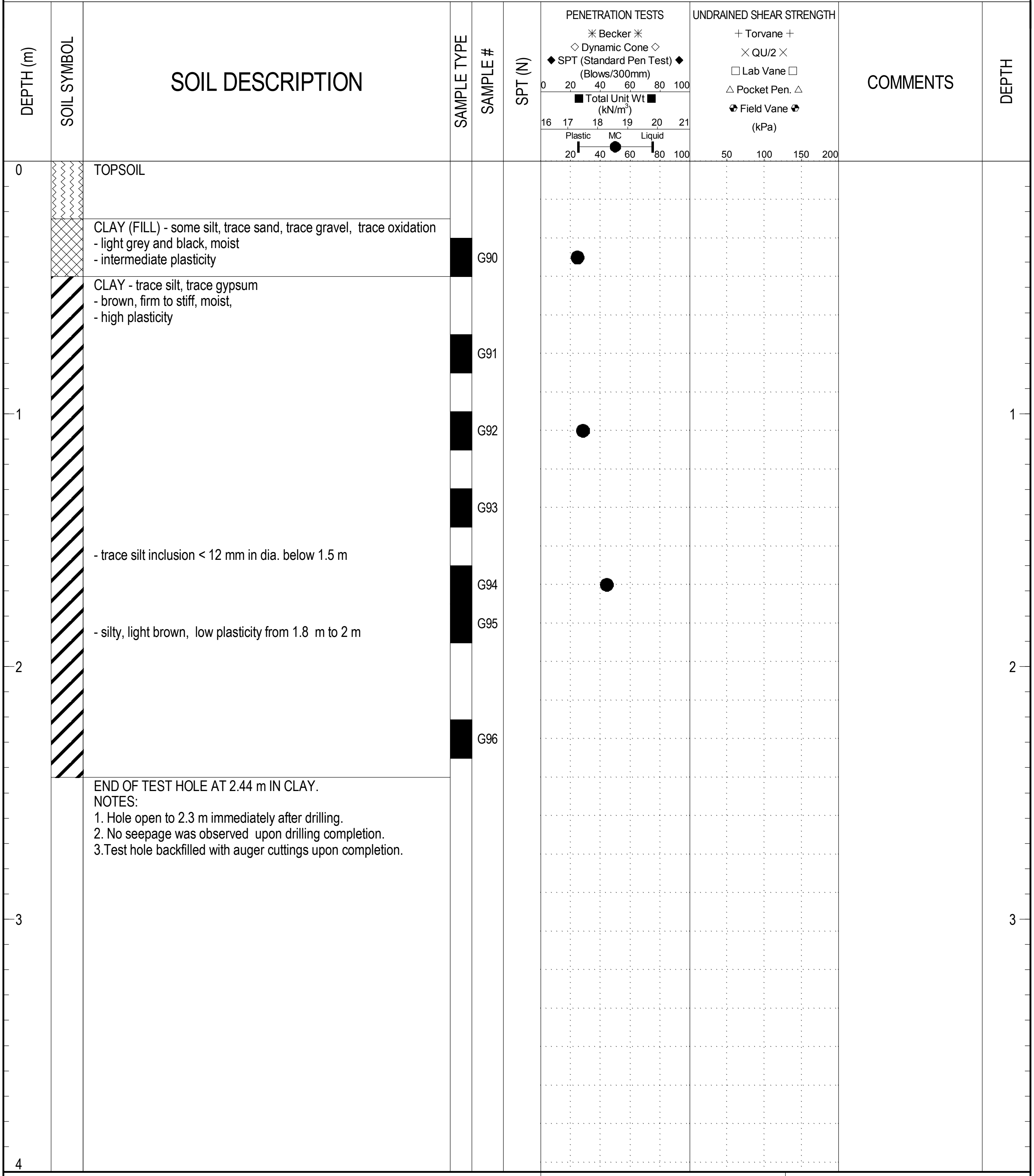


LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/23/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

AECOM

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-14**
LOCATION: UTM: 14U, 5523606 m N, 630544 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▱ NO RECOVERY ▌ CORE



TOPSOIL

CLAY (FILL) - some silt, trace sand, trace gravel, trace oxidation
- light grey and black, moist
- intermediate plasticity

CLAY - trace silt, trace gypsum
- brown, firm to stiff, moist,
- high plasticity

- trace silt inclusion < 12 mm in dia. below 1.5 m

- silty, light brown, low plasticity from 1.8 m to 2 m

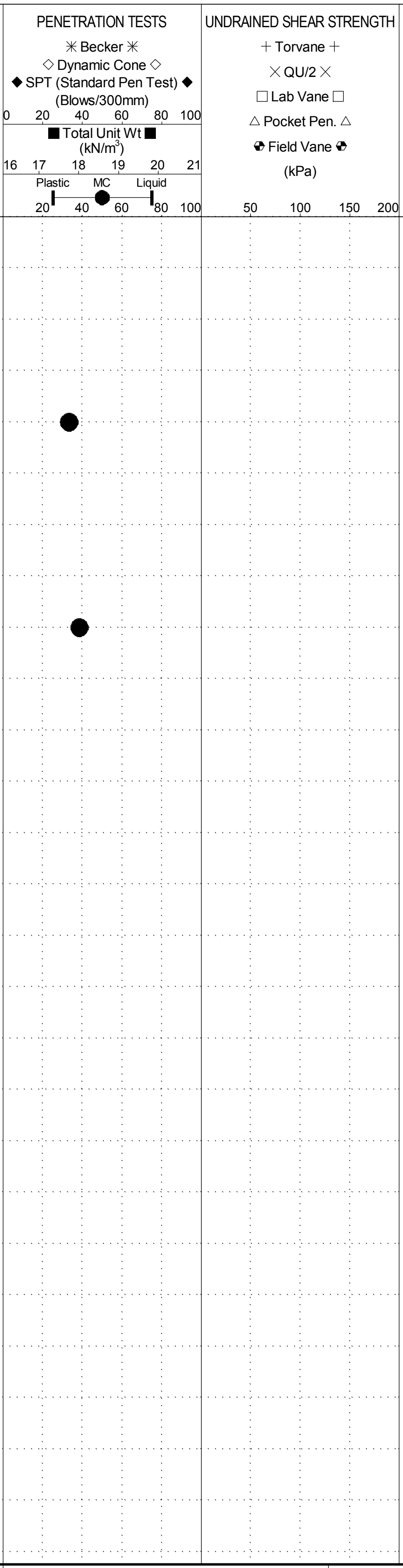
END OF TEST HOLE AT 2.44 m IN CLAY.
NOTES:
1. Hole open to 2.3 m immediately after drilling.
2. No seepage was observed upon drilling completion.
3.Test hole backfilled with auger cuttings upon completion.

AECOM	LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
	PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-15**
LOCATION: UTM: 14U, 5523571 m N, 630387 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - some silt, trace gravel - black and grey, soft to firm, moist, - intermediate to high plasticity	■	G97			
		CLAY - trace silt - grey, firm, moist, - high plasticity	■	G98			
			■	G99			
1							1
			■	G100			
		- silty, low plasticity, trace oxidation from 1.5 m to 1.7 m	■	G101			
			■	G102			
2							2
			■	G103			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.3 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



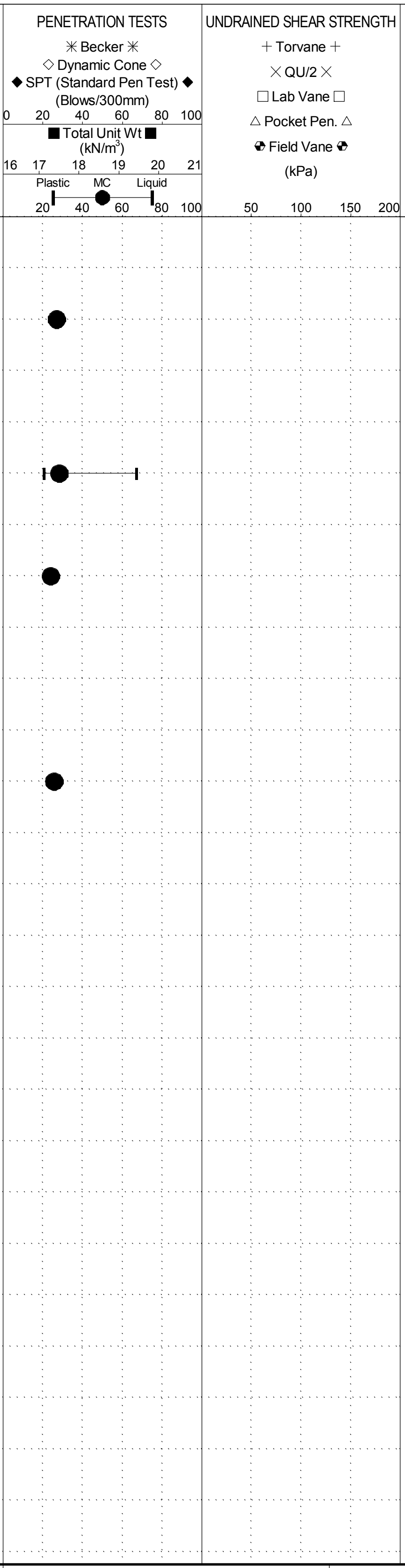
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-16**
LOCATION: UTM: 14U, 5523647 m N, 630668 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY - silty, trace sand - brown, firm, moist, - high plasticity	■	G104			
			■	G105		Gravel: 0.0%, Sand: 5.5%, Silt: 29.0%, Clay: 65.5% , AASHTO classification (A-7-6)	
1			■	G106			1
			■	G107			
		- light brown, soft below 1.5 m	■	G108			
2			■	G109			2
			■	G110			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



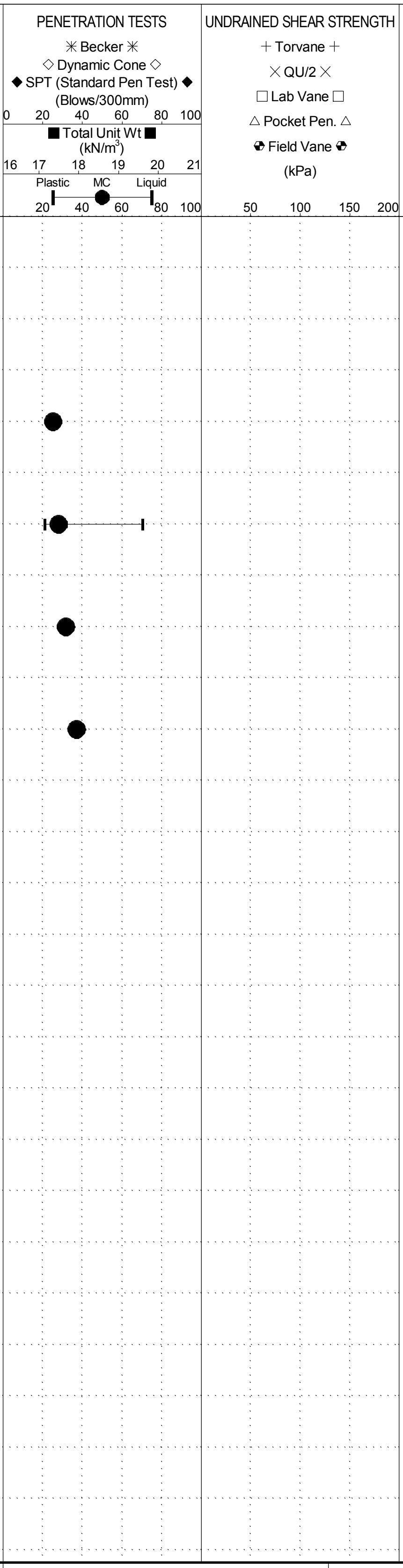
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-17**
LOCATION: UTM: 14U, 5523683 m N, 630802 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▐ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - trace sand, trace gravel, trace silt - black to grey, firm, moist, - intermediate to high plasticity	■	G111			
		CLAY - silty, trace sand - grey, firm to stiff, moist - high plasticity	■	G112			
			■	G113		Gravel: 0.0%, Sand: 5.1%, Silt: 24.4%, Clay: 70.5%, AASHTO classification (A-7-6)	
1							1
			■	G114			
		- grey mottled brown from 1.5 m to 1.8 m	■	G115			
		- brown, trace oxidation from 1.8 m to 2.2 m	■	G116			
2							2
		- some silt, grey below 2.2 m	■	G117			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



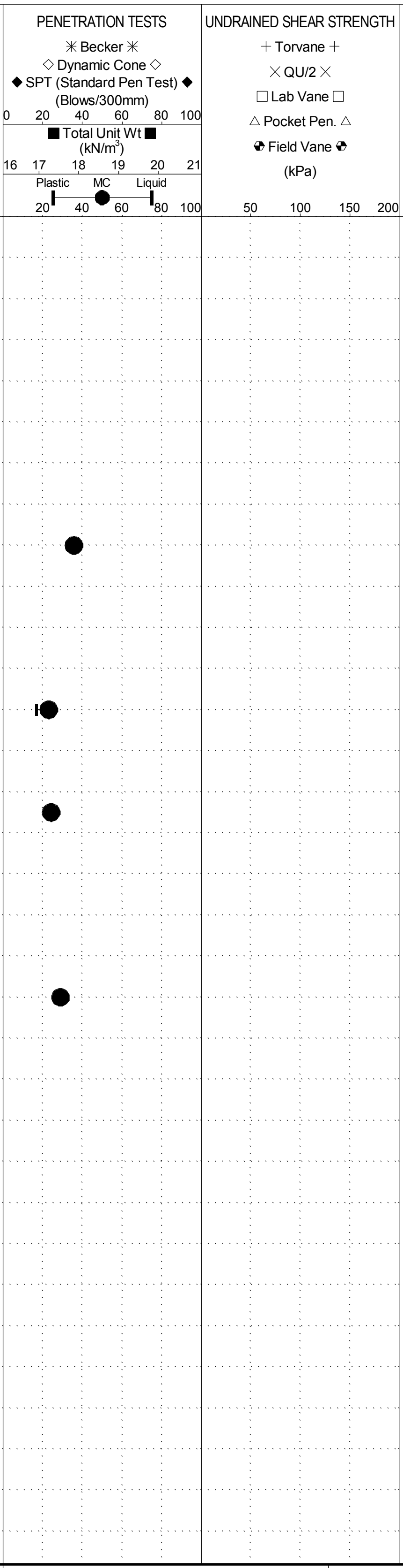
AECOM

LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 2.44 m
COMPLETION DATE: 10/24/14

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-18**
LOCATION: UTM: 14U, 5523429 m N, 630866 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◺ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - some gravel, some silt, trace sand - grey, firm to stiff, moist, - low to intermediate plasticity	■ ■ ■	G118 G119 G120			
1							1
		SILT - clayey, some sand - light brown, soft, moist, - low plasticity	■ ■	G121 G122		Gravel: 0.0%, Sand: 17.0%, Silt: 60.9%, Clay: 22.1%, AASHTO Classification (A-4)	
2							2
		CLAY- trace to some silt - grey mottled brown, firm, moist to wet, - high to intermediate plasticity	■	G123			
		SILT - clayey, some sand - light brown, soft, moist, - low plasticity	■	G124			
3							3
		CLAY- trace silt - grey mottled brown, firm to stiff, moist to wet, - high plasticity - silty below 3.8 m	■ ■	G125 G126			
4		END OF TEST HOLE AT 3.96 m IN CLAY. NOTES: 1. Hole open to 2.1 m upon drilling completion. 2. Seepage and sloughing were observed below 3 m. 3. Test hole backfilled with auger cuttings upon drilling completion.					4
5							



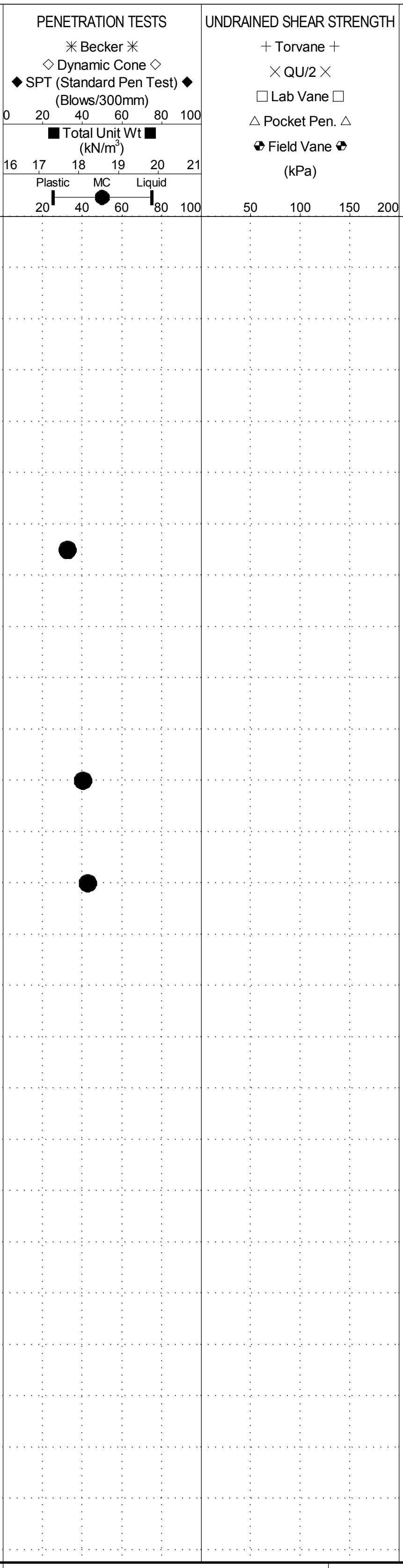
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.96 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-19**
LOCATION: UTM: 14U, 5523343 m N, 630875 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◺ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - trace gravel, trace silt, trace sand - black, firm to stiff, moist, - high plasticity	■	G127			
			■	G128			
1		CLAY - trace silt - brown, firm, moist, - high plasticity	■	G129			1
			■	G130			
		- grey mottled brown, silt inclusion < 6 mm in dia. below 1.5 m - silty to some silt, low to intermediate plasticity from 1.5m to 1.7m	■	G131			
2			■	G132			2
			■	G133			
			■	G134			
3		END OF TEST HOLE AT 3.0 m IN CLAY. NOTES: 1. Hole open to 2.7 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					3
4							

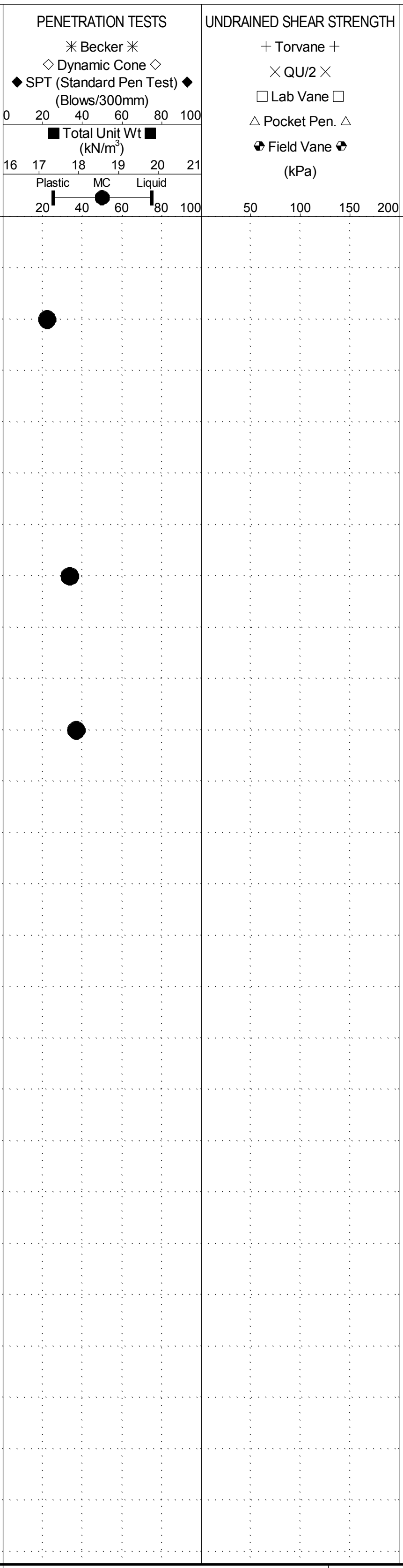


LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 3.05 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-20**
LOCATION: UTM: 14U, 5523235 m N, 630888 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	TOPSOIL					
	CLAY - some silt - grey, soft to firm, moist, - intermediate to high plasticity	■	G135			
	- trace silt below 0.5m	■	G136			
1		■	G137			1
		■	G138			
	- brown, trace silt inclusion < 6 mm in dia. below 1.5m - silty, light brown below 1.7 m	■	G139			
2		■	G140			2
		■	G141			
	END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3						3
4						



AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
PROJECT ENGINEER: Faris Khalil	

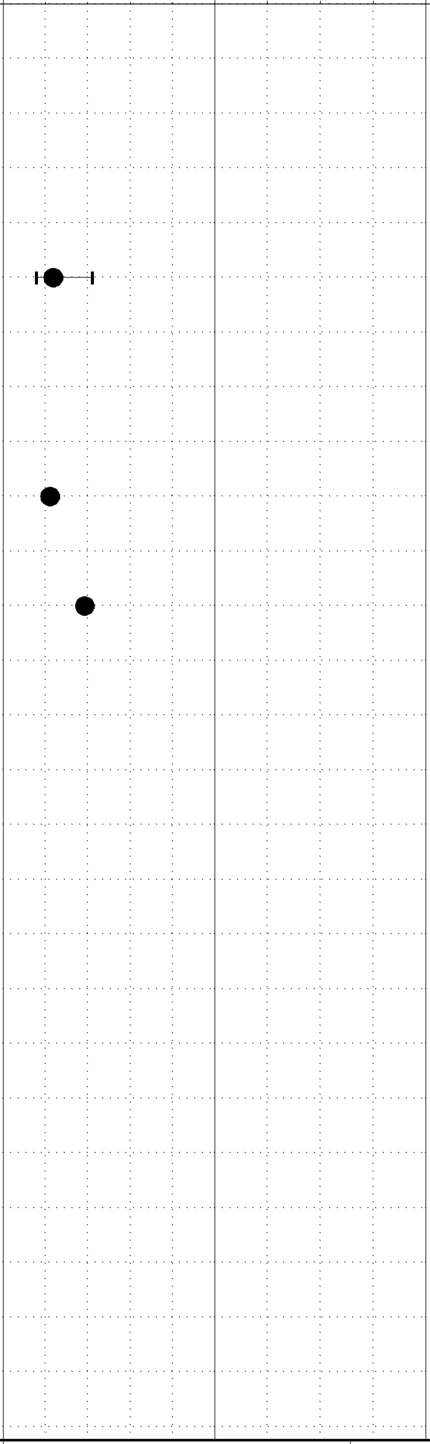
PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-21**
LOCATION: UTM: 14U, 5523227 m N, 631020 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▌ CORE

PENETRATION TESTS: ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ Field Vane (kPa) 50 100 150 200

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - some silt, trace organic - black, firm to stiff, moist - intermediate to high plasticity	■	G142			
		CLAY and SILT - some sand - light brown, soft to firm, moist, - intermediate plasticity	■	G143		Gravel: 0.0%, Sand: 13.3%, Silt: 42.8%, Clay: 43.9%, AASHTO Classification (A-7-6)	
1			■	G144			1
			■	G145			
		CLAY - trace silt - brown, moist - high plasticity - trace silt inclusion < 12 mm in dia., trace gravel below 1.7m	■	G146			
2			■	G147			2
			■	G148			
		END OF TEST HOLE AT 2.44 m IN CLAY. 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



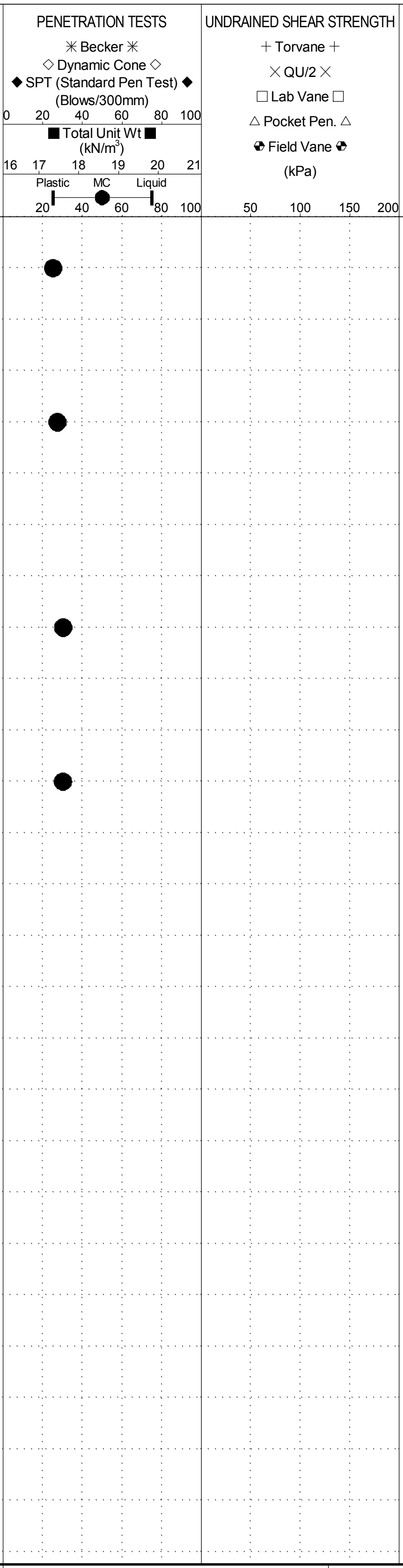
AECOM

LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 2.44 m
COMPLETION DATE: 10/24/14

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-22**
LOCATION: UTM: 14U, 5523219 m N, 631078 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◺ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL)- some silt, trace gravel, trace oxidation - black to brown, firm to stiff, moist, - high plasticity	■	G149			
		CLAY - silty, trace sand - brown, firm, moist, - intermediate plasticity - silt pocket between 0.3 m and 1.2 m	■	G150		Gravel: 0.0%, Sand: 1.4%, Silt: 33.1%, Clay: 65.5%	
			■	G151			
1							1
		- silt inclusion < 12 mm in dia. below 1.2 m	■	G152			
		- trace oxidation from 1.5 m to 2.1 m					
		SILT - some clay to clayey - light brown, soft to firm, moist, - low plasticity	■	G153			
2			■	G154			2
		- very soft below 2.1 m	■	G155			
		END OF TEST HOLE AT 2.44 m IN SILT. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



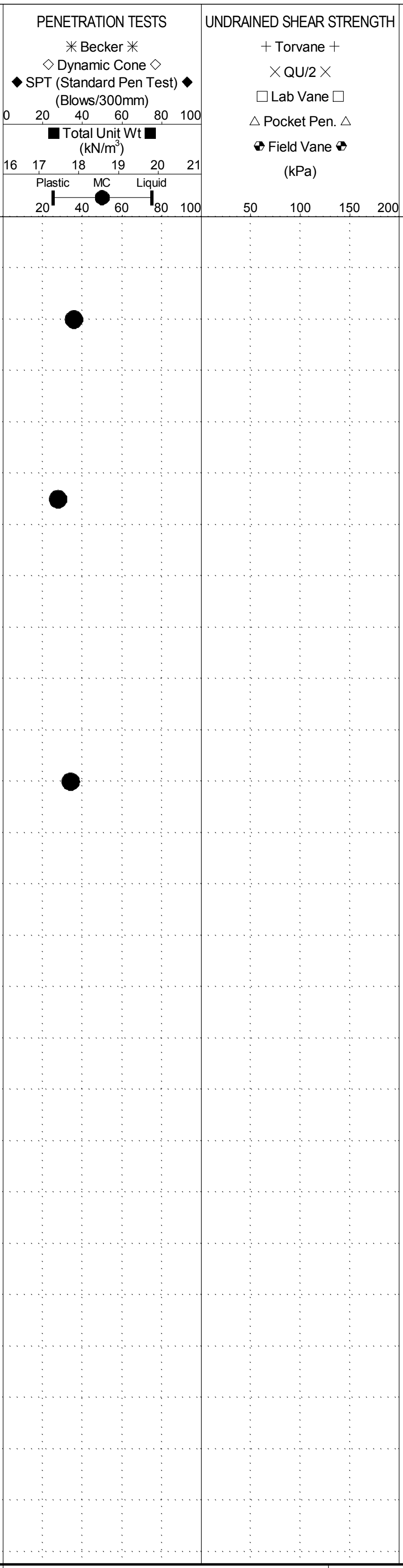
AECOM

LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 2.44 m
COMPLETION DATE: 10/24/14

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-23**
LOCATION: UTM: 14U, 5523200 m N, 631272 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▱ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY - trace silt - grey, firm, moist, - high plasticity	■	G156			
			■	G157			
		- trace gravel, dark grey from 0.7 m to 0.9 m	■	G158			
1		SILT- clayey - light brown, soft, moist - low plasticity	■	G159			1
		CLAY - trace silt - brown mottled grey, firm, moist, - high plasticity - silt pocket below 1.75 m - silt inclusion < 12 mm in dia. below 1.8 m	■	G160			
2			■	G161			2
			■	G162			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



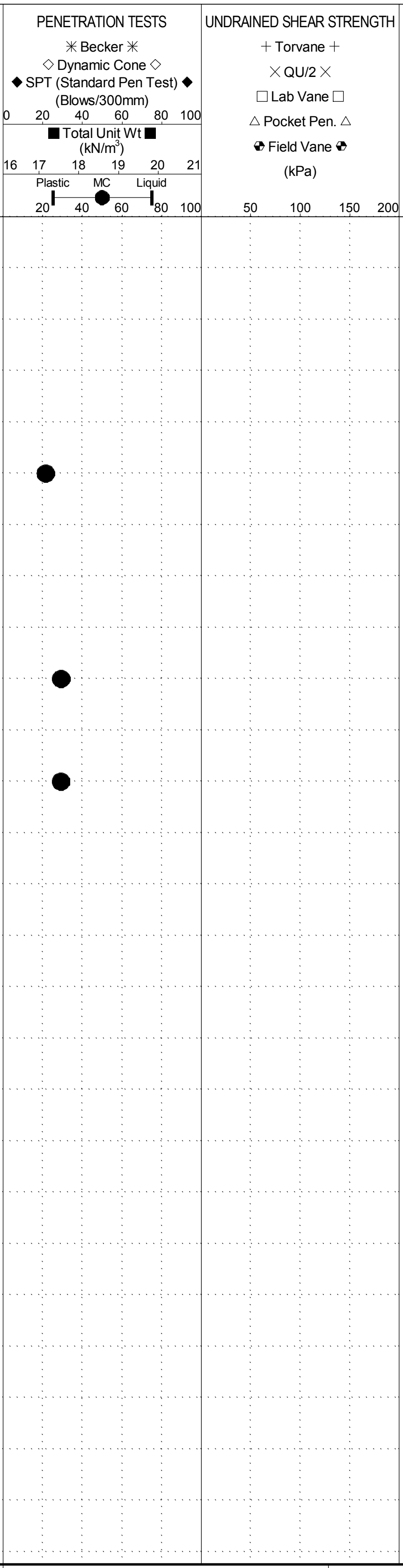
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/24/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-24**
LOCATION: UTM: 14U, 5523700 m N, 631092 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
0	TOPSOIL				
	CLAY (FILL) - some silt, trace sand - black to grey, moist - intermediate plasticity	■	G163		
	CLAY and SILT - trace sand - brown, firm to stiff, moist, - intermediate plasticity	■	G164		Gravel: 0.0%, Sand: 7.3%, Silt: 45.2%, Clay: 47.5%
1		■	G165		
		■	G166		
	CLAY - silty to some silt, trace oxidation - brown to light brown, soft to firm, moist, - high plasticity	■	G167		
2		■	G168		
		■	G169		
	END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. Seepage was observed below 2.3 m upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.				
3					
4					

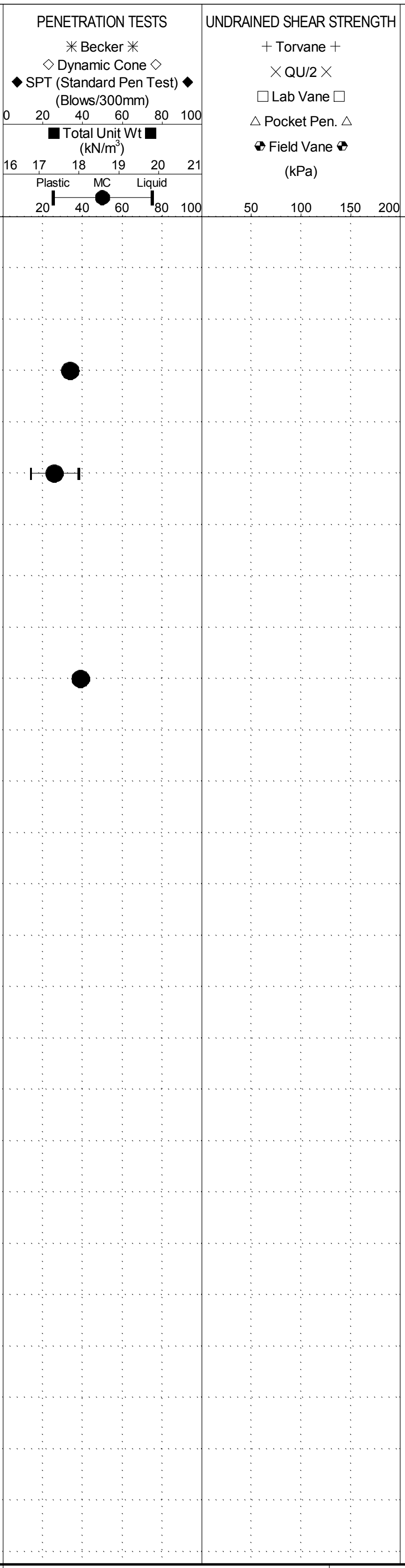


LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-25**
LOCATION: UTM: 14U, 5523746 m N, 631270 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◺ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - some silt, trace sand - black to dark grey, firm, moist, - intermediate to high plasticity	■	G170			
		SILT - clayey, trace sand - light brown, moist, - intermediate plasticity	■	G171		Gravel: 0.0%, Sand: 8.1%, Silt: 60.0%, Clay: 31.9%, AASHTO Classification (A-6)	
1		- brown from 0.9 m to 1.5 m	■	G172			1
		- silt pocket, silt inclusion < 6 mm in dia. below 1.2 m	■	G173			
		CLAY - trace silt - brown, firm to stiff, moist, - intermediate to high plasticity - trace oxidation at 1.7 m	■	G174			
2			■	G175			2
			■	G176			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

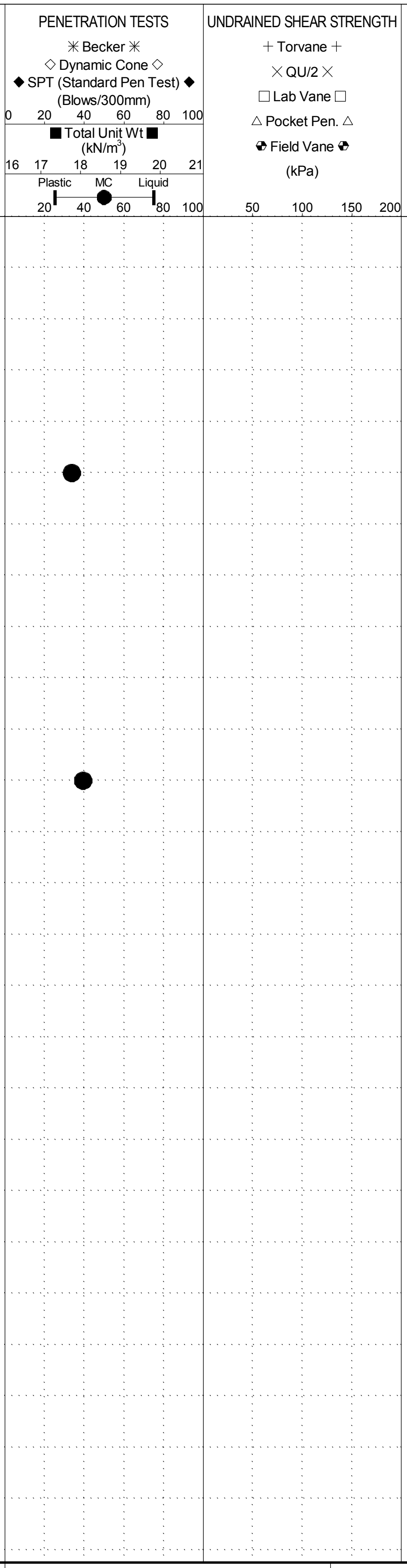
AECOM

LOG OF TEST HOLE WAVERLEY UP - PHASE II - TEST HOLE LOGS - WITH LAB DATA -REVISION 1.GPJ UMA WINN.GDT 1/12/15

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-26**
LOCATION: UTM: 14U, 5523720 m N, 630895 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ⧄ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0		TOPSOIL					
		CLAY (FILL) - some silt, trace sand - dark to light grey, firm, moist, - high plasticity	■	G177			
		CLAY - trace silt - grey, firm to stiff, moist, - high plasticity					
			■	G178			
1							1
			■	G179			
		- grey mottled brown below 1.5 m					
			■	G180			
			■	G181			
2							2
		- silty, trace oxidation below 2.1m					
			■	G182			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							



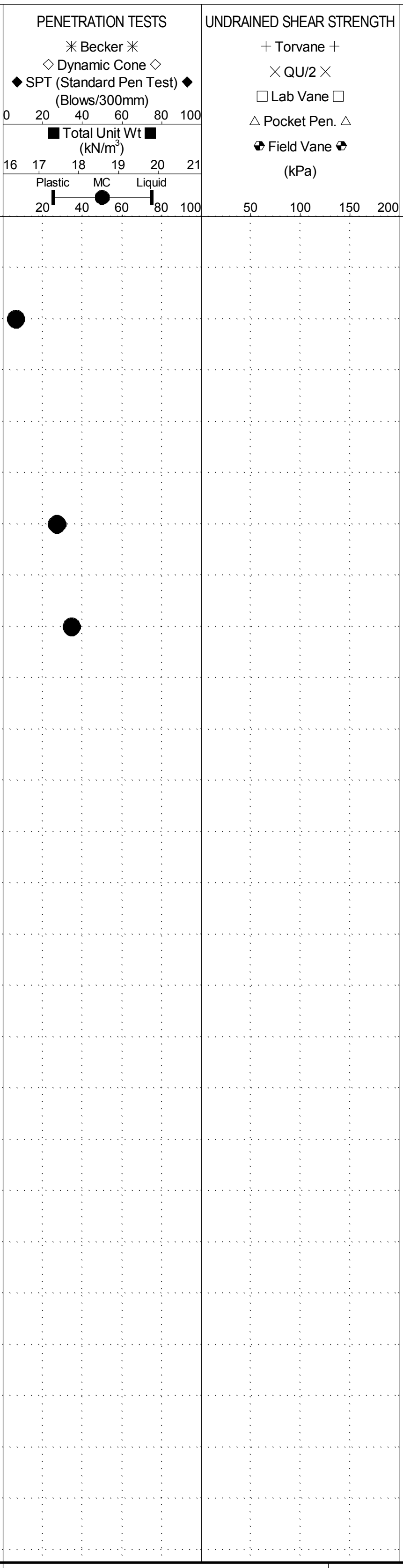
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-27**
LOCATION: UTM: 14U, 5523208 m N, 630727 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▨	GRAVEL and SAND (FILL) - some clay, some silt - light brown, dry to moist					
			■	G183			
	▧	CLAY - silty, trace sand - grey, firm, moist, - high plasticity					
			■	G184		(G184): Gravel: 0.0%, Sand: 6.8%, Silt: 27.7%, Clay: 65.5%	
1							1
			■	G185			
		- light brown, trace oxidation from 1.8 m to 2 m	■	G186			
2		- grey mottled brown below 2 m					2
			■	G187			
		END OF TEST HOLE AT 2.44 m IN CLAY. NOTES: 1. Hole open to 2.4 m immediately following drilling. 2. No seepage was observed upon drilling completion. 3. Test hole backfilled with auger cutting upon drilling completion.					
3							3
4							

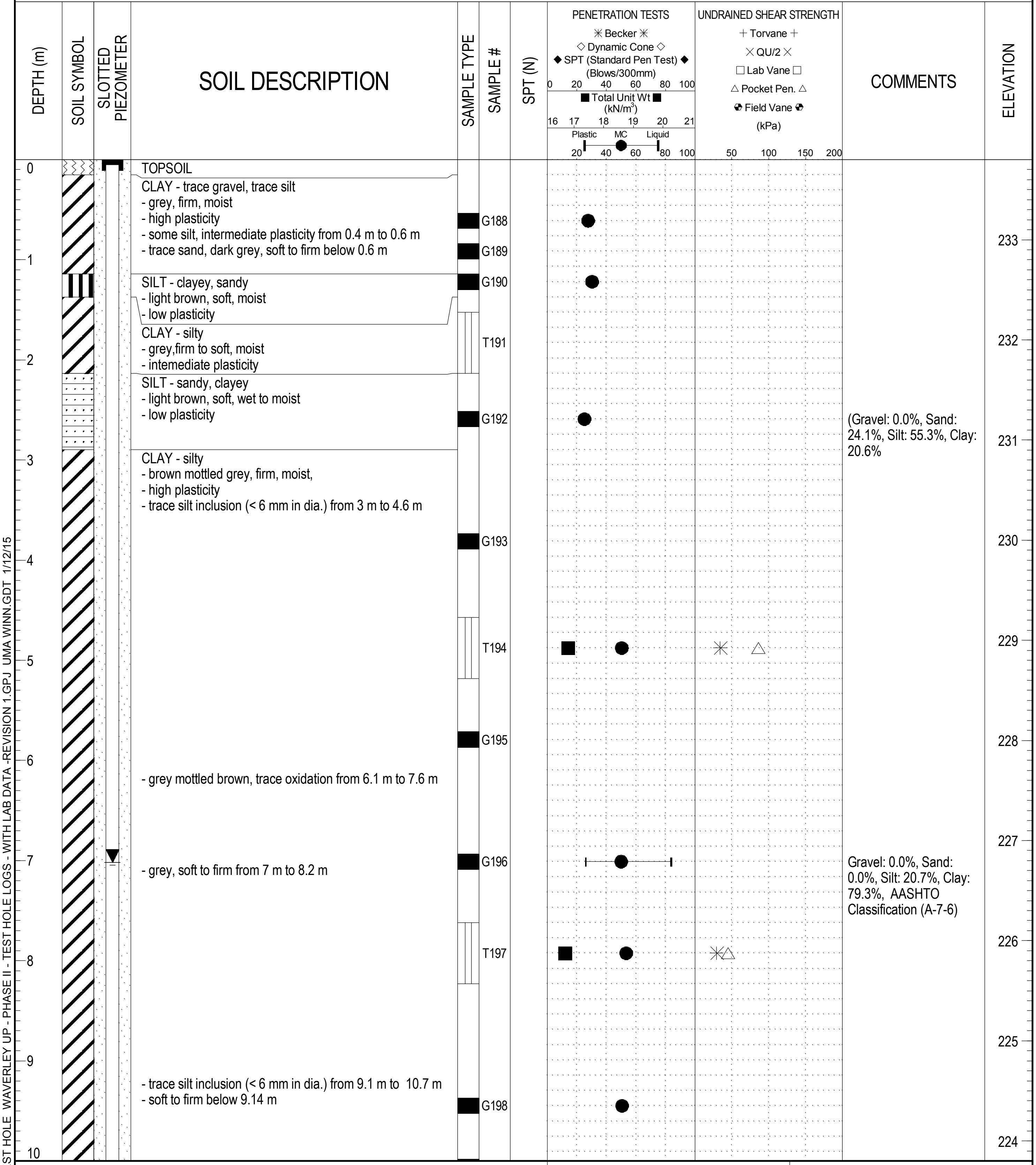


AECOM	LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 2.44 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/26/14
	PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-28**
LOCATION: UTM: 14U, 5523511 m N, 630871 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 233.80

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



TOPSOIL

CLAY - trace gravel, trace silt
- grey, firm, moist
- high plasticity
- some silt, intermediate plasticity from 0.4 m to 0.6 m
- trace sand, dark grey, soft to firm below 0.6 m

SILT - clayey, sandy
- light brown, soft, moist
- low plasticity

CLAY - silty
- grey,firm to soft, moist
- intemediate plasticity

SILT - sandy, clayey
- light brown, soft, wet to moist
- low plasticity

(Gravel: 0.0%, Sand: 24.1%, Silt: 55.3%, Clay: 20.6%

CLAY - silty
- brown mottled grey, firm, moist,
- high plasticity
- trace silt inclusion (< 6 mm in dia.) from 3 m to 4.6 m
- grey mottled brown, trace oxidation from 6.1 m to 7.6 m
- grey, soft to firm from 7 m to 8.2 m

Gravel: 0.0%, Sand: 0.0%, Silt: 20.7%, Clay: 79.3%, AASHTO Classification (A-7-6)

- trace silt inclusion (< 6 mm in dia.) from 9.1 m to 10.7 m
- soft to firm below 9.14 m

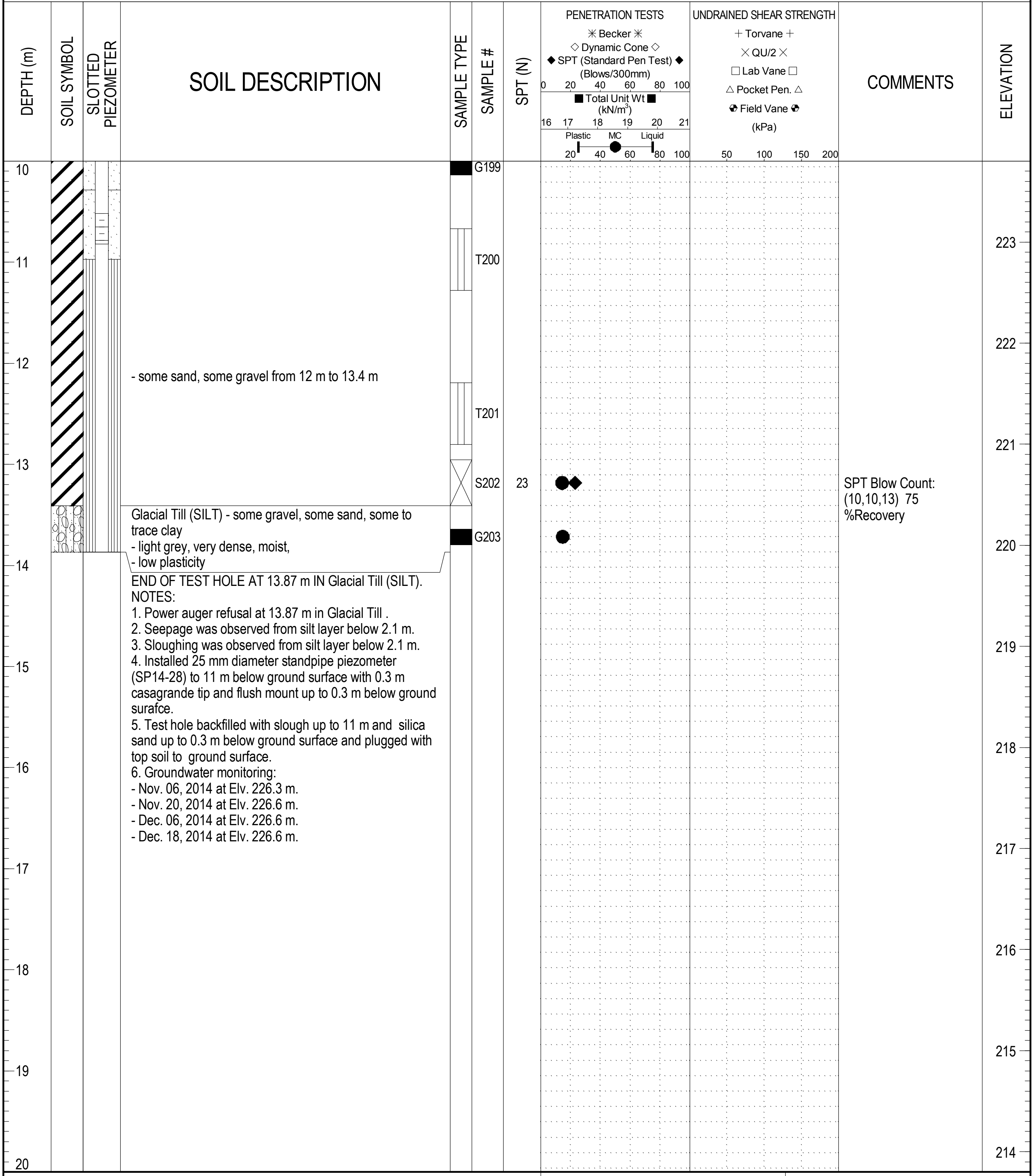
LOG OF TEST HOLE WAVERLEY UP - PHASE II - TEST HOLE LOGS - WITH LAB DATA -REVISION 1.GPJ UMA WINN.GDT 1/12/15

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 13.87 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil	

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-28**
LOCATION: UTM: 14U, 5523511 m N, 630871 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m): 233.80

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



- some sand, some gravel from 12 m to 13.4 m

Glacial Till (SILT) - some gravel, some sand, some to trace clay
- light grey, very dense, moist,
- low plasticity

END OF TEST HOLE AT 13.87 m IN Glacial Till (SILT).
NOTES:
1. Power auger refusal at 13.87 m in Glacial Till .
2. Seepage was observed from silt layer below 2.1 m.
3. Sloughing was observed from silt layer below 2.1 m.
4. Installed 25 mm diameter standpipe piezometer (SP14-28) to 11 m below ground surface with 0.3 m casagrande tip and flush mount up to 0.3 m below ground surafce.
5. Test hole backfilled with slough up to 11 m and silica sand up to 0.3 m below ground surface and plugged with top soil to ground surface.
6. Groundwater monitoring:
- Nov. 06, 2014 at Elv. 226.3 m.
- Nov. 20, 2014 at Elv. 226.6 m.
- Dec. 06, 2014 at Elv. 226.6 m.
- Dec. 18, 2014 at Elv. 226.6 m.

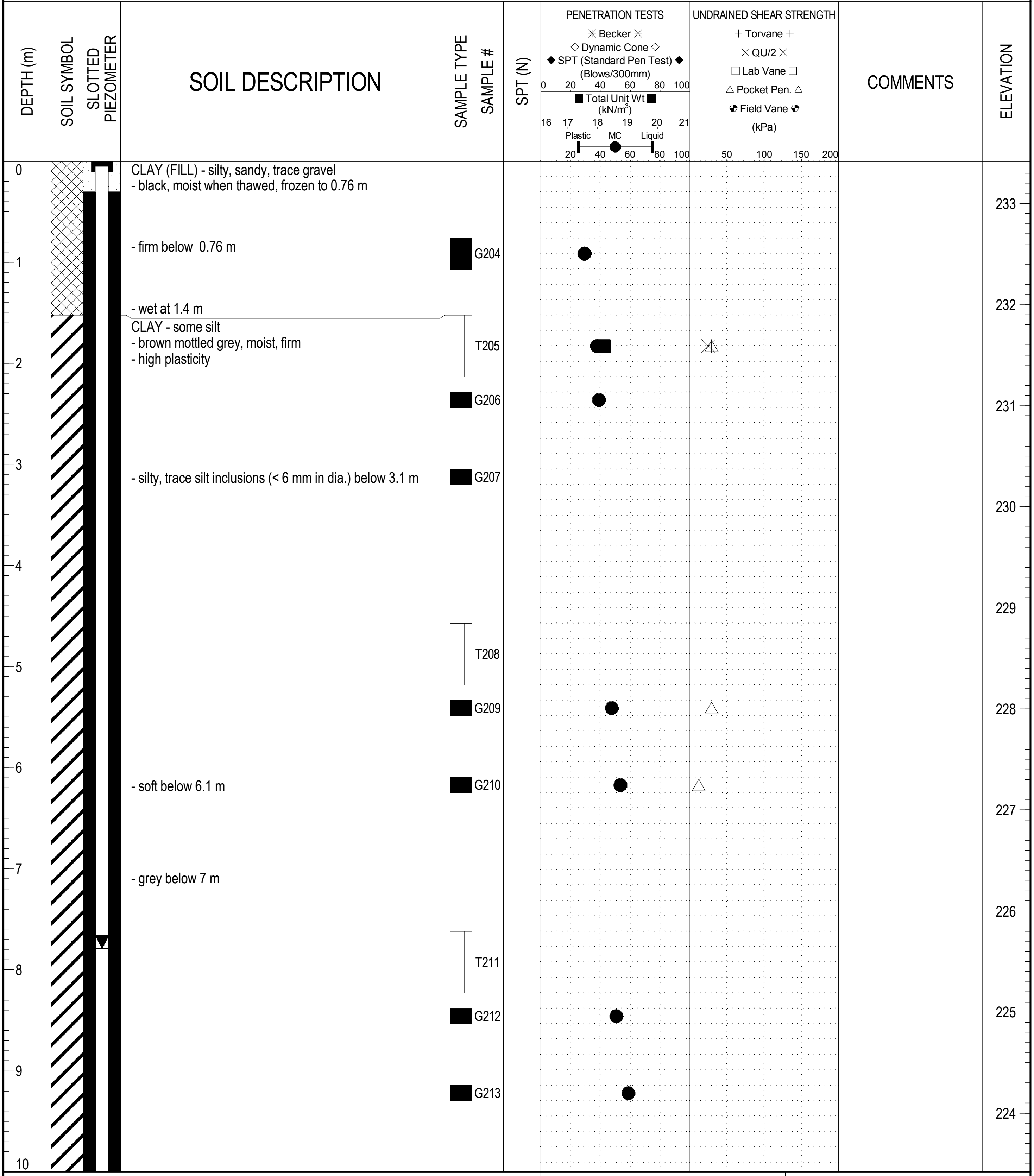
SPT Blow Count: (10,10,13) 75 %Recovery

AECOM
LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 13.87 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-29**
LOCATION: UTM: 14U, 5523602 m N, 630869 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 233.42

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁙ GRAVEL ▥ SLOUGH ⁙ GROUT ▨ CUTTINGS ⁙ SAND



CLAY (FILL) - silty, sandy, trace gravel
- black, moist when thawed, frozen to 0.76 m

- firm below 0.76 m

- wet at 1.4 m

CLAY - some silt
- brown mottled grey, moist, firm
- high plasticity

- silty, trace silt inclusions (< 6 mm in dia.) below 3.1 m

- soft below 6.1 m

- grey below 7 m

LOGGED BY: Mustafa Alkiki	COMPLETION DEPTH: 15.79 m
REVIEWED BY:	COMPLETION DATE: 12/1/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

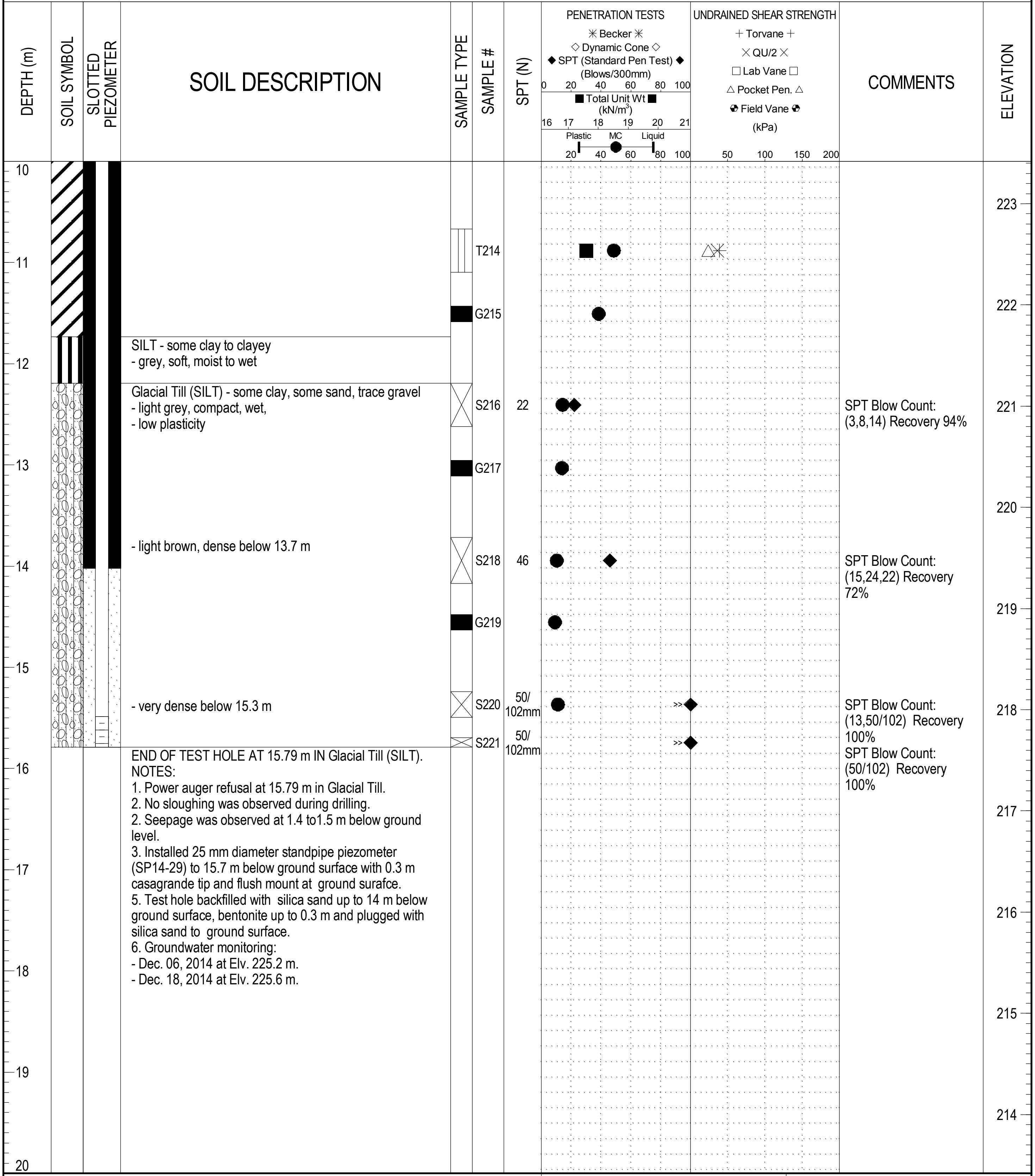
PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-29**
LOCATION: UTM: 14U, 5523602 m N, 630869 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 233.42

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁙ GRAVEL ▥ SLOUGH ⁙ GROUT ▨ CUTTINGS ⁙ SAND

DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
10–11.8		T214		
		G215		
11.8	SILT - some clay to clayey - grey, soft, moist to wet			
12.2	Glacial Till (SILT) - some clay, some sand, trace gravel - light grey, compact, wet, - low plasticity	S216	22	SPT Blow Count: (3,8,14) Recovery 94%
		G217		
	- light brown, dense below 13.7 m	S218	46	SPT Blow Count: (15,24,22) Recovery 72%
		G219		
	- very dense below 15.3 m	S220	50/ 102mm	SPT Blow Count: (13,50/102) Recovery 100%
		S221	50/ 102mm	SPT Blow Count: (50/102) Recovery 100%

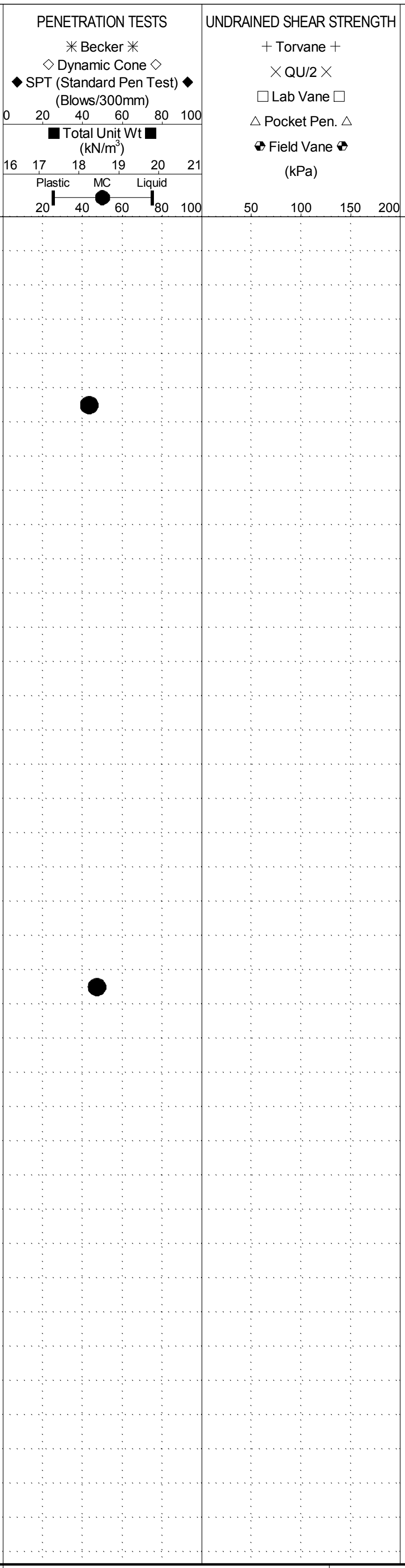
END OF TEST HOLE AT 15.79 m IN Glacial Till (SILT).
NOTES:
1. Power auger refusal at 15.79 m in Glacial Till.
2. No sloughing was observed during drilling.
2. Seepage was observed at 1.4 to1.5 m below ground level.
3. Installed 25 mm diameter standpipe piezometer (SP14-29) to 15.7 m below ground surface with 0.3 m casagrande tip and flush mount at ground surafce.
5. Test hole backfilled with silica sand up to 14 m below ground surface, bentonite up to 0.3 m and plugged with silica sand to ground surface.
6. Groundwater monitoring:
- Dec. 06, 2014 at Elv. 225.2 m.
- Dec. 18, 2014 at Elv. 225.6 m.



LOGGED BY: Mustafa Alkiki	COMPLETION DEPTH: 15.79 m
REVIEWED BY:	COMPLETION DATE: 12/1/14
PROJECT ENGINEER: Faris Khalil	Page 2 of 2

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-30**
LOCATION: UTM: 14U, 5523626 m N, 631117 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE



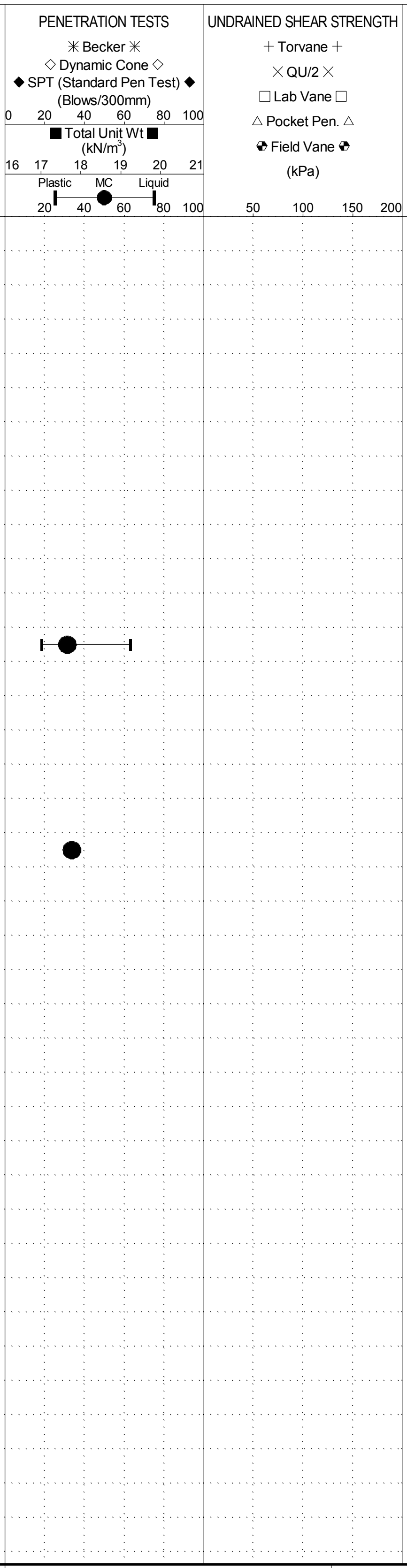
DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▩	SAND (FILL) - some gravel, trace cobble - brown, moist, frozen					
	▨	CLAY AND SILT, some sand, trace sulphates - brown, firm, moist, - low plasticity	■	G226		Gravel: 0.0%, Sand : 16.8%, Silt: 37.4 %, Clay: 45.9%	
1							1
2			■	G227			2
3			■	G228			3
4			■	G229			4
		END OF TEST HOLE AT 4.6 m IN CLAY AND SILT. NOTES: 1. No sloughing was observed upon drilling completion. 2. Seepage observed at 3.7 m. 3. Test hole backfilled with auger cuttings and silica sand, and sealed with bentonite at surface.					
5							5
6							

AECOM	LOGGED BY: Aaron Kaluzniak	COMPLETION DEPTH: 4.57 m
	REVIEWED BY:	COMPLETION DATE: 12/2/14
	PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-31**
LOCATION: UTM: 14U, 5523623 m N, 631090 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▩	SAND (FILL) - gravelly, some cobble, trace organics - brown, frozen					
			■	G222			
1	▨	CLAY - silty, trace sand - brown, firm, moist, - high plasticity					1
2			■	G223			2
			■	G224			
3	▥	SILT - clayey - brown, very soft, moist to wet, - intermediate plasticity					3
4			■	G225			4
		END OF TEST HOLE AT 4.6 m IN SILT. NOTES: 1. No sloughing was observed upon drilling completion. 2. Seepage was observed at 3.7 m. 3. Test hole backfilled with cuttings and silica sand, and sealed with bentonite at surface.					
5							5
6							



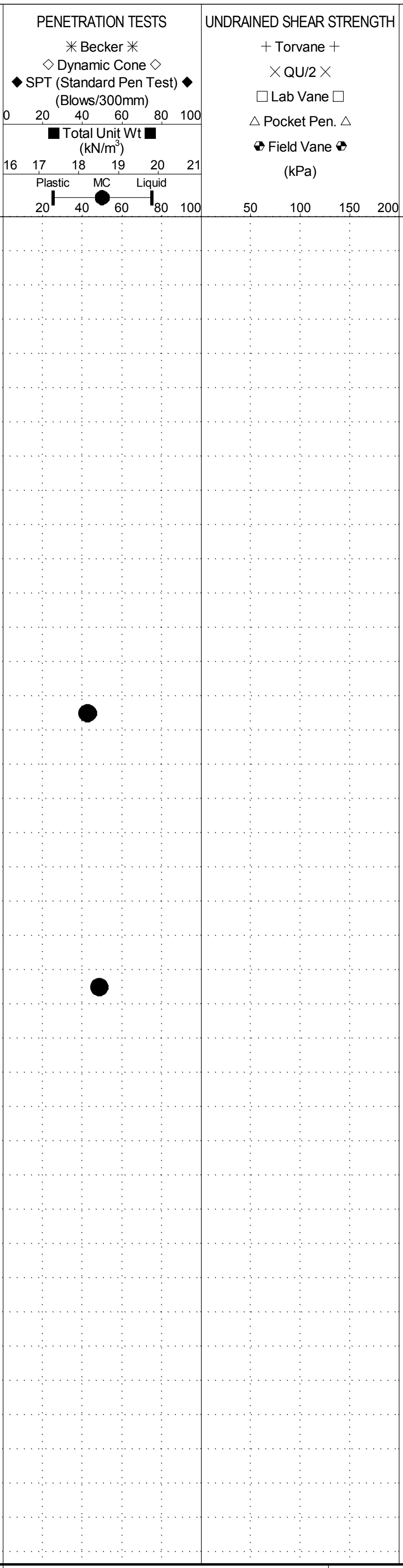
AECOM

LOGGED BY: Aaron Kaluzniak
REVIEWED BY:
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 4.57 m
COMPLETION DATE: 12/2/14

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-32**
LOCATION: UTM: 14U, 5523594 m N, 630979 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	DEPTH
0	▩	SAND (FILL) - gravelly, trace cobble, trace organics - brown, frozen					
	▩	CLAY (FILL) - some gravel - grey, moist, frozen - cobble (200 mm in dia., angular) at 2 m					
1							1
2	▨	CLAY - silty, trace sand lenses - brown to grey, moist, firm - high plasticity - trace sulphates	■ ■ ■	G230 G231 G232			2
3							3
4							4
5		END OF TEST HOLE AT 4.6 m IN CLAY. NOTES: 1. Seepage was observed at 2.0 m. 2. Sloughing was observed at 2.0 m. 3. Test hole backfilled with cuttings and silica sand, and sealed with bentonite at surface.					5
6							



LOGGED BY: Aaron Kaluzniak	COMPLETION DEPTH: 4.57 m
REVIEWED BY:	COMPLETION DATE: 12/2/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

AECOM

Appendix C

Laboratory Test Results

ENG-TECH
CONSULTING LIMITED

Unit 6 - 854 Marion Street
Winnipeg, Manitoba
R2J 0K4
eng-tech@mts.net
www.eng-tech.ca

ROCK CORE

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7

File No.: 14-027-01
Ref. No.: 14-27-1-10

Attention: Saba Ibrahim

Project: WAVERLY UNDERPASS; PROJECT # 60321148

Contractor: -
Date Cored: July 10, 14 and 15
Cored By: Client

Page: 1 of 1
Date Received: Nov 10/14
Received By: ENG-TECH

Core No.	Location	Length		Average Diameter (mm)	Compressive Strength (MPa)	Date Tested (m/d/y)
		Cored (mm)	Tested (mm)			
1	TH 14-02; sample No. R7, 24.0 – 24.3m.	254	113	63.0	194.4	Nov 13/14
2	TH 14-03; sample No. C6, 22.48 – 22.80m.	331	116	60.8	120.9	Nov 13/14
3	TH 14-04; sample No. R9, 25.4 – 25.6m.	244	118	60.9	114.9	Nov 13/14

__ METHOD ASTM D 2938
__ MOISTURE CONDITIONED
x OTHER (As received)
__ METHOD OTHER
__DRY CONDITIONED

Comments: The unconfined strength was determined in accordance with ASTM D2938-95 procedure with the cores in the as received moisture content. Core # 3 contained a vertical crack from the top to the bottom of specimen (as received).

Email: saba.ibrahim@aecom.com

ENG-TECH Consulting Limited

per
Danny Holfeld, Principal
Ph: (204) 233-1694 Fx: (204) 235-1579

AECOM Canada Ltd.
ATTN: SABA IBRAHIM
99 Commerce Drive
Winnipeg MB R3P 0Y7

Date Received:	17-SEP-14
Report Date:	26-SEP-14 08:06 (MT)
Version:	FINAL

Client Phone: 204-928-8461

Certificate of Analysis

Lab Work Order #:	**L1519224**
Project P.O. #:	NOT SUBMITTED
Job Reference:	60321148
C of C Numbers:	
Legal Site Desc:	

Judy Dalmaijer
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1519224-1 G2 G2-DEPTH 4' (TH14-01)							
Sampled By: CLIENT on 17-SEP-14							
Matrix: soil							
Miscellaneous Parameters							
% Moisture	24.2		0.10	%	19-SEP-14	20-SEP-14	R2953394
Sulphate	0.0187		0.0020	%	23-SEP-14	24-SEP-14	R2959632
Resistivity	2970		100	ohm cm	20-SEP-14	20-SEP-14	R2953569
pH	7.93		0.10	pH units	23-SEP-14	23-SEP-14	R2955764
L1519224-2 G12 G12-DEPTH 33' (TH14-01)							
Sampled By: CLIENT on 17-SEP-14							
Matrix: soil							
Miscellaneous Parameters							
% Moisture	38.2		0.10	%	19-SEP-14	20-SEP-14	R2953394
Sulphate	0.116		0.0020	%	23-SEP-14	24-SEP-14	R2959632
Resistivity	890		100	ohm cm	20-SEP-14	20-SEP-14	R2953569
pH	7.99		0.10	pH units	23-SEP-14	23-SEP-14	R2955764
L1519224-3 G17 G17-DEPTH 12.5 (TH-02)							
Sampled By: CLIENT on 17-SEP-14							
Matrix: soil							
Miscellaneous Parameters							
% Moisture	35.4		0.10	%	20-SEP-14	21-SEP-14	R2954088
Sulphate	0.0369		0.0020	%	23-SEP-14	24-SEP-14	R2959632
Resistivity	2870		100	ohm cm	20-SEP-14	20-SEP-14	R2953569
pH	7.84		0.10	pH units	23-SEP-14	23-SEP-14	R2955764
L1519224-4 G20 G20-DEPTH 22.5 (TH14-02)							
Sampled By: CLIENT on 17-SEP-14							
Matrix: soil							
Miscellaneous Parameters							
% Moisture	34.5		0.10	%	20-SEP-14	21-SEP-14	R2954088
Sulphate	0.102		0.0020	%	23-SEP-14	24-SEP-14	R2959632
Resistivity	1430		100	ohm cm	20-SEP-14	20-SEP-14	R2953569
pH	7.99		0.10	pH units	23-SEP-14	23-SEP-14	R2955764
L1519224-5 G43 G43-DEPTH 8' (TH14-04)							
Sampled By: CLIENT on 17-SEP-14							
Matrix: soil							
Miscellaneous Parameters							
% Moisture	29.0		0.10	%	20-SEP-14	21-SEP-14	R2954088
Sulphate	0.0089		0.0020	%	23-SEP-14	24-SEP-14	R2959632
Resistivity	2340		100	ohm cm	20-SEP-14	20-SEP-14	R2953569
pH	7.86		0.10	pH units	23-SEP-14	23-SEP-14	R2955764

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A
Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.			
RESISTIVITY-WT	Soil	Resistivity	MOEE E3137A
Resistivity on a soil is a 2:1 extraction of DI water to soil. Sample is tumbled for 30 min. Conductivity of the extraction is taken and the inverse is calculated for resistivity.			
SO4-WT	Soil	Sulphate	EPA 300.0

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Workorder: L1519224 Report Date: 26-SEP-14

Client: AECOM Canada Ltd.
99 Commerce Drive
Winnipeg MB R3P 0Y7
Contact: SABA IBRAHIM

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2953394								
WG1955193-2 LCS								
% Moisture			99.4		%		70-130	20-SEP-14
WG1955193-1 MB								
% Moisture			<0.10		%		0.1	20-SEP-14
Batch R2954088								
WG1955579-2 LCS								
% Moisture			100.2		%		70-130	21-SEP-14
WG1955579-1 MB								
% Moisture			<0.10		%		0.1	21-SEP-14
PH-WT	**Soil**							
Batch R2955764								
WG1956416-1 DUP		**L1519224-1**						
pH		7.93	7.90	J	pH units	0.03	0.3	23-SEP-14
WG1957107-1 LCS								
pH			7.00		pH units		6.9-7.1	23-SEP-14
WG1957107-2 LCS								
pH			7.02		pH units		6.9-7.1	23-SEP-14
RESISTIVITY-WT	**Soil**							
Batch R2953569								
WG1955539-2 DUP		**L1519224-5**						
Resistivity		2340	2700		ohm cm	15	25	20-SEP-14
SO4-WT	**Soil**							
Batch R2959632								
WG1957297-3 DUP		**L1519224-1**						
Sulphate		187	187		mg/kg	0.1	30	24-SEP-14
WG1957297-2 LCS								
Sulphate			101.2		%		70-130	24-SEP-14
WG1957297-1 MB								
Sulphate			<20		mg/kg		20	24-SEP-14

Quality Control Report

Workorder: L1519224 Report Date: 26-SEP-14

Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
J	Duplicate results and limits are expressed in terms of absolute difference.

Hold Time Exceedances:

All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.

AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Memorandum

To Saba Ibrahim

Page 1

CC

Subject Waverly Underpass

From Faris Khalil

Date December 8, 2014

Project Number 60321148

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Two (2) Moisture Content tests.
- Two (2) Atterberg Limits (3 points) tests.
- Two (2) Grain Size Distribution (hydrometer method) tests.

If you have any questions, please contact the undersigned.

Sincerely,

Faris Khalil, M.Sc., PMP, P.Eng.
Manager, Geotechnical Engineering

Att.

AECOM

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	EManimbao
Sample Number:	Varies	Date Tested:	November 27, 2014

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	**Moisture Content (%)**
TH14-17	G113	0.91 - 1.07 m	**28.0**
TH14-28	G196	7.01 - 7.16 m	**50.0**

Location	Sample	Depth (m)	**Moisture Content (%)**

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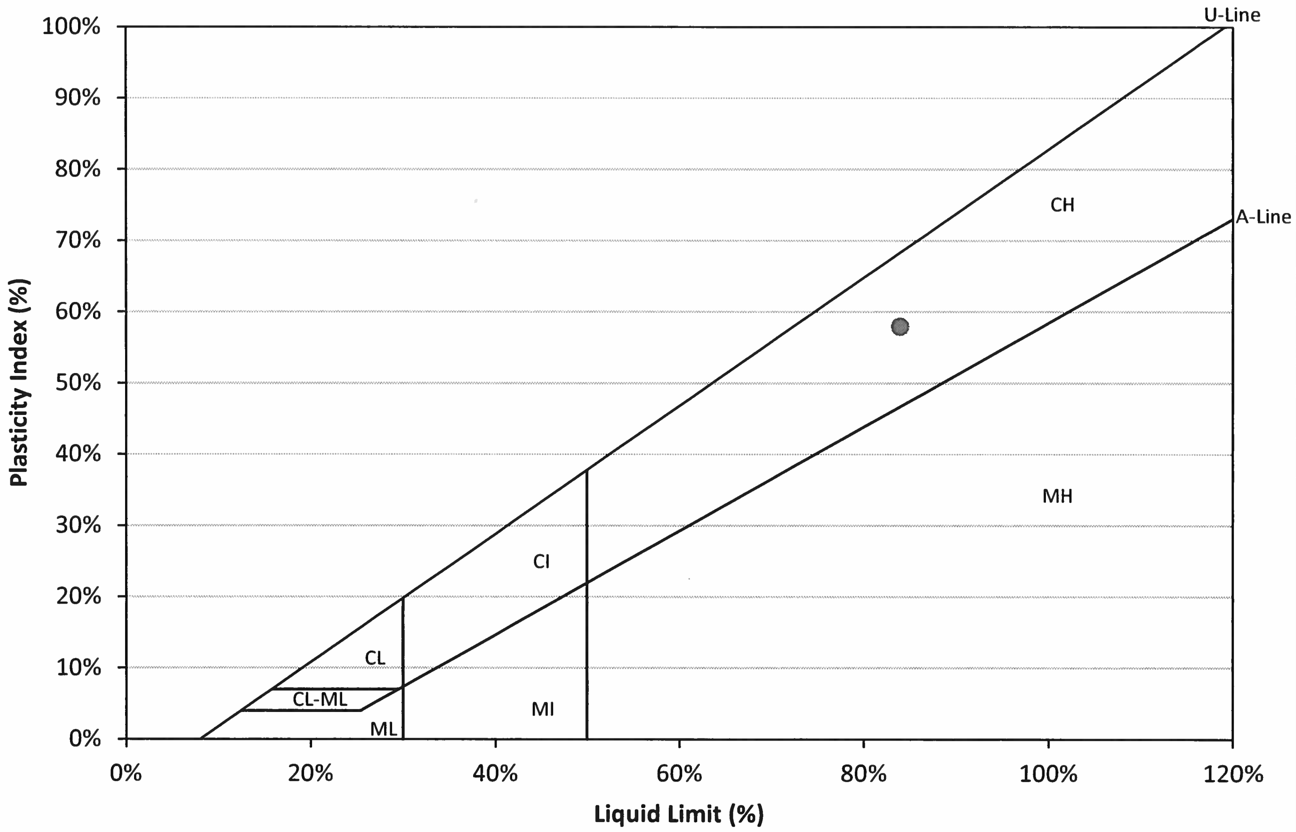
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-28	Sample Date:	November 1, 2014
Sample Depth:	7.01 m	Lab Technician:	EManimbao
Sample Number:	G196	Date Tested:	December 3, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	22	18
Wet Sample (g)	9.9	8.6	11.1
Dry Sample (g)	5.5	4.7	6.0
Water Content (%)	82.2%	84.1%	86.2%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.5	7.9
Dry Sample (g)	6.0	6.2
Water Content (%)	25.4%	26.7%



Liquid Limit (%): 83.9%	Plastic Limit (%): 26.0%	Plasticity Index (%): 57.9%

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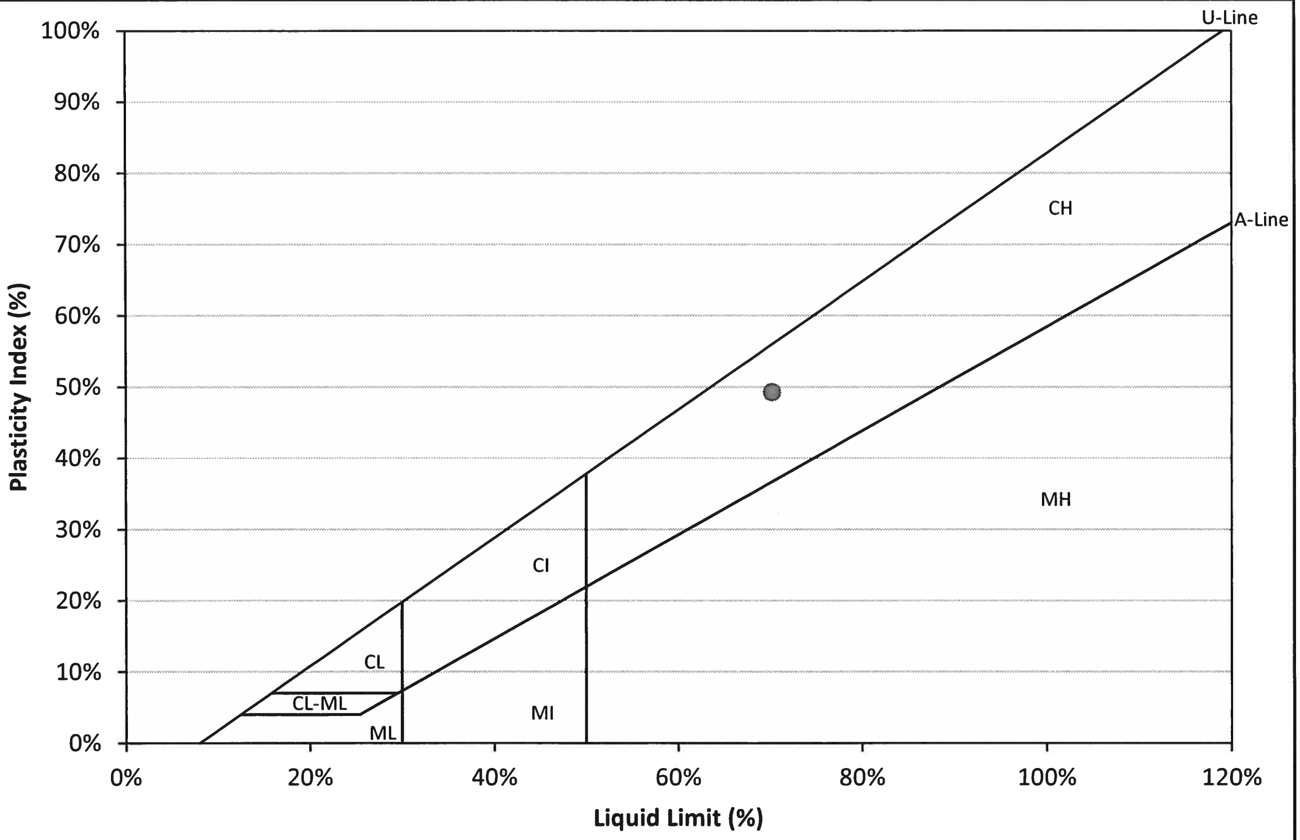
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-17	Sample Date:	November 1, 2014
Sample Depth:	0.91 m	Lab Technician:	EManimbao
Sample Number:	G113	Date Tested:	December 3, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	24	21
Wet Sample (g)	8.7	9.0	9.3
Dry Sample (g)	5.2	5.3	5.4
Water Content (%)	68.1%	70.0%	71.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.4	7.4
Dry Sample (g)	6.1	6.1
Water Content (%)	21.2%	20.7%



Liquid Limit (%): 70.2%	Plastic Limit (%): 21.0%	Plasticity Index (%): 49.2%

GRAIN SIZE DISTRIBUTION

(ASTM D422-63)

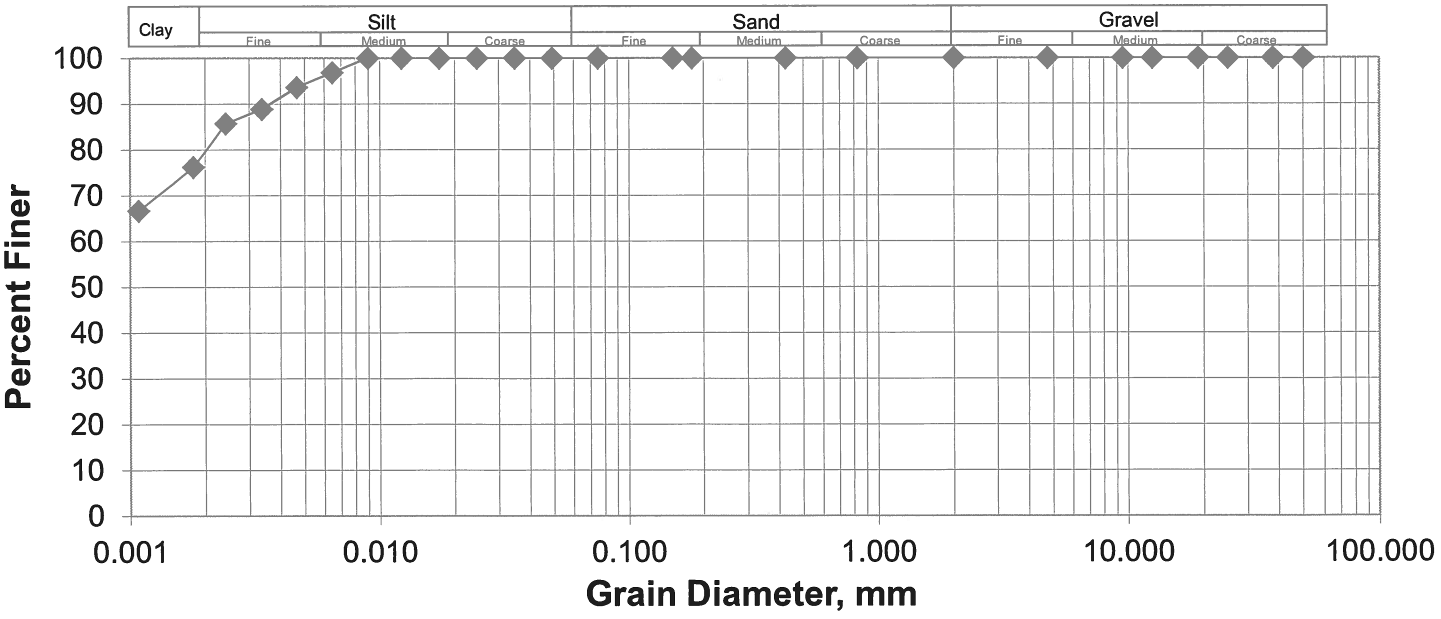
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 1-Dec-14
Tested By: MLotecki

Hole No.: 14-28
Sample No.: G196
Depth: 7.01 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	100.0
38.0	100.0	0.83	100.0	0.0491	100.0
25.0	100.0	0.43	100.0	0.0347	100.0
19.0	100.0	0.18	100.0	0.0246	100.0
12.5	100.0	0.15	100.0	0.0174	100.0
9.5	100.0	0.075	100.0	0.0123	100.0
4.75	100.0			0.0090	100.0
2.00	100.0			0.0065	96.8
				0.0047	93.6
				0.0034	88.9
				0.0024	85.7
				0.0018	76.2
				0.0011	66.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	0.0%	**Silt**	20.7%
Sand	0.0%	**Clay**	79.3%

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION

(ASTM D422-63)

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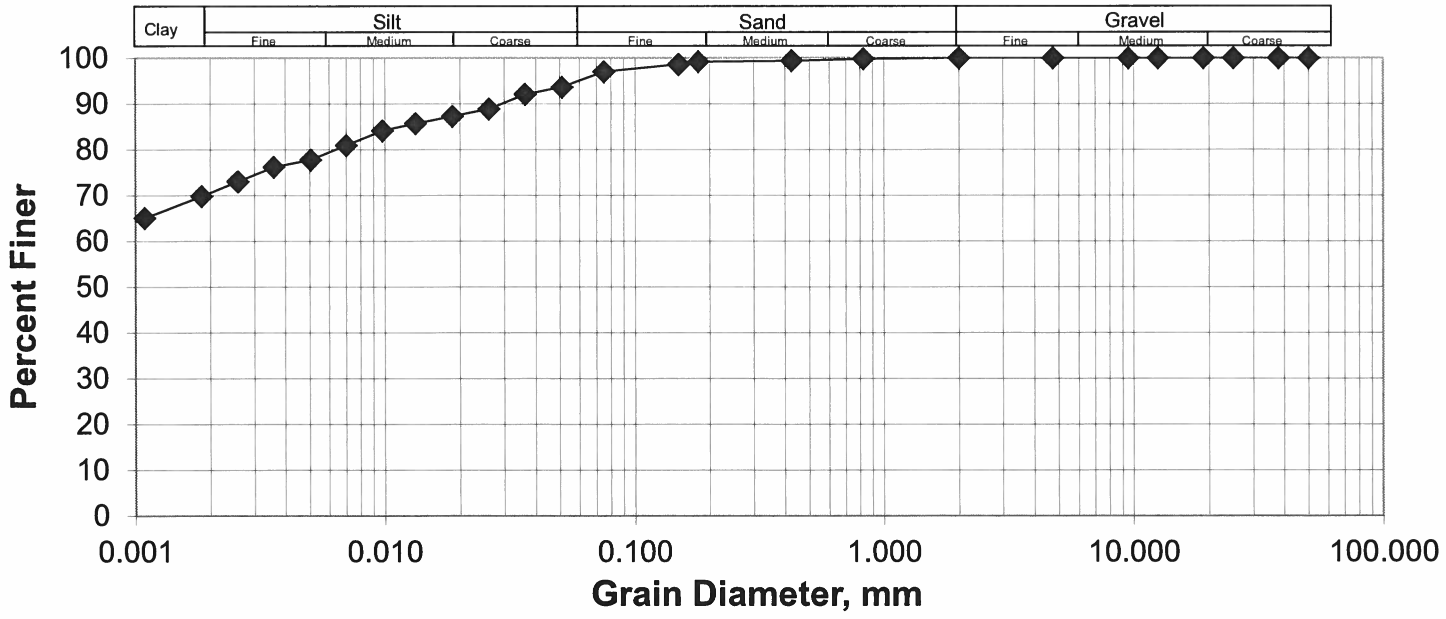
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 1-Dec-14
Tested By: MLotecki

Hole No.: 14-17
Sample No.: G113
Depth: 0.91 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	97.0
38.0	100.0	0.83	99.8	0.0510	93.6
25.0	100.0	0.43	99.4	0.0363	92.1
19.0	100.0	0.18	99.2	0.0261	88.9
12.5	100.0	0.15	98.6	0.0186	87.3
9.5	100.0	0.075	97.0	0.0133	85.7
4.75	100.0			0.0098	84.1
2.00	100.0			0.0070	80.9
				0.0050	77.8
				0.0036	76.2
				0.0026	73.0
				0.0018	69.8
				0.0011	65.1

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**24.4%**
Sand	**5.1%**	**Clay**	**70.5%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

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Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	MLotecki
Sample Number:	Varies	Date Tested:	December 4, 2014

Group Index (ASTM D3282-09)

Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes

Location	Sample	Depth (m)	% Passing No. 200	Liquid Limit	Plasticity Index	Group Classification
TH14-28	G196	7.01 - 7.16 m	100.0	83.9	57.9	A-7-6(68)
TH14-17	G113	0.91 - 1.07 m	97.0	70.2	49.2	A-7-6(54)

AECOM

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Memorandum

To	Saba Ibrahim	Page	1
CC			
Subject	Waverly Underpass		
From	Faris Khalil		
Date	December 16, 2014	Project Number	60321148

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Eighteen (18) Moisture Content tests.
- One (1) Atterberg Limits (3 points) test.
- One (1) Grain Size Distribution (hydrometer method) test.
- Two (2) Torvane, Pocket Penetrometer, Moisture Content, Bulk Density and Visual Description with Unconfined Compressive Strength, on Shelby tube samples.
- One (1) Oedometer Consolidation test.

If you have any questions, please contact the undersigned.

Sincerely,

Faris Khalil, M.Sc., PMP, P.Eng.
Manager, Geotechnical Engineering

Att.

AECOM

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Winnipeg Geotechnical Laboratory
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Project Name:	Waverly Underpass Phase III	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting Ltd.	Field Technician:	AKaluzniak
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	EManimbao
Sample Number:	Varies	Date Tested:	December 4, 2014

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	Moisture Content (%)
TH14-30	G226	0.76 - 0.91 m	**43.3**
-	G228	3.35 - 3.51 m	**47.2**
TH14-31	G223	1.83 - 1.98 m	**31.4**
-	G224	2.74 - 2.90 m	**33.6**
TH14-32	G230	2.13 - 2.29 m	**42.6**
-	G231	3.35 - 3.51 m	**48.5**
TH14-29	G204	0.84 - 0.99 m	**29.3**
-	G206	2.29 - 2.44 m	**39.0**
-	G209	5.33 - 5.49 m	**47.6**
-	G210	6.10 - 6.25 m	**53.4**
-	G212	8.38 - 8.53 m	**50.7**
-	G213	9.14 - 9.30 m	**58.8**
-	G215	11.43 - 11.58 m	**38.4**
-	S216	12.19 - 12.34 m	**14.2**
-	G217	12.95 - 13.11 m	**13.8**
-	S218	13.72 - 13.87 m	**10.3**
-	G219	14.48 - 14.63 m	**9.1**
-	S220	15.24 - 15.39 m	**11.1**

Location	Sample	Depth (m)	Moisture Content (%)

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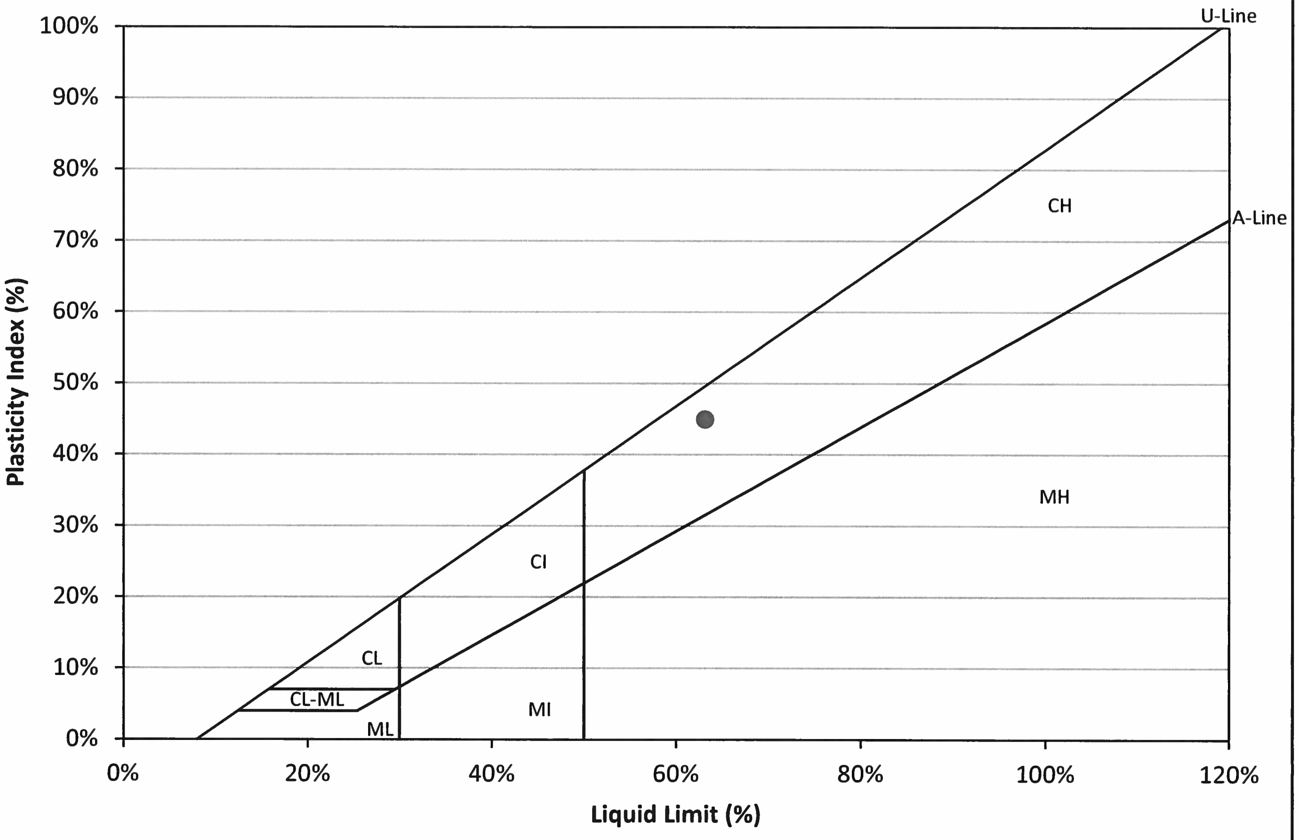
Project Name:	Waverly Underpass Phase III	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	AKaluzniak
Sample Location:	TH14-31	Sample Date:	November 1, 2014
Sample Depth:	1.83 m	Lab Technician:	MLotecki
Sample Number:	G223	Date Tested:	November 26, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	19	26	31
Wet Sample (g)	11.1	9.5	11.9
Dry Sample (g)	6.8	5.8	7.4
Water Content (%)	64.8%	62.9%	61.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.0	9.4
Dry Sample (g)	5.1	7.9
Water Content (%)	18.2%	18.2%



Liquid Limit (%): 63.1%	Plastic Limit (%): 18.2%	Plasticity Index (%): 44.9%

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

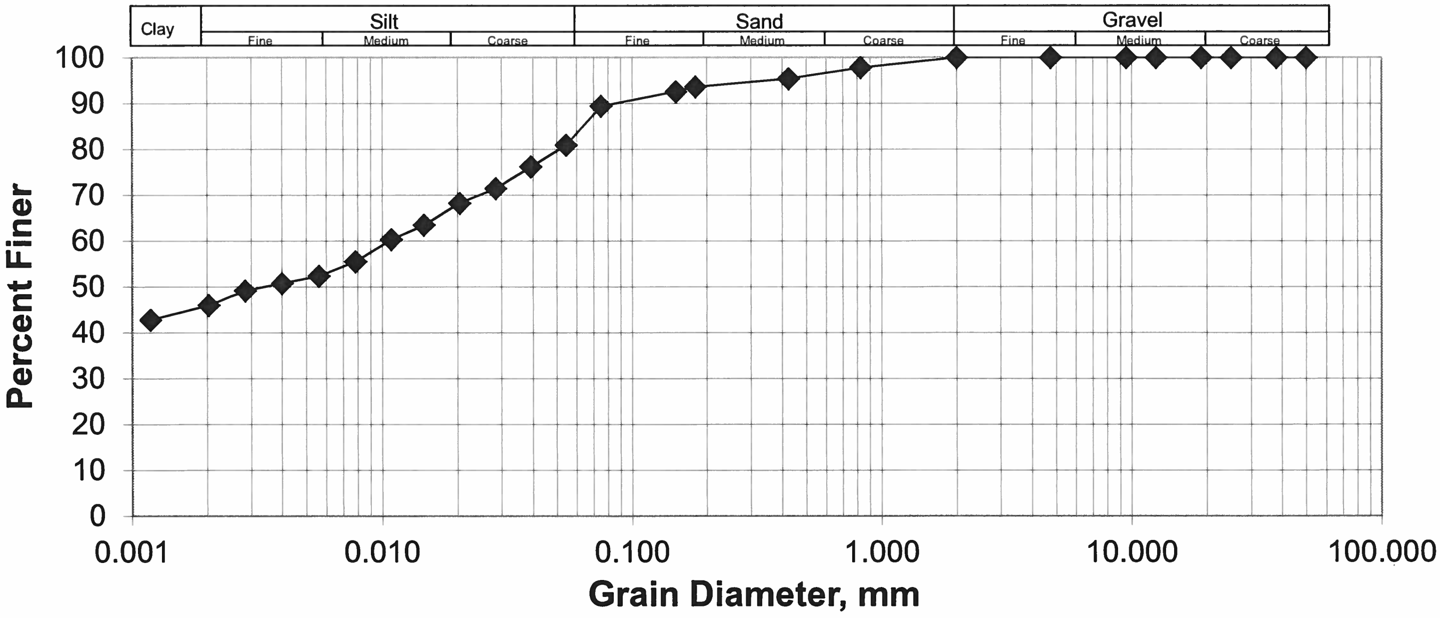
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase III
Date Tested: 9-Dec-14
Tested By: MLotecki

Hole No.: TH14-30
Sample No.: G226
Depth: 0.76 m
Date Sampled: 2-Dec-14
Sampled By: AECOM (AKaluzniak)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	89.4
38.0	100.0	0.83	97.8	0.0544	80.9
25.0	100.0	0.43	95.4	0.0394	76.2
19.0	100.0	0.18	93.6	0.0284	71.4
12.5	100.0	0.15	92.6	0.0204	68.2
9.5	100.0	0.075	89.4	0.0147	63.5
4.75	100.0			0.0109	60.3
2.00	100.0			0.0078	55.5
				0.0056	52.3
				0.0040	50.8
				0.0028	49.2
				0.0020	46.0
				0.0012	42.8

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**37.4%**
Sand	**16.8%**	**Clay**	**45.9%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-29**
SAMPLE NO.:	**T214**
SAMPLE DEPTH:	10.67 - 11.28 m
DATE TESTED:	9-Dec-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.40
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**39.2**
Undrained Shear Strength (ksf)	0.82
POCKET PENETROMETER	
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**71.4**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**35.7**
Undrained Shear Strength (ksf)	**0.746**
MOISTURE CONTENT	
Tare Number	AB01
Wt. Sample wet + tare (g)	415.2
Wt. Sample dry + tare (g)	282.2
Wt. Tare (g)	8.7
Moisture Content %	**48.6**
BULK DENSITY	
Sample Wt. (g)	1119.8
Diameter 1 (cm)	7.23
Diameter 2 (cm)	7.22
Diameter 3 (cm)	7.23
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	15.35
Length 2 (cm)	15.36
Length 3 (cm)	15.30
Avg. Length (cm)	**15.34**
Volume (cm^3)	629.1
Moisture content (%)	48.6
Bulk Density (g/cm^3)	1.780
Bulk Density (kN/m^3)	**17.5**
Bulk Density (pcf)	**111.1**
Dry Density (kN/m^3)	**11.75**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

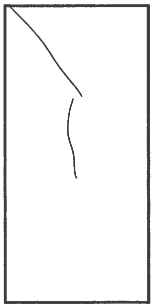
AECOM

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-29
SAMPLE NO.:	T214
SAMPLE DEPTH:	10.67 - 11.28 m
SAMPLE DATE:	February, 2014
TEST DATE:	9-Dec-14

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, trace stones, grey, moist, firm,	
int. - high plasticity,	
MOISTURE CONTENT:	48.6

SAMPLE DIAM.(Do):	72.27	**(mm)**	**INITIAL AREA, Ao:**	4101.7	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.37	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.84	**(0.5<R<2 % / minute)**



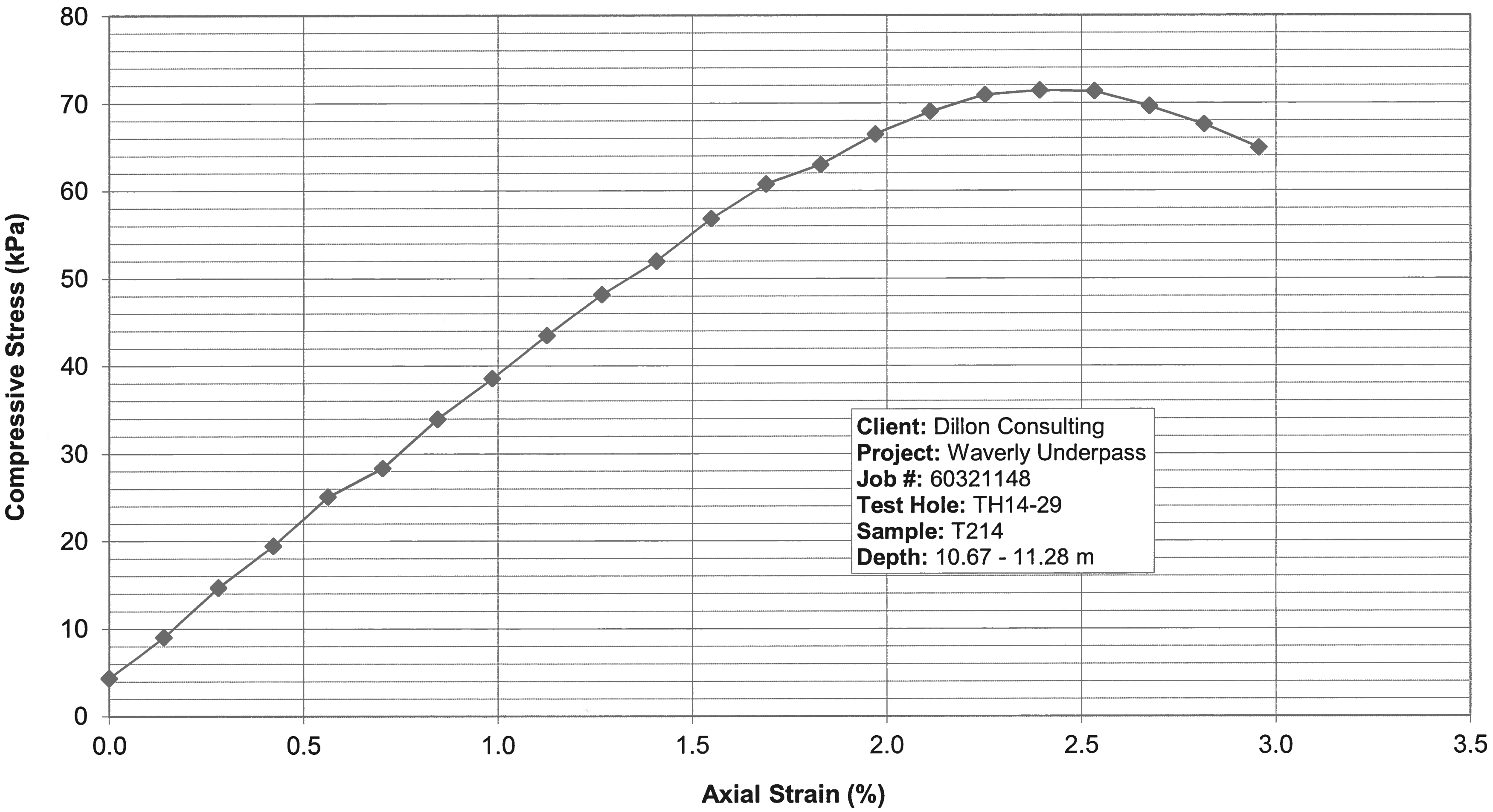
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_c		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0004	0.00	6.36	4.03	0.63	0.091	4.4
0.02	0.0009	0.14	6.37	8.34	1.31	0.189	9.0
0.03	0.0015	0.28	6.38	13.59	2.13	0.307	14.7
0.03	0.0019	0.42	6.38	17.99	2.82	0.406	19.4
0.04	0.0025	0.56	6.39	23.24	3.63	0.523	25.1
0.05	0.0028	0.70	6.40	26.33	4.11	0.592	28.4
0.06	0.0034	0.84	6.41	31.58	4.92	0.709	34.0
0.07	0.0038	0.99	6.42	35.89	5.59	0.805	38.5
0.08	0.0043	1.13	6.43	40.57	6.31	0.909	43.5
0.09	0.0048	1.27	6.44	44.98	6.98	1.006	48.2
0.09	0.0052	1.41	6.45	48.63	7.54	1.086	52.0
0.10	0.0057	1.55	6.46	53.22	8.24	1.187	56.8
0.11	0.0061	1.69	6.47	56.97	8.81	1.269	60.7
0.12	0.0063	1.83	6.48	59.12	9.13	1.315	62.9
0.13	0.0067	1.97	6.49	62.50	9.64	1.388	66.4
0.14	0.0069	2.11	6.49	65.03	10.01	1.442	69.0
0.14	0.0071	2.25	6.50	66.90	10.29	1.481	70.9
0.15	0.0072	2.39	6.51	67.46	10.36	1.491	71.4
0.16	0.0072	2.53	6.52	67.46	10.34	1.489	71.3
0.17	0.0070	2.67	6.53	65.96	10.10	1.454	69.6
0.18	0.0068	2.82	6.54	64.09	9.80	1.411	67.5
0.19	0.0066	2.96	6.55	61.65	9.41	1.355	64.9

UNCONFINED COMPRESSIVE STRENGTH, q_u:	71.41	kPa
(based on maximum q_u value)	1.491	ksf
UNDRAINED SHEAR STRENGTH, S_u:	35.71	kPa
(based on maximum q_u value)	0.746	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-29**
SAMPLE NO.:	**T205**
SAMPLE DEPTH:	3.05 - 3.66 m
DATE TESTED:	9-Dec-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.30
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**29.4**
Undrained Shear Strength (ksf)	0.61
POCKET PENETROMETER	
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.75
Undrained Shear Strength (kPa)	**35.9**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**49.0**
Unconfined compressive strength (ksf)	**1.0**
Undrained Shear Strength (kPa)	**24.5**
Undrained Shear Strength (ksf)	**0.512**
MOISTURE CONTENT	
Tare Number	AB19
Wt. Sample wet + tare (g)	411.2
Wt. Sample dry + tare (g)	301.2
Wt. Tare (g)	8.6
Moisture Content %	**37.6**
BULK DENSITY	
Sample Wt. (g)	1156
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.22
Diameter 3 (cm)	7.22
Avg. Diameter (cm)	**7.21**
Length 1 (cm)	15.30
Length 2 (cm)	15.32
Length 3 (cm)	15.28
Avg. Length (cm)	**15.30**
Volume (cm^3)	625.2
Moisture content (%)	37.6
Bulk Density (g/cm^3)	1.849
Bulk Density (kN/m^3)	**18.1**
Bulk Density (pcf)	**115.4**
Dry Density (kN/m^3)	**13.18**

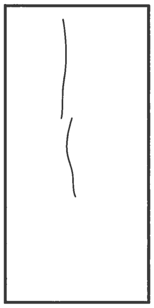
AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-29
SAMPLE NO.:	T205
SAMPLE DEPTH:	3.05 - 3.66 m
SAMPLE DATE:	February, 2014
TEST DATE:	9-Dec-14

SOIL DESCRIPTION:	
CLAY; silty, trace sand, brown, moist, soft, crumbly, high plasticity	
MOISTURE CONTENT:	37.6

SAMPLE DIAM.(Do):	72.13	(mm)	INITIAL AREA, Ao:	4086.6	(mm^2)
SAMPLE LENGTH, (Lo):	153.00	(mm)	PISTON RATE:	0.051	(inches / minute)
L / D RATIO:	2.12	(2 < L/D < 2.5)	AXIAL STRAIN RATE, R:	0.85	(0.5<R<2 % / minute)



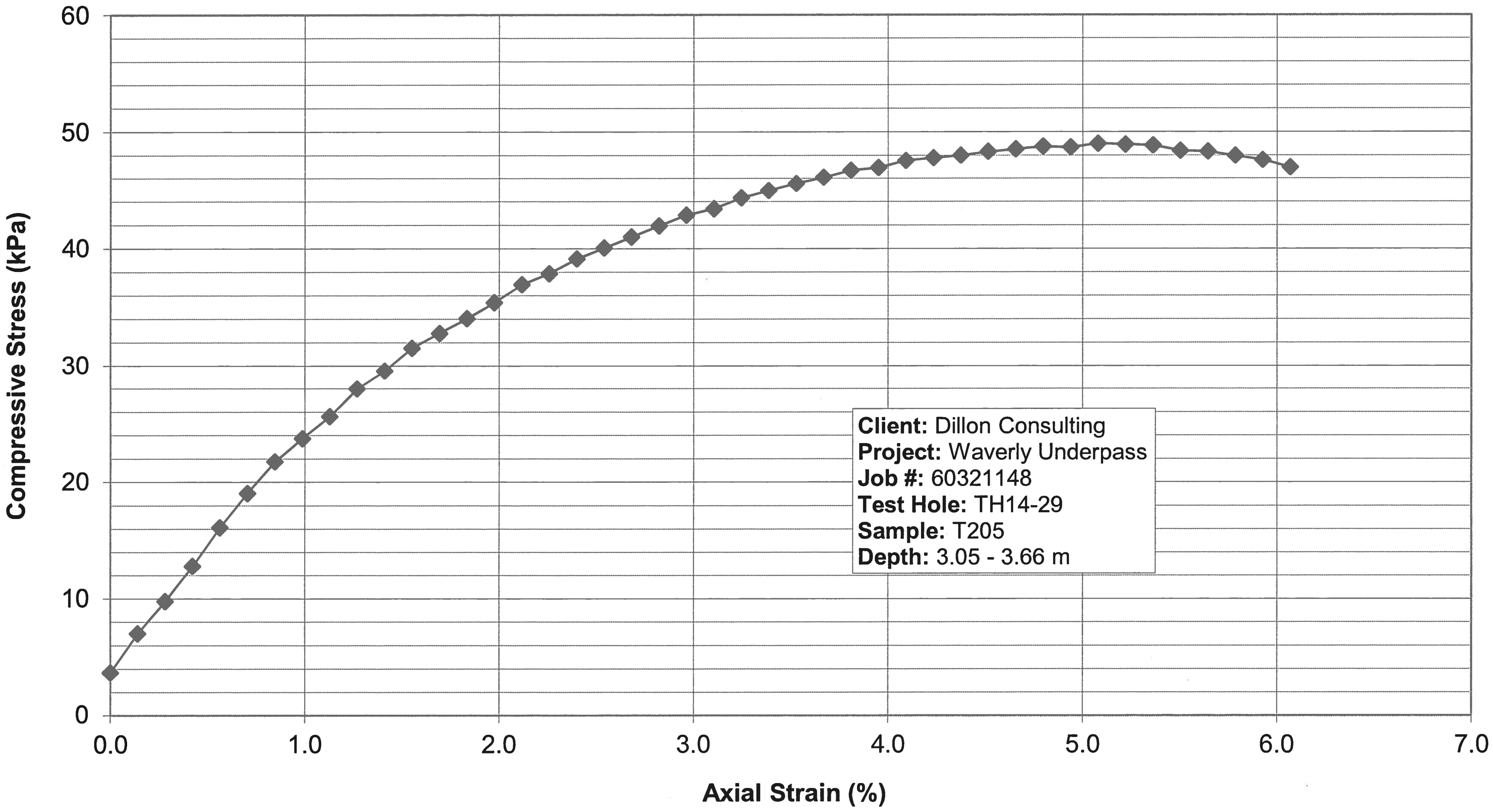
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_C		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0004	0.00	6.33	3.37	0.53	0.077	3.7
0.02	0.0007	0.14	6.34	6.47	1.02	0.147	7.0
0.03	0.0010	0.28	6.35	9.00	1.42	0.204	9.8
0.03	0.0013	0.42	6.36	11.81	1.86	0.267	12.8
0.04	0.0016	0.56	6.37	14.90	2.34	0.337	16.1
0.05	0.0019	0.71	6.38	17.62	2.76	0.398	19.0
0.06	0.0022	0.85	6.39	20.15	3.15	0.454	21.7
0.07	0.0024	0.99	6.40	22.02	3.44	0.496	23.7
0.08	0.0025	1.13	6.41	23.80	3.71	0.535	25.6
0.09	0.0028	1.27	6.42	26.05	4.06	0.585	28.0
0.09	0.0029	1.41	6.42	27.55	4.29	0.617	29.6
0.10	0.0031	1.55	6.43	29.42	4.57	0.658	31.5
0.11	0.0033	1.69	6.44	30.64	4.76	0.685	32.8
0.12	0.0034	1.83	6.45	31.86	4.94	0.711	34.0
0.13	0.0035	1.98	6.46	33.17	5.13	0.739	35.4
0.14	0.0037	2.12	6.47	34.67	5.36	0.771	36.9
0.14	0.0038	2.26	6.48	35.61	5.49	0.791	37.9
0.15	0.0039	2.40	6.49	36.82	5.67	0.817	39.1
0.16	0.0040	2.54	6.50	37.76	5.81	0.837	40.1
0.17	0.0041	2.68	6.51	38.70	5.95	0.856	41.0
0.18	0.0042	2.82	6.52	39.64	6.08	0.876	41.9
0.19	0.0043	2.96	6.53	40.57	6.22	0.895	42.9
0.20	0.0044	3.10	6.54	41.13	6.29	0.906	43.4
0.20	0.0045	3.25	6.55	42.07	6.43	0.925	44.3
0.21	0.0046	3.39	6.56	42.73	6.52	0.938	44.9
0.22	0.0046	3.53	6.57	43.38	6.61	0.951	45.6
0.23	0.0047	3.67	6.58	43.95	6.68	0.962	46.1
0.24	0.0048	3.81	6.59	44.60	6.77	0.975	46.7
0.25	0.0048	3.95	6.59	44.88	6.81	0.980	46.9
0.26	0.0049	4.09	6.60	45.54	6.90	0.993	47.5
0.26	0.0049	4.23	6.61	45.82	6.93	0.998	47.8
0.27	0.0049	4.37	6.62	46.10	6.96	1.002	48.0
0.28	0.0050	4.52	6.63	46.48	7.01	1.009	48.3
0.29	0.0050	4.66	6.64	46.76	7.04	1.013	48.5
0.30	0.0050	4.80	6.65	47.04	7.07	1.018	48.7
0.31	0.0050	4.94	6.66	47.04	7.06	1.017	48.7
0.31	0.0051	5.08	6.67	47.41	7.10	1.023	49.0
0.32	0.0051	5.22	6.68	47.41	7.09	1.022	48.9
0.33	0.0051	5.36	6.69	47.41	7.08	1.020	48.8
0.34	0.0050	5.50	6.70	47.04	7.02	1.010	48.4
0.35	0.0050	5.64	6.71	47.04	7.01	1.009	48.3
0.36	0.0050	5.79	6.72	46.76	6.95	1.001	47.9
0.37	0.0050	5.93	6.73	46.48	6.90	0.994	47.6
0.37	0.0049	6.07	6.74	45.91	6.81	0.980	46.9

UNCONFINED COMPRESSIVE STRENGTH, q_u:	48.99	kPa
(based on maximum q_u value)	1.023	ksf
UNDRAINED SHEAR STRENGTH, S_u:	24.49	kPa
(based on maximum q_u value)	0.512	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-29
Sample: T205
Depth: 3.05 - 3.66 m

Consolidation Test

AECOM
MATERIALS LABORATORY
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99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 284-0580 fax (204) 475-3646

Client: Dillon Consulting Ltd.
Project: Waverley Underpass Phase II
Job No: 60321148
Date : November 28 to December 12, 2014

Hole No. TH14-28
Sample No. T197
Depth: 7.62 - 8.23 m
Sample Description:

Box Size 70.0 mm ϕ

Height 20 mm

Moisture Content	Intial	Final
Tare Number		
Wt. Wet Soil & Tare (g)	353.4	237.5
Wt. Dry Soil & Tare (g)	233.3	192.6
Wt. Water (g)	120.1	44.9
Wt. Tare (g)	8.3	112.6
% Moisture	53.4	56.1
Hs (mm)	8.1	7.7

Density	Intial	Final
Wt. Sh. Box & Soil (g)	231.6	227.5
Wt. Sh. Box (g)	102.5	102.5
Wt. Soil Specimen (g)	129.1	125
Height of Spec. (mm)	20.0	17.7
Volume (mm^3)	76967	67984
Bulk Density (kN/m^3)	16.77	18.39
Dry Density (kN/m3)	10.94	11.78

Machine # 1
e (void Ratio) 1.47

Ring #
Spec. Gravity (assumed 2.7

Load 0.209 kg

Free Swell

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
11/28/2014 14:00	0	1674	0	0.00	0.00	1.469	0.53	-
11/28/2014 14:53	53	1718	7.28	0.11	0.56	1.483	0.53	-
11/28/2014 15:15	75	1752	8.66	0.20	0.99	1.493	0.53	-
11/28/2014 16:40	160	1817	12.65	0.36	1.82	1.514	0.53	-
11/29/2014 16:15	1575	1901	39.69	0.58	2.88	1.540	0.53	-
11/30/2014 14:40	2920	1911	54.04	0.60	3.01	1.543	0.53	-
12/1/2014 8:30	3990	1914	63.17	0.61	3.05	1.544	0.53	-
12/1/2014 11:15	4155	1915	64.46	0.61	3.06	1.544	0.53	-

Load 0.909 kg 2 LBS

Consolidation Load 1

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/2/2014 9:30	0	1915	0	0.00	0.00	1.514	25.48	0.00
	0.25	1880	0.50	-0.09	-0.44	1.503	25.48	0.44
	0.5	1877	0.71	-0.10	-0.48	1.502	25.48	0.48
	1	1873	1.00	-0.11	-0.53	1.501	25.48	0.53
	2	1867	1.41	-0.12	-0.61	1.499	25.48	0.61
	4	1861	2.00	-0.14	-0.69	1.497	25.48	0.69
	8	1853	2.83	-0.16	-0.79	1.494	25.48	0.79
	15	1845	3.87	-0.18	-0.89	1.492	25.48	0.89
	30	1837	5.48	-0.20	-0.99	1.489	25.48	0.99
	60	1833	7.75	-0.21	-1.04	1.488	25.48	1.04
	120	1827	10.95	-0.22	-1.12	1.486	25.48	1.12
	240	1823	15.49	-0.23	-1.17	1.485	25.48	1.17
	480	1820	21.91	-0.24	-1.21	1.484	25.48	1.21
	1440	1817	37.95	-0.25	-1.24	1.483	25.48	1.24

Load 1.818 kg 4 LBS

Consolidation Load 2

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/3/2014 8:00	0	1817	0	0.00	0.00	1.483	50.96	1.24
	0.25	1801	0.50	-0.04	-0.20	1.478	50.96	1.45
	0.5	1798	0.71	-0.05	-0.24	1.477	50.96	1.49
	1	1795	1.00	-0.06	-0.28	1.476	50.96	1.52
	2	1791	1.41	-0.07	-0.33	1.475	50.96	1.57
	4	1785	2.00	-0.08	-0.41	1.473	50.96	1.65
	8	1778	2.83	-0.10	-0.50	1.471	50.96	1.74
	15	1766	3.87	-0.13	-0.65	1.467	50.96	1.89
	30	1754	5.48	-0.16	-0.80	1.463	50.96	2.04
	60	1743	7.75	-0.19	-0.94	1.460	50.96	2.18
	120	1735	10.95	-0.21	-1.04	1.457	50.96	2.29
	240	1729	15.49	-0.22	-1.12	1.455	50.96	2.36
	480	1725	21.91	-0.23	-1.17	1.454	50.96	2.41
	1440	1721	37.95	-0.24	-1.22	1.453	50.96	2.46

Load 3.636 kg 8 LBS

Consolidation Load 3

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/4/2014 8:00	0	1721	0	0.00	0.00	1.453	101.92	2.46
	0.25	1700	0.50	-0.05	-0.27	1.446	101.92	2.73
	0.5	1687	0.71	-0.09	-0.43	1.442	101.92	2.90
	1	1684	1.00	-0.09	-0.47	1.441	101.92	2.93
	2	1677	1.41	-0.11	-0.56	1.439	101.92	3.02
	4	1663	2.00	-0.15	-0.74	1.435	101.92	3.20
	8	1648	2.83	-0.19	-0.93	1.430	101.92	3.39
	15	1634	3.87	-0.22	-1.10	1.426	101.92	3.57
	30	1617	5.48	-0.26	-1.32	1.420	101.92	3.78
	60	1600	7.75	-0.31	-1.54	1.415	101.92	4.00
	120	1590	10.95	-0.33	-1.66	1.412	101.92	4.13
	240	1580	15.49	-0.36	-1.79	1.409	101.92	4.25
	480	1576	21.91	-0.37	-1.84	1.407	101.92	4.31
	1440	1569	37.95	-0.39	-1.93	1.405	101.92	4.39

Load 7.273 kg 16 LBS

Consolidation Load 4

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/5/2014 8:00	0	1469	0	0.00	0.00	1.405	203.85	4.39
	0.25	1410	0.50	-0.15	-0.75	1.387	203.85	5.14
	0.5	1406	0.71	-0.16	-0.80	1.385	203.85	5.19
	1	1399	1.00	-0.18	-0.89	1.383	203.85	5.28
	2	1390	1.41	-0.20	-1.00	1.380	203.85	5.40
	4	1376	2.00	-0.24	-1.18	1.376	203.85	5.58
	8	1355	2.83	-0.29	-1.45	1.370	203.85	5.84
	15	1324	3.87	-0.37	-1.84	1.360	203.85	6.24
	30	1295	5.48	-0.44	-2.21	1.351	203.85	6.60
	60	1269	7.75	-0.51	-2.54	1.343	203.85	6.93
	120	1258	10.95	-0.54	-2.68	1.339	203.85	7.07
	240	1244	15.49	-0.57	-2.86	1.335	203.85	7.25
	480	1237	21.91	-0.59	-2.95	1.333	203.85	7.34
	1440	1228	37.95	-0.61	-3.06	1.330	203.85	7.45

Load 15.909 kg 35 LBS

Consolidation Load 5

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/8/2014 8:00	0	1228	0	0.00	0.00	1.330	445.92	7.45
	0.25	1183	0.50	-0.11	-0.57	1.316	445.92	8.03
	0.5	1171	0.71	-0.14	-0.72	1.312	445.92	8.18
	1	1156	1.00	-0.18	-0.91	1.307	445.92	8.37
	2	1136	1.41	-0.23	-1.17	1.301	445.92	8.62
	4	1107	2.00	-0.31	-1.54	1.292	445.92	8.99
	8	1070	2.83	-0.40	-2.01	1.280	445.92	9.46
	15	1021	3.87	-0.53	-2.63	1.265	445.92	10.08
	30	960	5.48	-0.68	-3.40	1.246	445.92	10.86
	60	884	7.75	-0.87	-4.37	1.222	445.92	11.82
	120	832	10.95	-1.01	-5.03	1.206	445.92	12.48
	240	793	15.49	-1.10	-5.52	1.193	445.92	12.98
	480	757	21.91	-1.20	-5.98	1.182	445.92	13.44
	1440	720	37.95	-1.29	-6.45	1.170	445.92	13.91

Load 31.818 kg 70 LBS

Consolidation Load 6

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/9/2014 8:00	0	3198	0	0.00	0.00	1.170	891.84	13.91
	0.25	3150	0.50	-0.12	-0.61	1.155	891.84	14.52
	0.5	3140	0.71	-0.15	-0.74	1.152	891.84	14.64
	1	3126	1.00	-0.18	-0.91	1.148	891.84	14.82
	2	3106	1.41	-0.23	-1.17	1.142	891.84	15.07
	4	3075	2.00	-0.31	-1.56	1.132	891.84	15.47
	8	3028	2.83	-0.43	-2.16	1.117	891.84	16.07
	15	2973	3.87	-0.57	-2.86	1.100	891.84	16.76
	30	2885	5.48	-0.80	-3.98	1.072	891.84	17.88
	60	2760	7.75	-1.11	-5.56	1.033	891.84	19.47
	120	2638	10.95	-1.42	-7.11	0.995	891.84	21.02
	240	2544	15.49	-1.66	-8.31	0.965	891.84	22.21
	480	2475	21.91	-1.84	-9.18	0.944	891.84	23.09
	1440	2466	37.95	-1.86	-9.30	0.941	891.84	23.20

Unload 7.273 kg 16 LBS

Consolidation Unload 1

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/10/2014 8:00	0	2466	0	0.00	0.00	0.941	203.85	-
	0.25	2471	0.50	0.01	0.06	0.942	203.85	-
	0.5	2479	0.71	0.03	0.17	0.945	203.85	-
	1	2489	1.00	0.06	0.29	0.948	203.85	-
	2	2504	1.41	0.10	0.48	0.953	203.85	-
	4	2524	2.00	0.15	0.74	0.959	203.85	-
	8	2550	2.83	0.21	1.07	0.967	203.85	-
	15	2587	3.87	0.31	1.54	0.979	203.85	-
	30	2635	5.48	0.43	2.15	0.994	203.85	-
	60	2687	7.75	0.56	2.81	1.010	203.85	-
	120	2732	10.95	0.68	3.38	1.024	203.85	-
	240	2753	15.49	0.73	3.64	1.031	203.85	-
	480	2763	21.91	0.75	3.77	1.034	203.85	-
	1440	2770	37.95	0.77	3.86	1.036	203.85	-

Unload 1.818 kg 4 LBS

Consolidation Unload 2

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/11/2014 8:00	0	2770	0	0.00	0.00	1.036	50.96	-
	0.25	2790	0.50	0.05	0.25	1.042	50.96	-
	0.5	2795	0.71	0.06	0.32	1.044	50.96	-
	1	2800	1.00	0.08	0.38	1.046	50.96	-
	2	2808	1.41	0.10	0.48	1.048	50.96	-
	4	2820	2.00	0.13	0.64	1.052	50.96	-
	8	2839	2.83	0.18	0.88	1.058	50.96	-
	15	2861	3.87	0.23	1.16	1.065	50.96	-
	30	2900	5.48	0.33	1.65	1.077	50.96	-
	60	2954	7.75	0.47	2.34	1.094	50.96	-
	120	3002	10.95	0.59	2.95	1.109	50.96	-
	240	3070	15.49	0.76	3.81	1.130	50.96	-
	480	3108	21.91	0.86	4.29	1.142	50.96	-
	1440	3147	37.95	0.96	4.79	1.154	50.96	-

Unload 0.455 kg 1 LBS

Consolidation Unload 2

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)
12/12/2014 8:00	0	3147	0	0.00	0.00	1.154	12.74	-
	0.25	3158	0.50	0.03	0.14	1.158	12.74	-
	0.5	3160	0.71	0.03	0.17	1.158	12.74	-
	1	3163	1.00	0.04	0.20	1.159	12.74	-
	2	3168	1.41	0.05	0.27	1.161	12.74	-
	4	3175	2.00	0.07	0.36	1.163	12.74	-
	8	3185	2.83	0.10	0.48	1.166	12.74	-
	15	3198	3.87	0.13	0.65	1.170	12.74	-
	30	3217	5.48	0.18	0.89	1.176	12.74	-
	60	3248	7.75	0.26	1.28	1.186	12.74	-
	120	3291	10.95	0.37	1.83	1.200	12.74	-
	240	3340	15.49	0.49	2.45	1.215	12.74	-
	480	3398	21.91	0.64	3.19	1.233	12.74	-
	1440	3446	37.95	0.76	3.80	1.248	12.74	-

Unload kg LBS

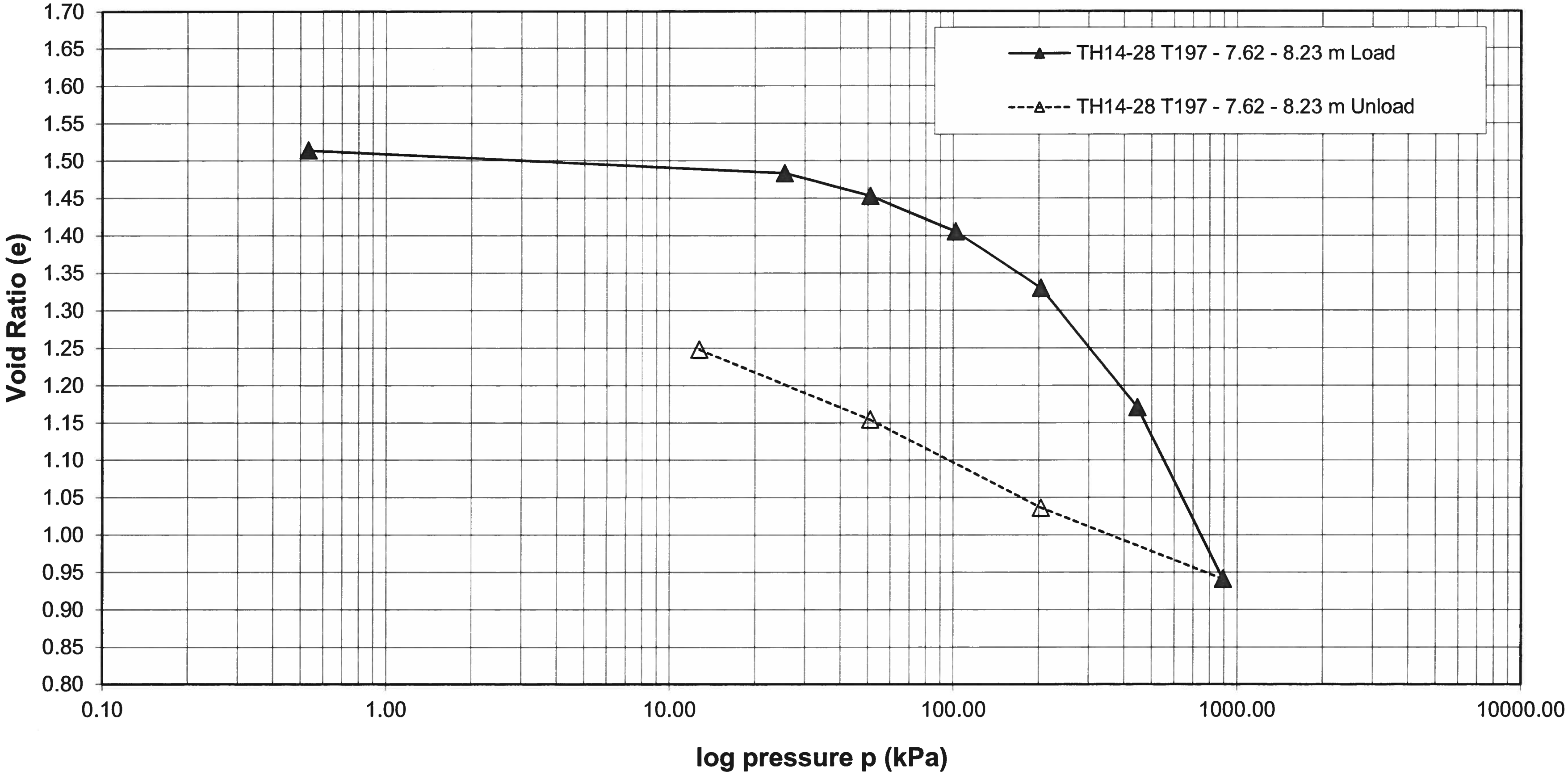
Consolidation Unload 2

Time	Elapsed Time (min)	Normal Dial Reading	Sq. Root Elapsed Time (min)	Deflection Disp. (mm)	Normal Strain %	Void Ratio (mm)	Pressure kPa	Consolidation (%)

Consolidation Results - Void Ratio (e) *versus* log pressure p

Client: Dillon Consulting Ltd.
Project: Waverley Underpass Phase II
Sample: TH14-28 T197 7.62 - 8.23 m

Project No: 60321148
Date: November 28 to December 12, 2014



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MATERIALS LABORATORY
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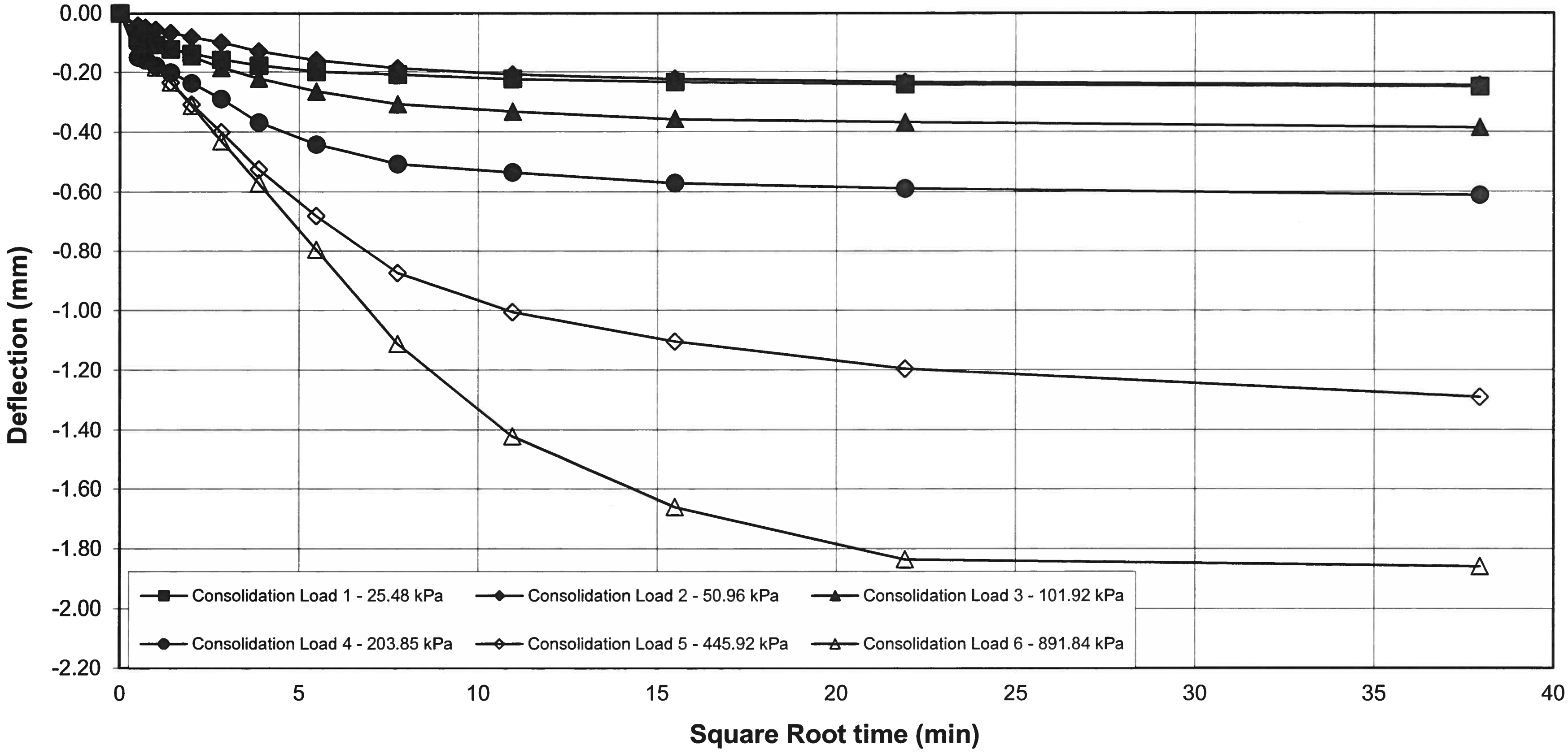
99 Commerce Dr. Wpg. Mb
tel (204) 477-5381 fax (204) 284-2040

Deflection (mm) *versus* Normal Stress (σ)

Client: Dillon Consulting Ltd.
Project: Waverley Underpass Phase II
Sample: TH14-28 T197 7.62 - 8.23 m

Project No: 60321148
Date: November 28 to December 12, 2014



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Memorandum

To	Saba Ibrahim	Page	1
CC			
Subject	Waverly Underpass		
From	Faris Khalil		
Date	December 16, 2014	Project Number	60321148

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Sixty-nine (69) Moisture Content tests.
- Five (5) Atterberg Limits (3 points) tests.
- Twelve (12) Grain Size Distribution (hydrometer method) tests.
- Two (2) Torvane, Pocket Penetrometer, Moisture Content, Bulk Density and Visual Description with Unconfined Compressive Strength, on Shelby tube samples.

If you have any questions, please contact the undersigned.

Sincerely,

Faris Khalil, M.Sc., PMP, P.Eng.
Manager, Geotechnical Engineering

Att.

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
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Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	EManimbao
Sample Number:	Varies	Date Tested:	November 12, 2014

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	**Moisture Content (%)**
TH14-05	G54	0.61 - 0.76 m	**33.0**
-	G56	2.13 - 2.29 m	**37.3**
TH14-06	G59	1.07 - 1.22 m	**28.9**
-	G61	2.59 - 2.74 m	**30.8**
TH14-07	G62	0.76 - 0.91 m	**34.1**
-	G64	1.83 - 1.98 m	**46.7**
TH14-08	G66	0.61 - 0.76 m	**28.3**
-	G67	1.07 - 1.22 m	**33.2**
-	G69	2.44 - 2.59 m	**46.6**
TH14-09	G70	0.61 - 0.76 m	**41.3**
-	G72	1.68 - 1.83 m	**18.3**
TH14-10	G75	1.07 - 1.22 m	**39.0**
-	G77	2.74 - 2.90 m	**50.0**
TH14-11	G78	0.30 - 0.46 m	**20.2**
-	G80	1.37 - 1.52 m	**27.0**
TH14-12	G83	1.22 - 1.37 m	**20.7**
-	G85	2.29 - 2.44 m	**38.8**
TH14-13	G86	0.76 - 0.91 m	**22.2**
-	G88	1.83 - 1.98 m	**37.2**
TH14-14	G90	0.46 - 0.61 m	**24.6**
-	G92	1.07 - 1.22 m	**28.3**
-	G94	1.68 - 1.83 m	**44.3**
TH14-15	G98	0.61 - 0.76 m	**33.3**
-	G100	1.22 - 1.37 m	**38.5**
TH14-16	G104	0.30 - 0.46 m	**27.0**
-	G105	0.76 - 0.91 m	**28.3**
-	G106	1.07 - 1.22 m	**24.0**
-	G108	1.68 - 1.83 m	**25.7**
TH14-17	G112	0.61 - 0.76 m	**25.2**
-	G114	1.22 - 1.37 m	**31.7**
-	G115	1.52 - 1.68 m	**37.1**
TH14-18	G120	1.37 - 1.52 m	**35.7**
-	G121	1.83 - 1.98 m	**23.0**
-	G122	2.21 - 2.36 m	**24.2**
-	G124	2.90 - 3.05 m	**28.9**
TH14-19	G129	0.99 - 1.14 m	**32.5**
-	G131	1.68 - 1.83 m	**40.3**
-	G132	1.91 - 2.06 m	**42.7**

Location	Sample	Depth (m)	**Moisture Content (%)**
TH14-20	G135	0.30 - 0.46 m	**22.2**
-	G137	0.99 - 1.14 m	**33.6**
-	G138	1.52 - 1.68 m	**36.9**
TH14-21	G143	0.76 - 0.91 m	**23.6**
-	G145	1.37 - 1.52 m	**22.1**
-	G146	1.68 - 1.83 m	**38.5**
TH14-22	G149	0.15 - 0.30 m	**25.2**
-	G150	0.61 - 0.76 m	**27.4**
-	G152	1.22 - 1.37 m	**30.3**
-	G153	1.68 - 1.83 m	**30.2**
TH14-23	G156	0.30 - 0.46 m	**35.7**
-	G158	0.91 - 1.07 m	**27.7**
-	G160	1.68 - 1.83 m	**34.1**
TH14-24	G164	0.76 - 0.91 m	**21.5**
-	G166	1.37 - 1.52 m	**29.3**
-	G167	1.68 - 1.83 m	**29.2**
TH14-25	G170	0.46 - 0.61 m	**33.8**
-	G171	0.76 - 0.91 m	**25.8**
-	G173	1.37 - 1.52 m	**39.0**
TH14-26	G178	0.76 - 0.91 m	**33.7**
-	G180	1.68 - 1.83 m	**39.4**
TH14-27	G183	0.30 - 0.46 m	**6.5**
-	G184	0.91 - 1.07 m	**27.2**
-	G185	1.22 - 1.37 m	**34.6**
TH14-28	G188	0.61 - 0.76 m	**27.6**
-	G190	1.22 - 1.37 m	**30.3**
-	G192	2.59 - 2.74 m	**25.1**
-	G196	7.01 - 7.16 m	**50.0**
-	G198	9.45 - 9.60 m	**50.6**
-	S202	12.95 - 13.11 m	**14.3**
-	G203	13.72 - 13.87 m	**14.7**

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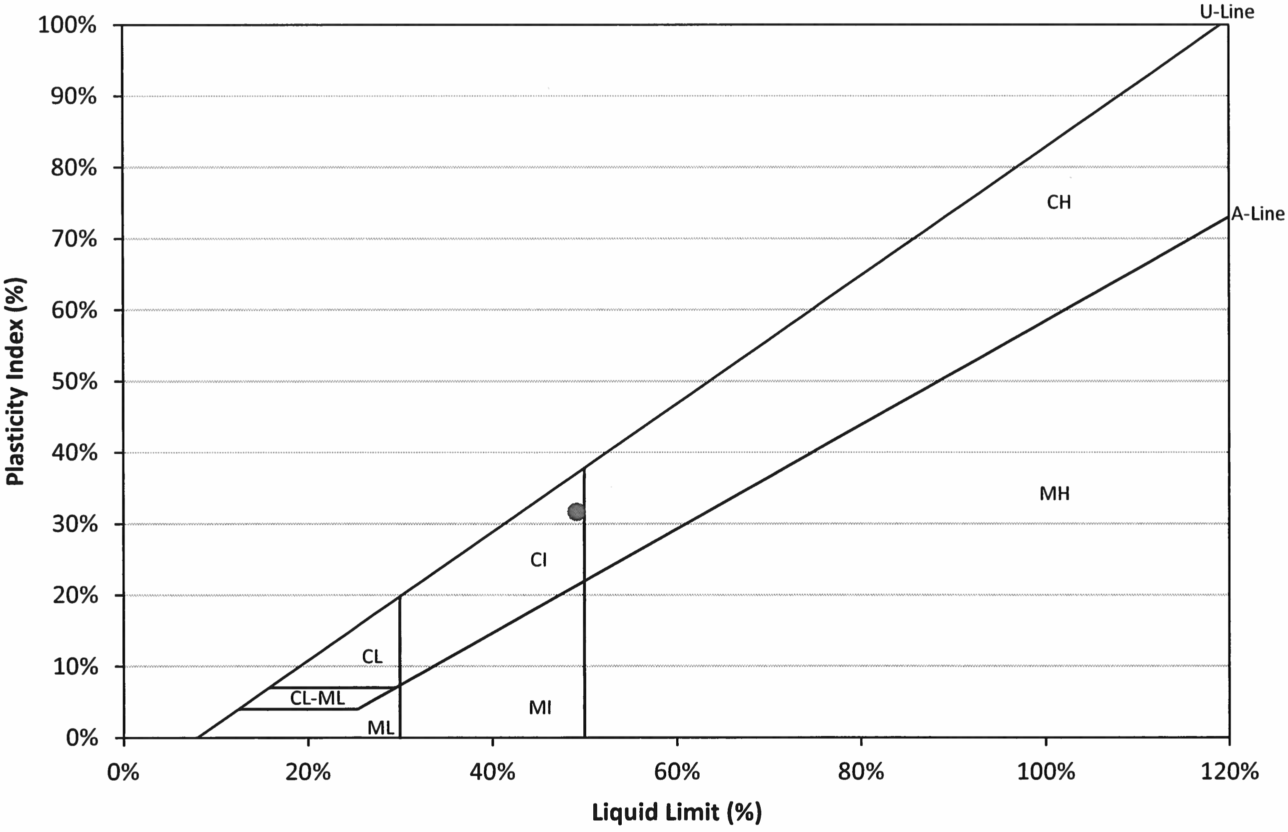
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-07	Sample Date:	November 1, 2014
Sample Depth:	0.76 m	Lab Technician:	EManimbao
Sample Number:	G62	Date Tested:	November 26, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	33	24	21
Wet Sample (g)	8.3	9.7	10.7
Dry Sample (g)	5.6	6.5	7.2
Water Content (%)	48.5%	49.4%	49.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.9	6.7
Dry Sample (g)	6.7	5.7
Water Content (%)	17.2%	17.9%



Liquid Limit (%): 49.2%	Plastic Limit (%): 17.6%	Plasticity Index (%): 31.6%

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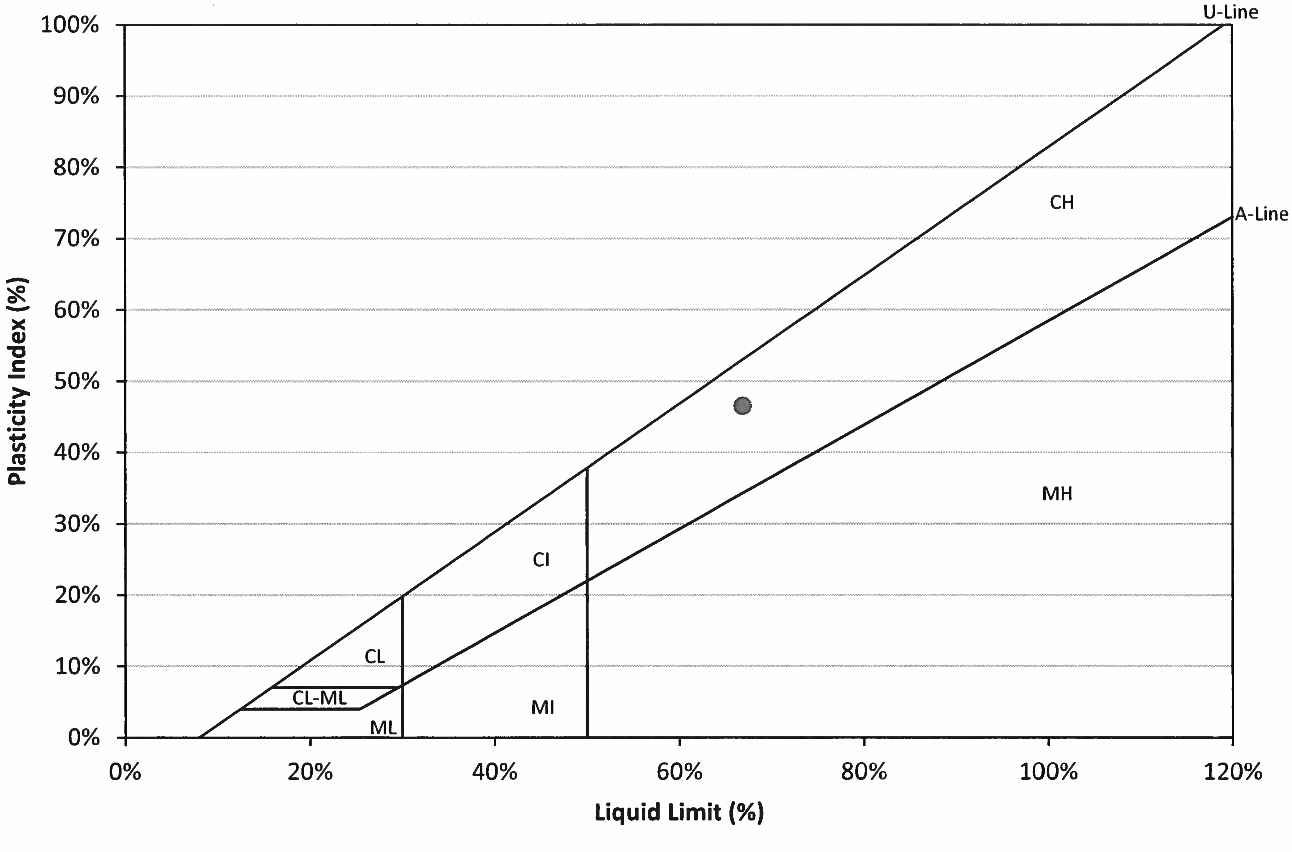
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-16	Sample Date:	November 1, 2014
Sample Depth:	0.76 m	Lab Technician:	EManimbao
Sample Number:	G105	Date Tested:	November 26, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	27	17
Wet Sample (g)	8.8	8.8	10.0
Dry Sample (g)	5.3	5.3	5.9
Water Content (%)	64.3%	65.7%	70.0%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.4	6.4
Dry Sample (g)	5.4	5.3
Water Content (%)	20.3%	20.6%



Liquid Limit (%): 66.9%	Plastic Limit (%): 20.4%	Plasticity Index (%): 46.5%

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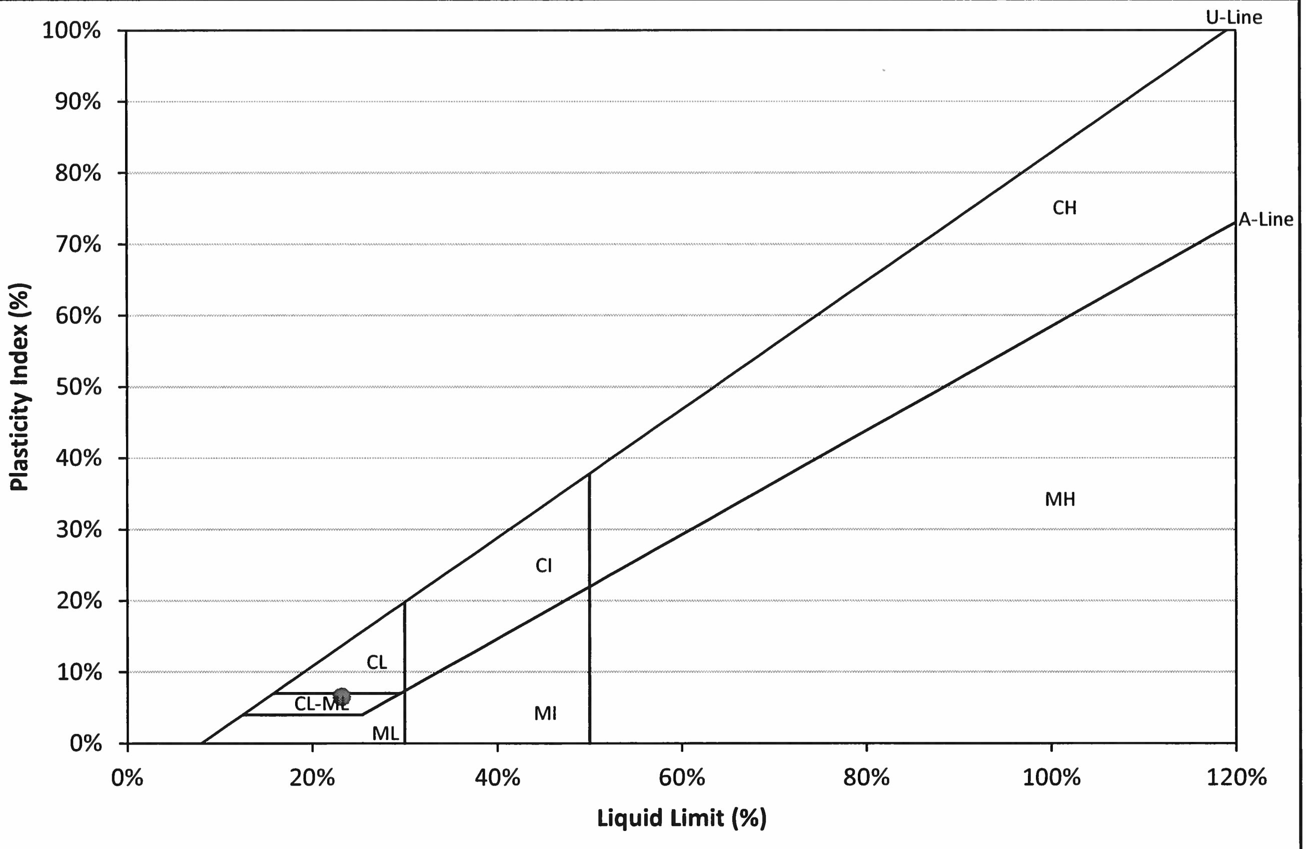
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-18	Sample Date:	November 1, 2014
Sample Depth:	1.83 m	Lab Technician:	EManimbao
Sample Number:	G121	Date Tested:	November 26, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	27	21	17
Wet Sample (g)	12.9	12.5	9.9
Dry Sample (g)	10.5	10.0	7.9
Water Content (%)	22.8%	24.5%	25.4%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.3	6.0
Dry Sample (g)	6.3	5.2
Water Content (%)	16.9%	16.6%



Liquid Limit (%): 23.2%	Plastic Limit (%): 16.7%	Plasticity Index (%): 6.5%

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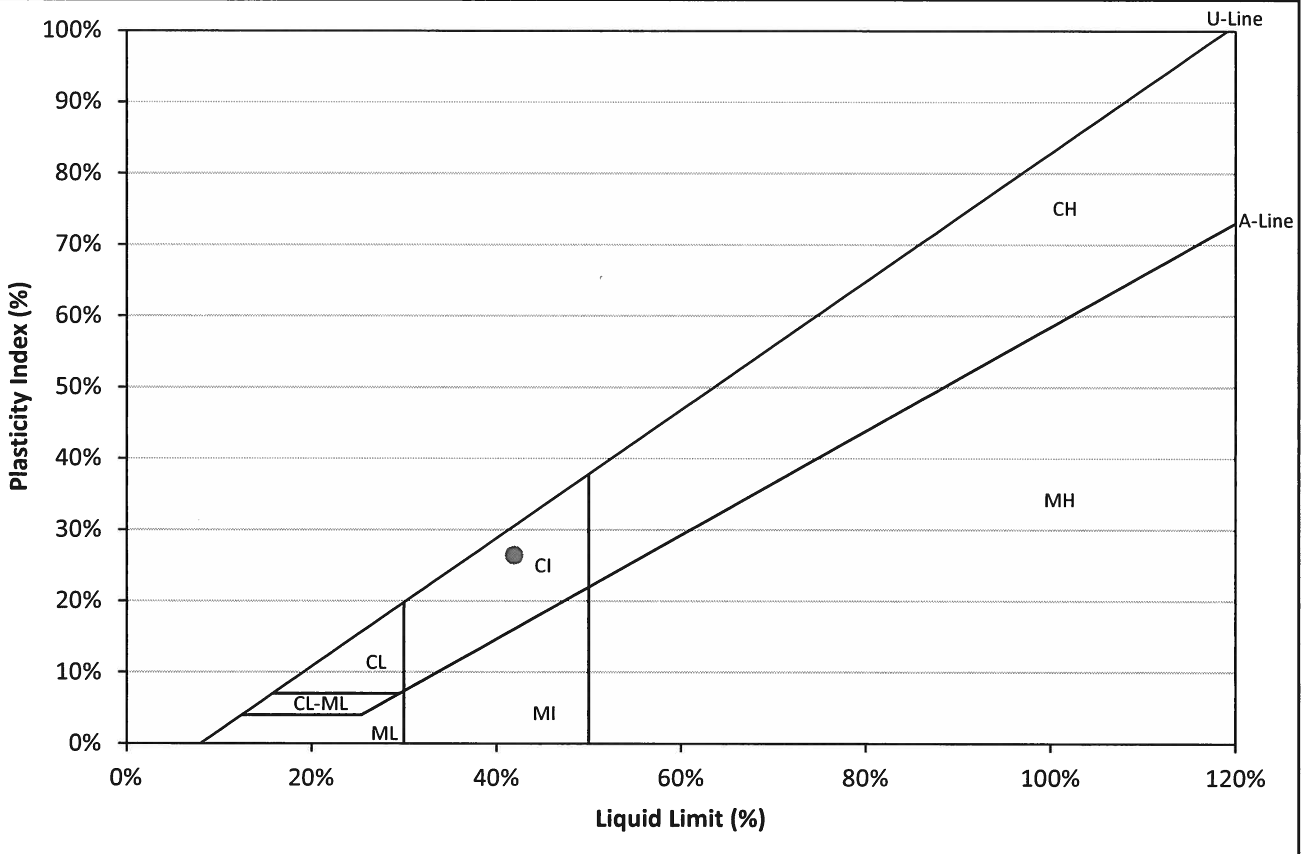
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-21	Sample Date:	November 1, 2014
Sample Depth:	0.76 m	Lab Technician:	EManimbao
Sample Number:	G143	Date Tested:	November 26, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	33	26	22
Wet Sample (g)	7.4	8.5	8.3
Dry Sample (g)	5.3	6.0	5.8
Water Content (%)	39.0%	41.6%	43.1%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.6	6.2
Dry Sample (g)	5.7	5.4
Water Content (%)	15.9%	15.1%



Liquid Limit (%): 41.9%	Plastic Limit (%): 15.5%	Plasticity Index (%): 26.4%

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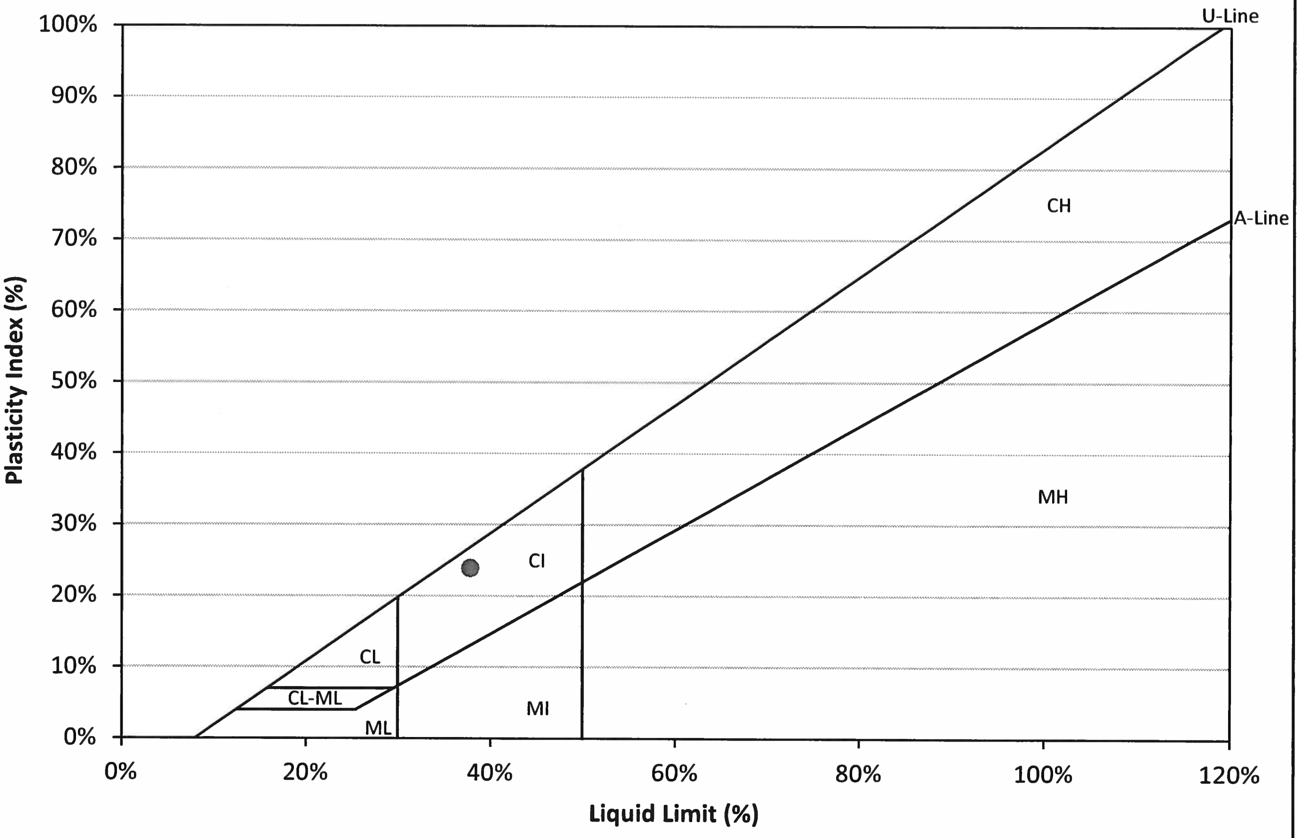
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH14-25	Sample Date:	November 1, 2014
Sample Depth:	0.76 m	Lab Technician:	EManimbao
Sample Number:	G171	Date Tested:	November 26, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	27	22	19
Wet Sample (g)	8.1	10.5	8.1
Dry Sample (g)	5.9	7.6	5.9
Water Content (%)	37.8%	38.1%	38.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	8.1	8.1
Dry Sample (g)	7.1	7.1
Water Content (%)	13.5%	14.4%



Liquid Limit (%): 37.8%	Plastic Limit (%): 13.9%	Plasticity Index (%): 23.9%

GRAIN SIZE DISTRIBUTION
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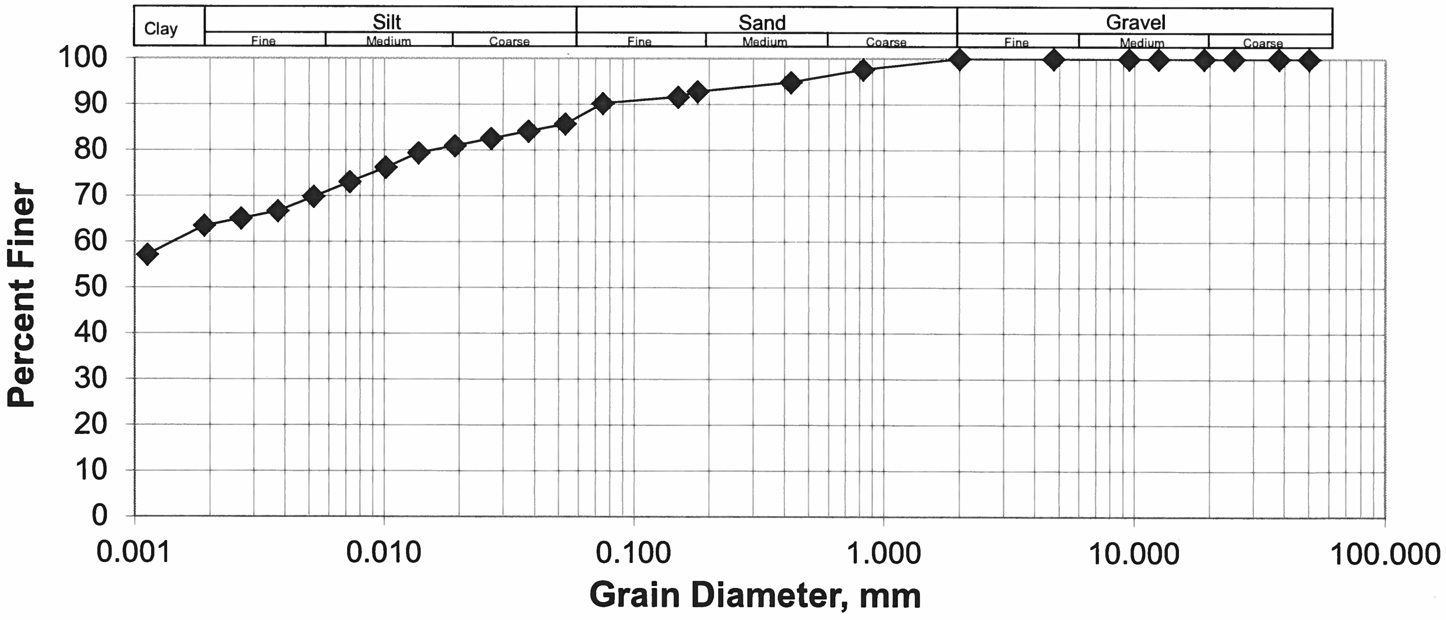
Job No.:	60321148	Hole No.:	14-05
Client:	Dillon Consulting	Sample No.:	G54
Project :	Waverley Underpass Phase II	Depth:	0.61 m
Date Tested:	20-Nov-14	Date Sampled:	1-Nov-14
Tested By:	MLotecki	Sampled By:	AECOM (SIbrahim)

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	90.2
38.0	100.0	0.83	97.6	0.0531	85.7
25.0	100.0	0.43	94.8	0.0379	84.1
19.0	100.0	0.18	92.8	0.0270	82.5
12.5	100.0	0.15	91.6	0.0192	80.9
9.5	100.0	0.075	90.2	0.0137	79.3
4.75	100.0			0.0102	76.2
2.00	100.0			0.0073	73.0
				0.0052	69.8
				0.0037	66.6
				0.0027	65.1
				0.0019	63.5
				0.0011	57.1

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**23.4%**
Sand	**12.9%**	**Clay**	**63.7%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION
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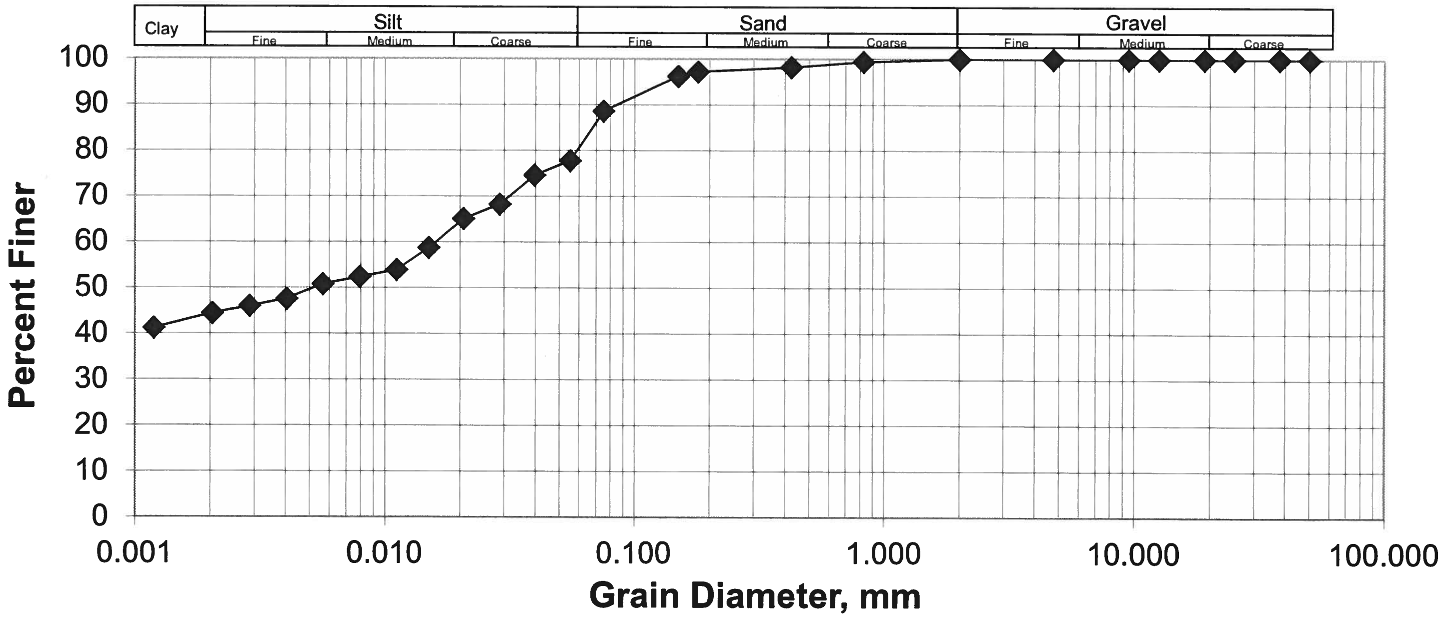
Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 20-Nov-14
Tested By: MLotecki

Hole No.: 14-07
Sample No.: G62
Depth: 0.76 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	88.6
38.0	100.0	0.83	99.4	0.0552	77.8
25.0	100.0	0.43	98.2	0.0396	74.6
19.0	100.0	0.18	97.2	0.0288	68.2
12.5	100.0	0.15	96.2	0.0207	65.1
9.5	100.0	0.075	88.6	0.0150	58.7
4.75	100.0			0.0112	53.9
2.00	100.0			0.0079	52.3
				0.0056	50.8
				0.0040	47.6
				0.0029	46.0
				0.0020	44.4
				0.0012	41.2

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**36.1%**
Sand	**19.6%**	**Clay**	**44.2%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

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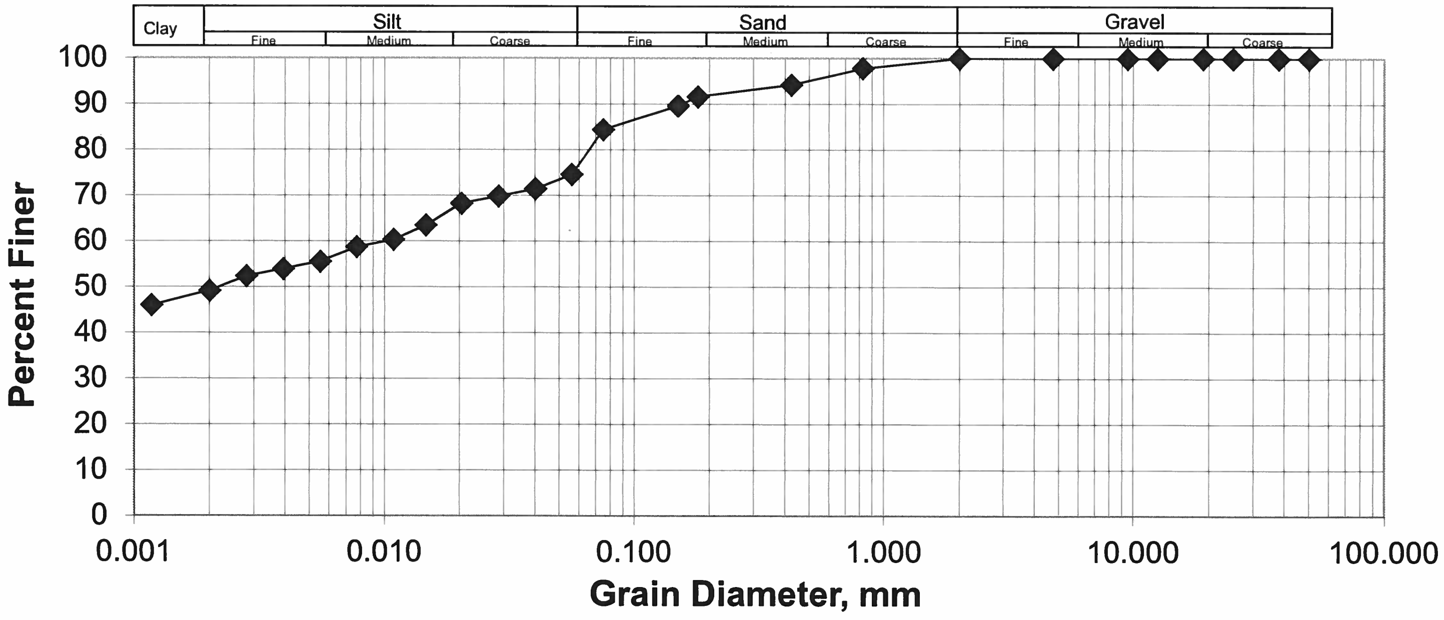
Job No.:	60321148	Hole No.:	14-08
Client:	Dillon Consulting	Sample No.:	G66
Project :	Waverley Underpass Phase II	Depth:	0.61 m
Date Tested:	20-Nov-14	Date Sampled:	1-Nov-14
Tested By:	MLotecki	Sampled By:	AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	84.4
38.0	100.0	0.83	97.8	0.0561	74.6
25.0	100.0	0.43	94.2	0.0402	71.4
19.0	100.0	0.18	91.6	0.0286	69.8
12.5	100.0	0.15	89.6	0.0204	68.2
9.5	100.0	0.075	84.4	0.0147	63.5
4.75	100.0			0.0109	60.3
2.00	100.0			0.0077	58.7
				0.0055	55.5
				0.0039	53.9
				0.0028	52.3
				0.0020	49.2
				0.0012	46.0

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**27.5%**
Sand	**23.4%**	**Clay**	**49.1%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

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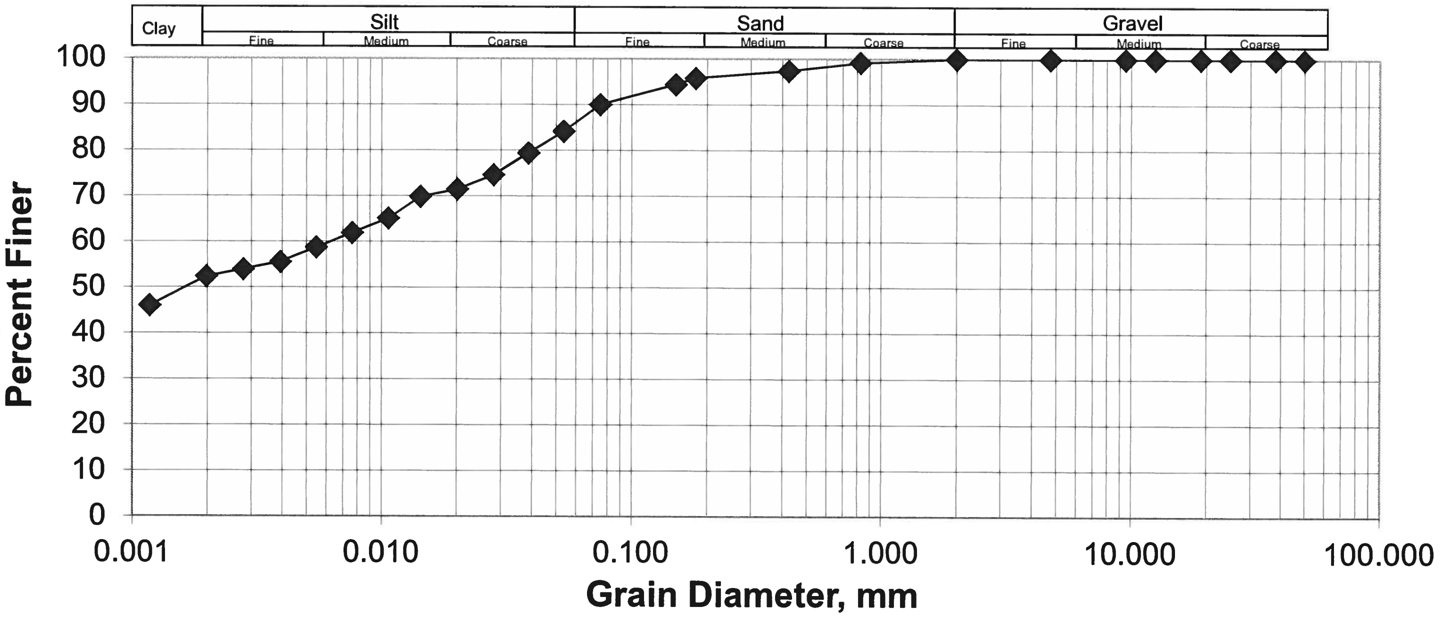
Job No.:	60321148	Hole No.:	14-10
Client:	Dillon Consulting	Sample No.:	G75
Project :	Waverley Underpass Phase II	Depth:	1.07 m
Date Tested:	20-Nov-14	Date Sampled:	1-Nov-14
Tested By:	MLotecki	Sampled By:	AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	90.0
38.0	100.0	0.83	99.2	0.0536	84.1
25.0	100.0	0.43	97.4	0.0388	79.3
19.0	100.0	0.18	95.8	0.0280	74.6
12.5	100.0	0.15	94.4	0.0201	71.4
9.5	100.0	0.075	90.0	0.0143	69.8
4.75	100.0			0.0107	65.1
2.00	100.0			0.0076	61.9
				0.0055	58.7
				0.0039	55.5
				0.0028	53.9
				0.0020	52.3
				0.0012	46.0

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**33.5%**
Sand	**14.1%**	**Clay**	**52.4%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

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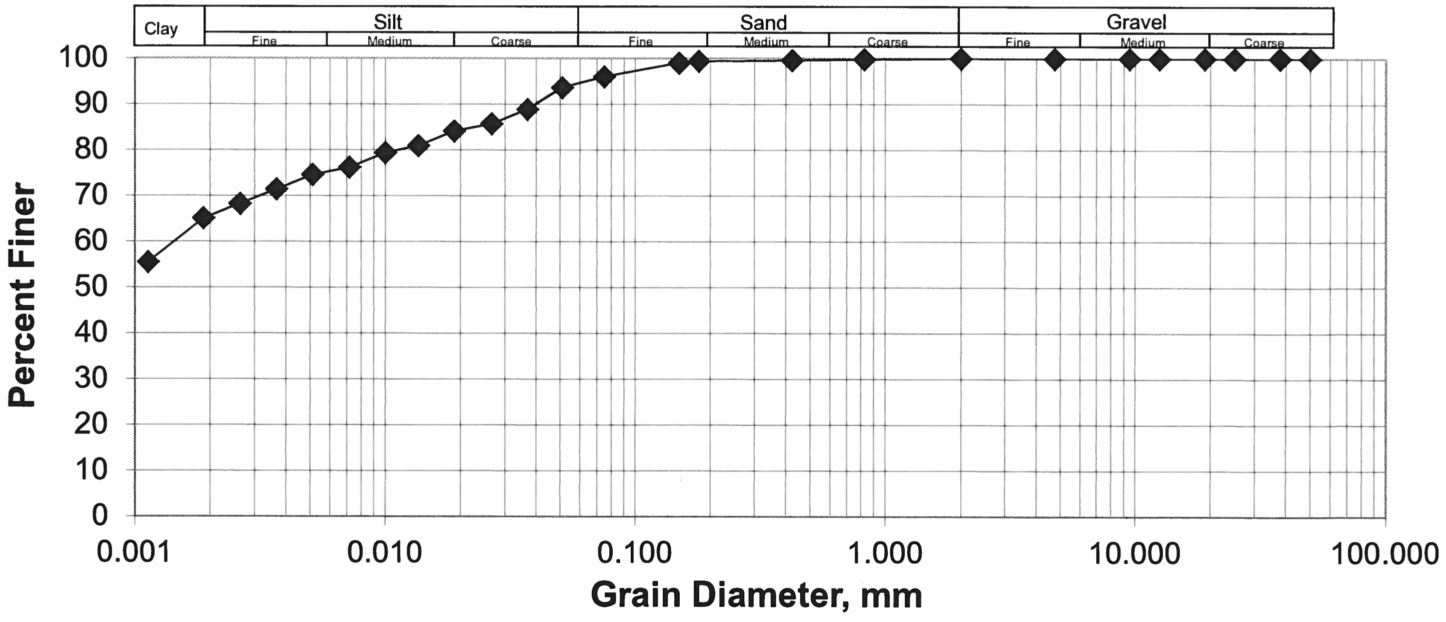
Job No.:	60321148	Hole No.:	14-16
Client:	Dillon Consulting	Sample No.:	G105
Project :	Waverley Underpass Phase II	Depth:	0.76 m
Date Tested:	20-Nov-14	Date Sampled:	1-Nov-14
Tested By:	MLotecki	Sampled By:	AECOM (SIbrahim)

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	96.0
38.0	100.0	0.83	99.8	0.0510	93.6
25.0	100.0	0.43	99.6	0.0370	88.9
19.0	100.0	0.18	99.4	0.0266	85.7
12.5	100.0	0.15	99.0	0.0189	84.1
9.5	100.0	0.075	96.0	0.0136	80.9
4.75	100.0			0.0100	79.3
2.00	100.0			0.0072	76.2
				0.0051	74.6
				0.0037	71.4
				0.0026	68.2
				0.0019	65.1
				0.0011	55.5

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**29.0%**
Sand	**5.5%**	**Clay**	**65.5%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION

(ASTM D422-63)

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MATERIALS LABORATORY
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99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
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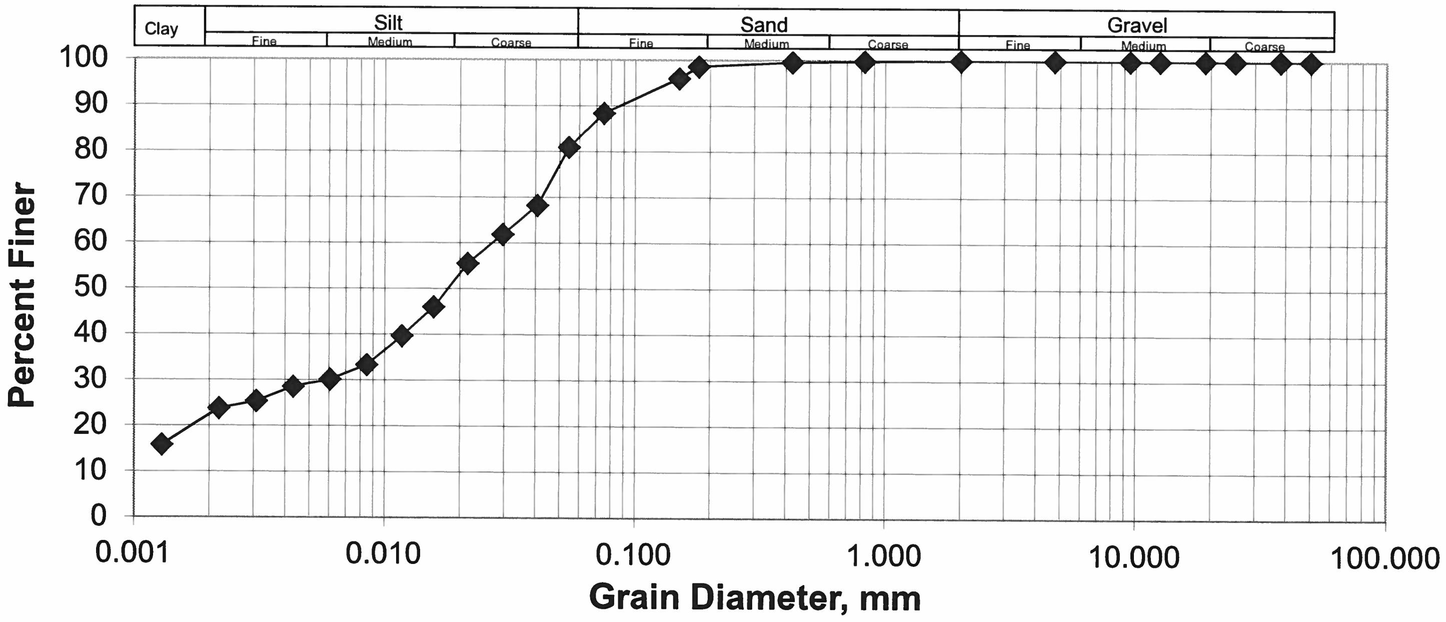
Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 20-Nov-14
Tested By: MLotecki

Hole No.: 14-18
Sample No.: G121
Depth: 1.83 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (MLotecki)

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	88.4
38.0	100.0	0.83	99.8	0.0544	80.9
25.0	100.0	0.43	99.6	0.0408	68.2
19.0	100.0	0.18	98.6	0.0296	61.9
12.5	100.0	0.15	96.0	0.0215	55.5
9.5	100.0	0.075	88.4	0.0157	46.0
4.75	100.0			0.0118	39.6
2.00	100.0			0.0085	33.3
				0.0061	30.1
				0.0043	28.5
				0.0031	25.3
				0.0022	23.8
				0.0013	15.8

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**60.9%**
Sand	**17.0%**	**Clay**	**22.1%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

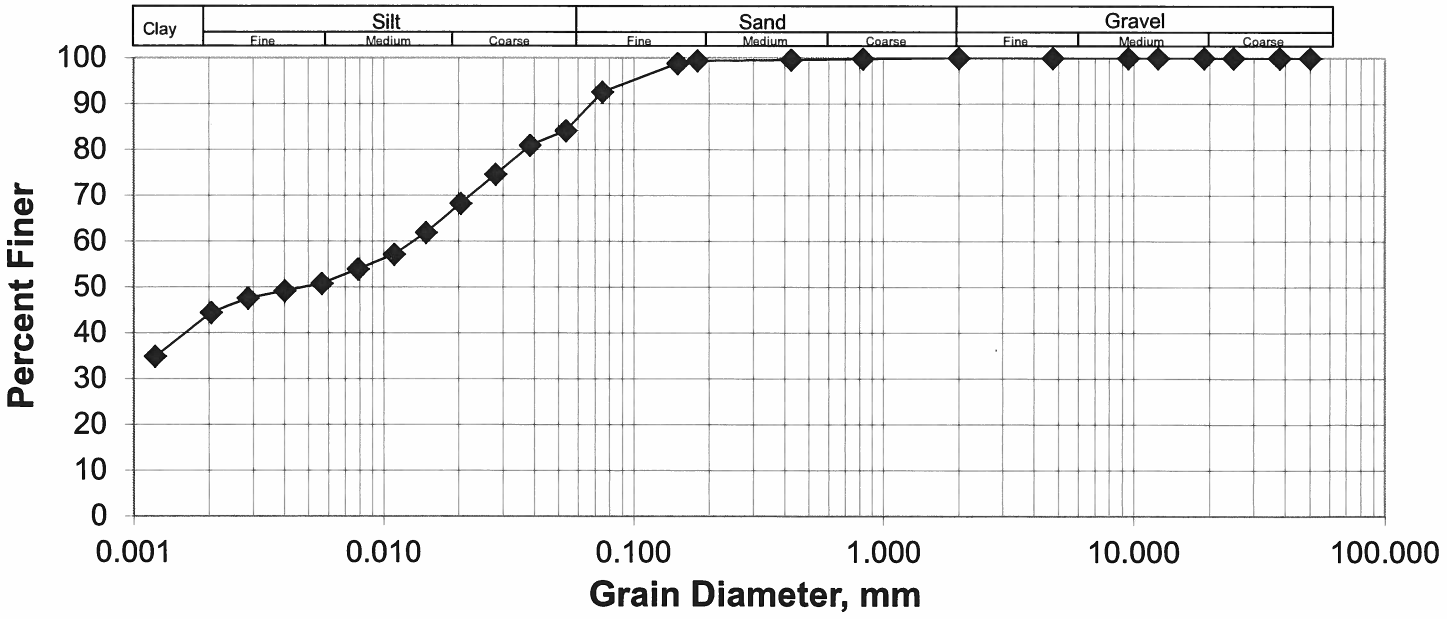
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 20-Nov-14
Tested By: MLotecki

Hole No.: 14-21
Sample No.: G143
Depth: 0.76 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	92.6
38.0	100.0	0.83	99.8	0.0536	84.1
25.0	100.0	0.43	99.6	0.0385	80.9
19.0	100.0	0.18	99.4	0.0280	74.6
12.5	100.0	0.15	98.8	0.0204	68.2
9.5	100.0	0.075	92.6	0.0148	61.9
4.75	100.0			0.0110	57.1
2.00	100.0			0.0079	53.9
				0.0056	50.8
				0.0040	49.2
				0.0029	47.6
				0.0020	44.4
				0.0012	34.9

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**42.8%**
Sand	**13.3%**	**Clay**	**43.9%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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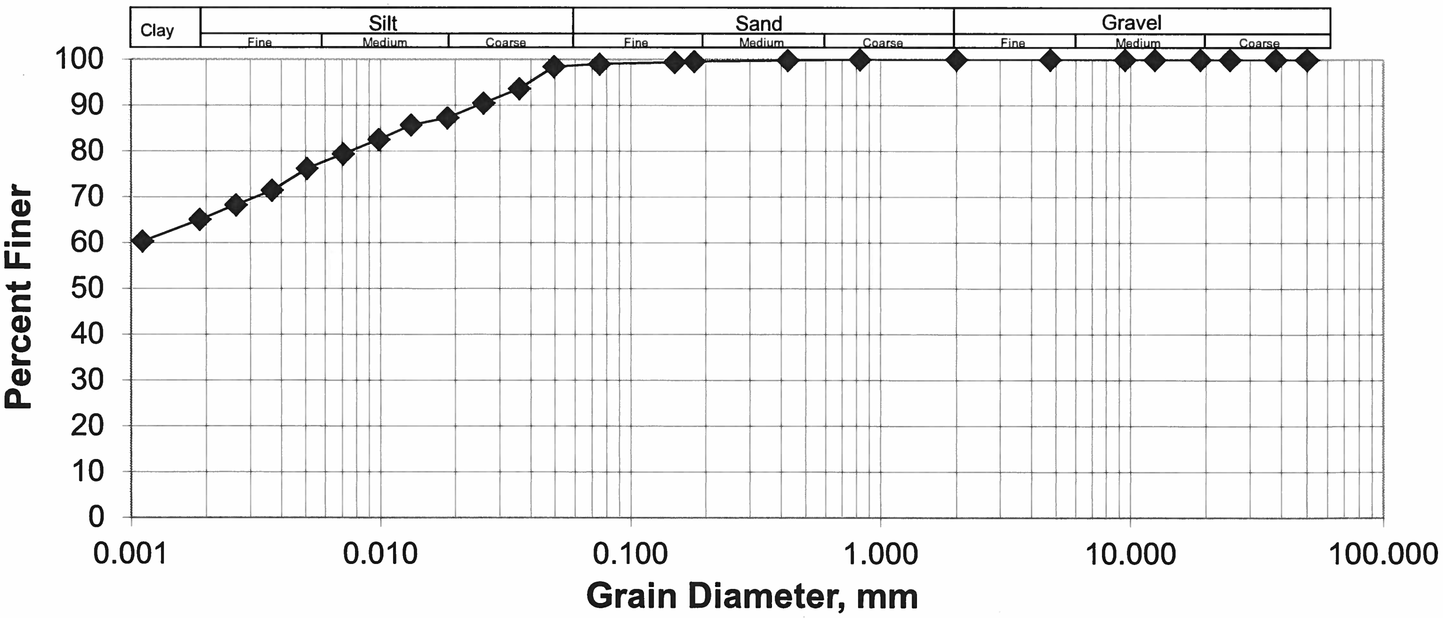
Job No.:	60321148	Hole No.:	14-22
Client:	Dillon Consulting	Sample No.:	G150
Project :	Waverley Underpass Phase II	Depth:	0.61 m
Date Tested:	20-Nov-14	Date Sampled:	1-Nov-14
Tested By:	MLotecki	Sampled By:	AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	99.0
38.0	100.0	0.83	100.0	0.0496	98.4
25.0	100.0	0.43	99.8	0.0360	93.6
19.0	100.0	0.18	99.6	0.0259	90.5
12.5	100.0	0.15	99.4	0.0186	87.3
9.5	100.0	0.075	99.0	0.0133	85.7
4.75	100.0			0.0099	82.5
2.00	100.0			0.0071	79.3
				0.0051	76.2
				0.0037	71.4
				0.0026	68.2
				0.0019	65.1
				0.0011	60.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**33.1%**
Sand	**1.4%**	**Clay**	**65.5%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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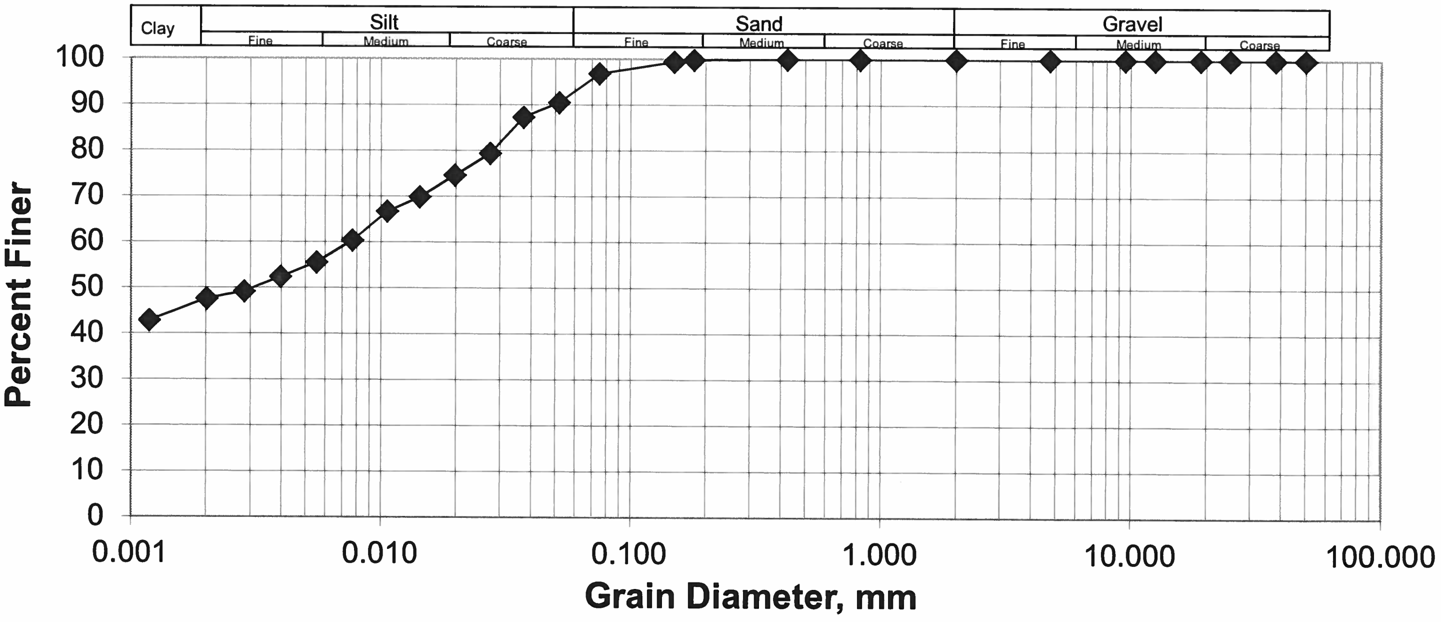
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 20-Nov-14
Tested By: MLotecki

Hole No.: 14-24
Sample No.: G164
Depth: 0.76 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	96.8
38.0	100.0	0.83	100.0	0.0518	90.5
25.0	100.0	0.43	100.0	0.0373	87.3
19.0	100.0	0.18	99.8	0.0274	79.3
12.5	100.0	0.15	99.4	0.0198	74.6
9.5	100.0	0.075	96.8	0.0143	69.8
4.75	100.0			0.0106	66.6
2.00	100.0			0.0077	60.3
				0.0055	55.5
				0.0040	52.3
				0.0028	49.2
				0.0020	47.6
				0.0012	42.8

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**45.2%**
Sand	**7.3%**	**Clay**	**47.5%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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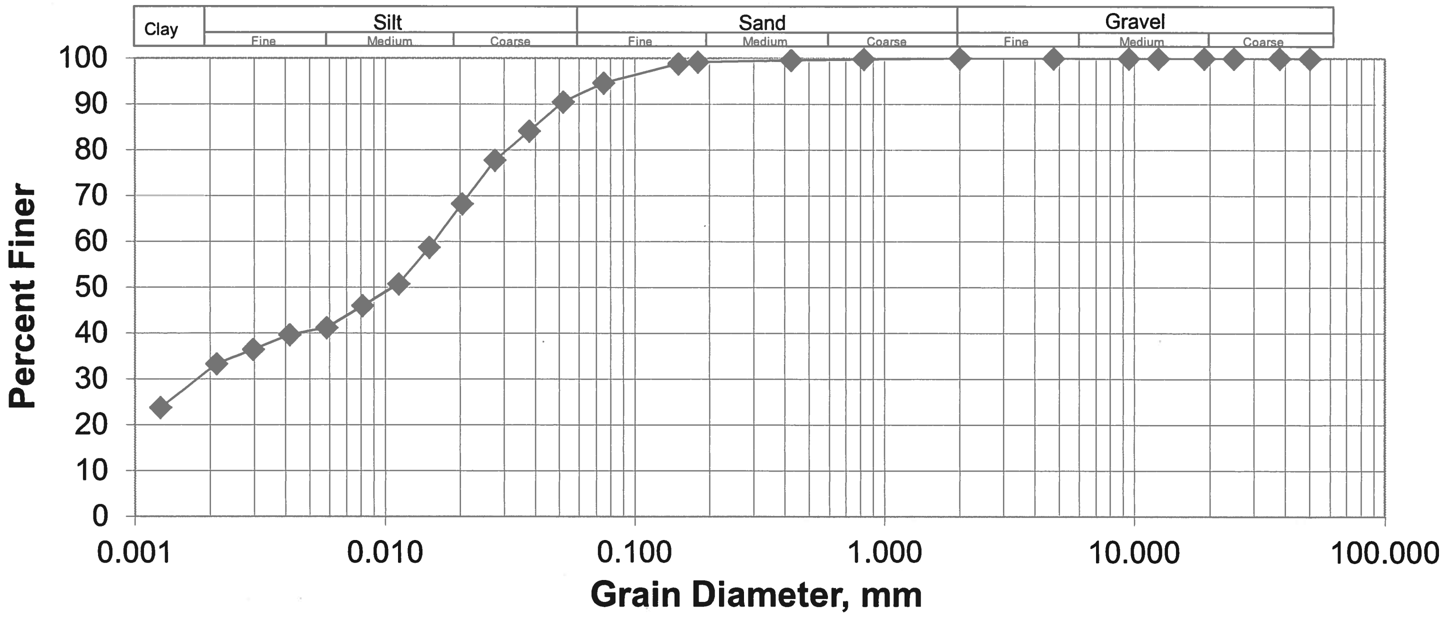
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 20-Nov-14
Tested By: MLotecki

Hole No.: 14-25
Sample No.: G171
Depth: 0.76 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	94.6
38.0	100.0	0.83	99.8	0.0518	90.5
25.0	100.0	0.43	99.6	0.0379	84.1
19.0	100.0	0.18	99.2	0.0276	77.8
12.5	100.0	0.15	98.8	0.0204	68.2
9.5	100.0	0.075	94.6	0.0150	58.7
4.75	100.0			0.0113	50.8
2.00	100.0			0.0081	46.0
				0.0058	41.2
				0.0042	39.6
				0.0030	36.5
				0.0021	33.3
				0.0013	23.8

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**60.0%**
Sand	**8.1%**	**Clay**	**31.9%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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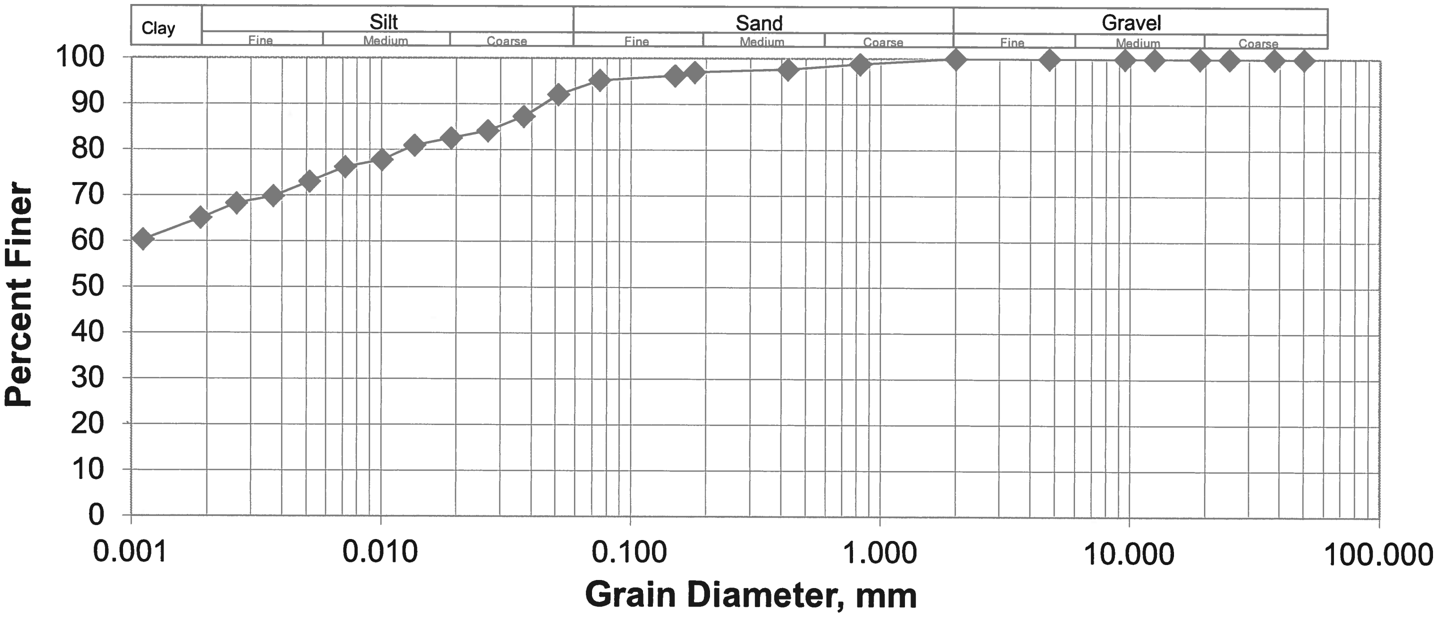
Job No.: 60321148
Client: Dillon Consulting
Project : Waverley Underpass Phase II
Date Tested: 20-Nov-14
Tested By: MLotecki

Hole No.: 14-27
Sample No.: G184
Depth: 0.91 m
Date Sampled: 1-Nov-14
Sampled By: AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	95.2
38.0	100.0	0.83	98.8	0.0514	92.1
25.0	100.0	0.43	97.6	0.0373	87.3
19.0	100.0	0.18	97.0	0.0268	84.1
12.5	100.0	0.15	96.2	0.0191	82.5
9.5	100.0	0.075	95.2	0.0136	80.9
4.75	100.0			0.0101	77.8
2.00	100.0			0.0072	76.2
				0.0052	73.0
				0.0037	69.8
				0.0026	68.2
				0.0019	65.1
				0.0011	60.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**27.7%**
Sand	**6.8%**	**Clay**	**65.5%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

GRAIN SIZE DISTRIBUTION

(ASTM D422-63)

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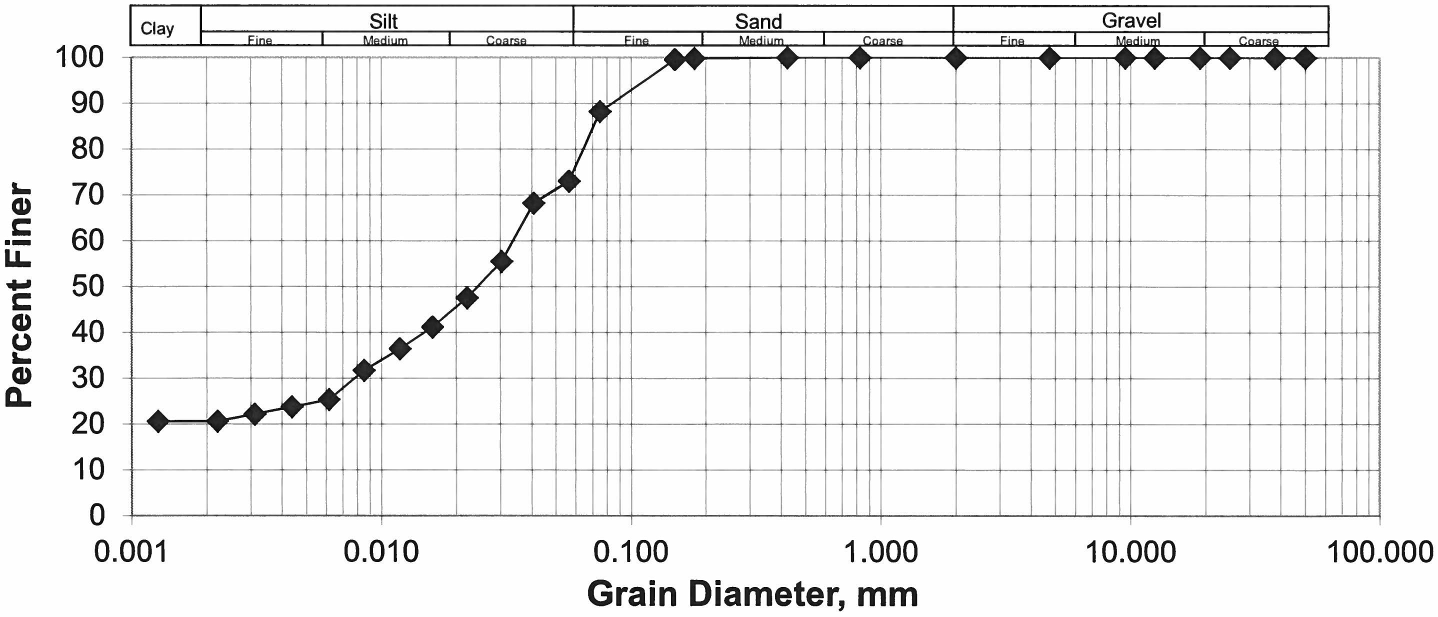
Job No.:	60321148	Hole No.:	14-28
Client:	Dillon Consulting	Sample No.:	G192
Project :	Waverley Underpass Phase II	Depth:	2.59 m
Date Tested:	20-Nov-14	Date Sampled:	1-Nov-14
Tested By:	MLotecki	Sampled By:	AECOM (SIbrahim)

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	2.00	100.0	0.0750	88.2
38.0	100.0	0.83	100.0	0.0565	73.0
25.0	100.0	0.43	100.0	0.0408	68.2
19.0	100.0	0.18	99.8	0.0304	55.5
12.5	100.0	0.15	99.6	0.0221	47.6
9.5	100.0	0.075	88.2	0.0160	41.2
4.75	100.0			0.0119	36.5
2.00	100.0			0.0085	31.7
				0.0062	25.3
				0.0044	23.8
				0.0031	22.2
				0.0022	20.6
				0.0013	20.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**55.3%**
Sand	**24.1%**	**Clay**	**20.6%**

** Note: Soil Classification based on Grain Size from Canadian Foundation Engineering Manual, 3rd edition (1992).

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-28**
SAMPLE NO.:	**T194**
SAMPLE DEPTH:	4.57 - 5.18 m
DATE TESTED:	28-Nov-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.70
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**68.7**
Undrained Shear Strength (ksf)	1.43
POCKET PENETROMETER	
Reading - Qu (tsf)	1.50
Undrained Shear Strength (kPa)	**71.8**
Reading - Qu (tsf)	1.75
Undrained Shear Strength (kPa)	**83.8**
Reading - Qu (tsf)	2.00
Undrained Shear Strength (kPa)	**95.8**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**70.6**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**35.3**
Undrained Shear Strength (ksf)	**0.737**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	358.5
Wt. Sample dry + tare (g)	241.1
Wt. Tare (g)	8.3
Moisture Content %	**50.4**
BULK DENSITY	
Sample Wt. (g)	1068.1
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.18
Diameter 3 (cm)	7.23
Avg. Diameter (cm)	**7.20**
Length 1 (cm)	15.36
Length 2 (cm)	15.35
Length 3 (cm)	15.36
Avg. Length (cm)	**15.36**
Volume (cm^3)	625.8
Moisture content (%)	50.4
Bulk Density (g/cm^3)	1.707
Bulk Density (kN/m^3)	**16.7**
Bulk Density (pcf)	**106.6**
Dry Density (kN/m^3)	**11.13**

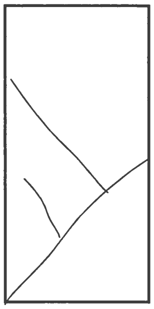
AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-28
SAMPLE NO.:	T194
SAMPLE DEPTH:	4.57 - 5.18 m
SAMPLE DATE:	February, 2014
TEST DATE:	28-Nov-14

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, trace sulphate lenses, brown, moist, firm,	
int. - high plasticity,	
MOISTURE CONTENT:	50.4

SAMPLE DIAM.(Do):	72.03	**(mm)**	**INITIAL AREA, Ao:**	4075.3	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.57	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.13	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.84	**(0.5<R<2 % / minute)**



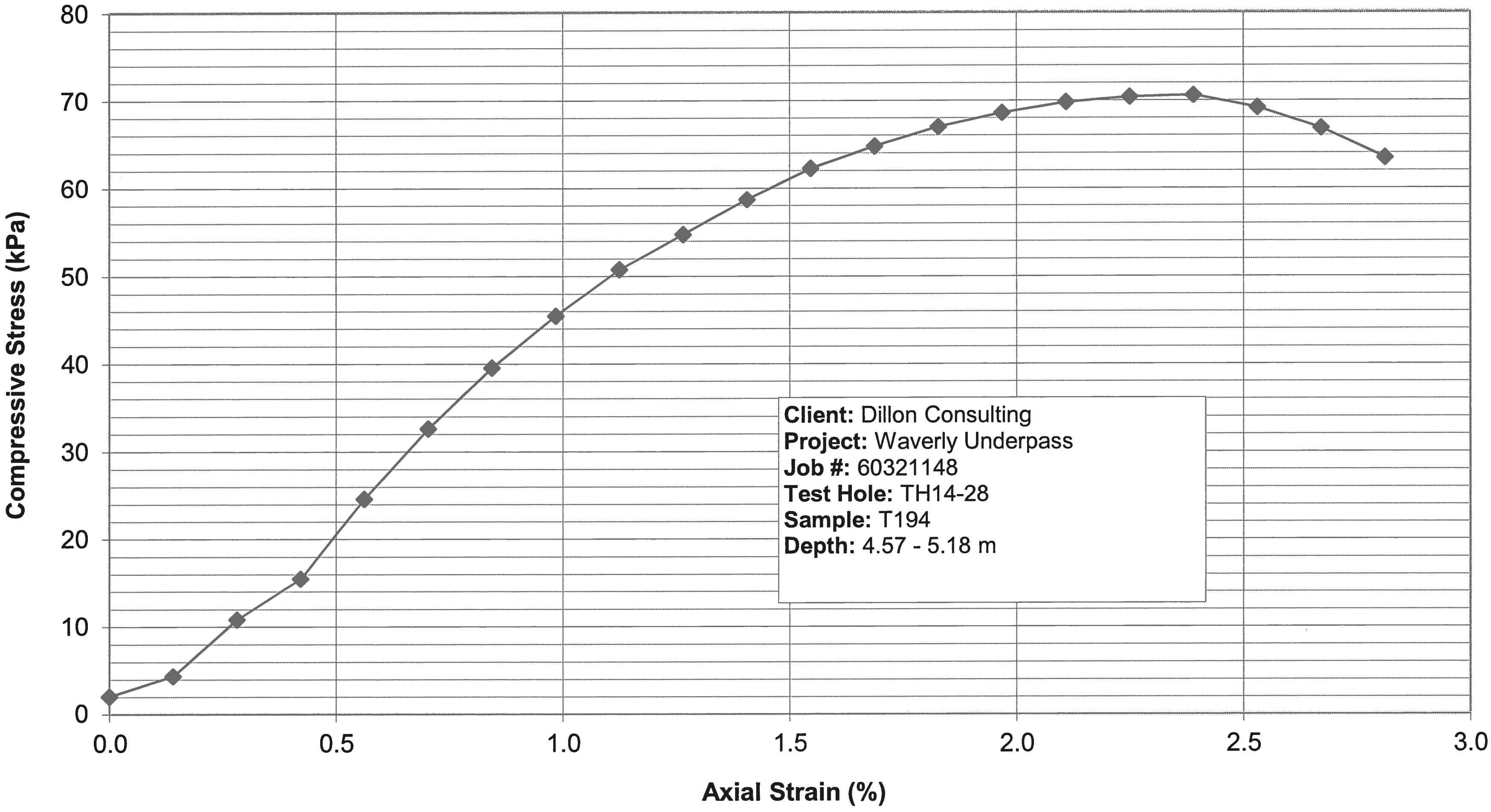
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_C**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0002	0.00	6.32	1.87	0.30	0.043	2.0
0.02	0.0004	0.14	6.33	4.03	0.64	0.092	4.4
0.03	0.0011	0.28	6.33	9.93	1.57	0.226	10.8
0.03	0.0015	0.42	6.34	14.24	2.25	0.323	15.5
0.04	0.0024	0.56	6.35	22.68	3.57	0.514	24.6
0.05	0.0032	0.70	6.36	30.08	4.73	0.681	32.6
0.06	0.0039	0.84	6.37	36.54	5.74	0.826	39.6
0.07	0.0045	0.98	6.38	42.07	6.59	0.950	45.5
0.08	0.0050	1.12	6.39	47.04	7.36	1.060	50.8
0.09	0.0054	1.27	6.40	50.79	7.94	1.143	54.7
0.09	0.0058	1.41	6.41	54.53	8.51	1.226	58.7
0.10	0.0062	1.55	6.42	57.91	9.03	1.300	62.2
0.11	0.0064	1.69	6.43	60.34	9.39	1.352	64.8
0.12	0.0067	1.83	6.43	62.50	9.71	1.399	67.0
0.13	0.0068	1.97	6.44	64.09	9.95	1.432	68.6
0.14	0.0070	2.11	6.45	65.31	10.12	1.457	69.8
0.14	0.0070	2.25	6.46	65.96	10.21	1.470	70.4
0.15	0.0071	2.39	6.47	66.25	10.24	1.474	70.6
0.16	0.0069	2.53	6.48	65.03	10.03	1.445	69.2
0.17	0.0067	2.67	6.49	62.87	9.69	1.395	66.8
0.18	0.0064	2.81	6.50	59.78	9.20	1.324	63.4

UNCONFINED COMPRESSIVE STRENGTH, q_u: (based on maximum q_u value)	70.58 1.474	kPa ksf
UNDRAINED SHEAR STRENGTH, S_u: (based on maximum q_u value)	35.29 0.737	kPa ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-28
Sample: T194
Depth: 4.57 - 5.18 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-28**
SAMPLE NO.:	**T197**
SAMPLE DEPTH:	7.62 - 8.23 m
DATE TESTED:	28-Nov-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.60
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**58.8**
Undrained Shear Strength (ksf)	1.23
POCKET PENETROMETER	
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
Reading - Qu (tsf)	0.75
Undrained Shear Strength (kPa)	**35.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**60.1**
Unconfined compressive strength (ksf)	**1.3**
Undrained Shear Strength (kPa)	**30.1**
Undrained Shear Strength (ksf)	**0.628**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	353.4
Wt. Sample dry + tare (g)	233.3
Wt. Tare (g)	8.3
Moisture Content %	**53.4**
BULK DENSITY	
Sample Wt. (g)	1061.5
Diameter 1 (cm)	7.22
Diameter 2 (cm)	7.20
Diameter 3 (cm)	7.21
Avg. Diameter (cm)	**7.21**
Length 1 (cm)	15.36
Length 2 (cm)	15.38
Length 3 (cm)	15.35
Avg. Length (cm)	**15.36**
Volume (cm^3)	627.3
Moisture content (%)	53.4
Bulk Density (g/cm^3)	1.692
Bulk Density (kN/m^3)	**16.6**
Bulk Density (pcf)	**105.7**
Dry Density (kN/m^3)	**10.82**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

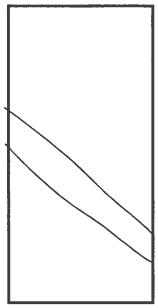
TEST HOLE NO.:	TH14-28
SAMPLE NO.:	T197
SAMPLE DEPTH:	7.62 - 8.23 m
SAMPLE DATE:	February, 2014
TEST DATE:	28-Nov-14

SOIL DESCRIPTION:

CLAY; silty, trace silt inclusions, brown, moist, firm,
int. - high plasticity,

MOISTURE CONTENT: 53.4

SAMPLE DIAM.(Do):	72.10	**(mm)**	**INITIAL AREA, Ao:**	4082.8	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.63	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.13	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.84	**(0.5<R<2 % / minute)**



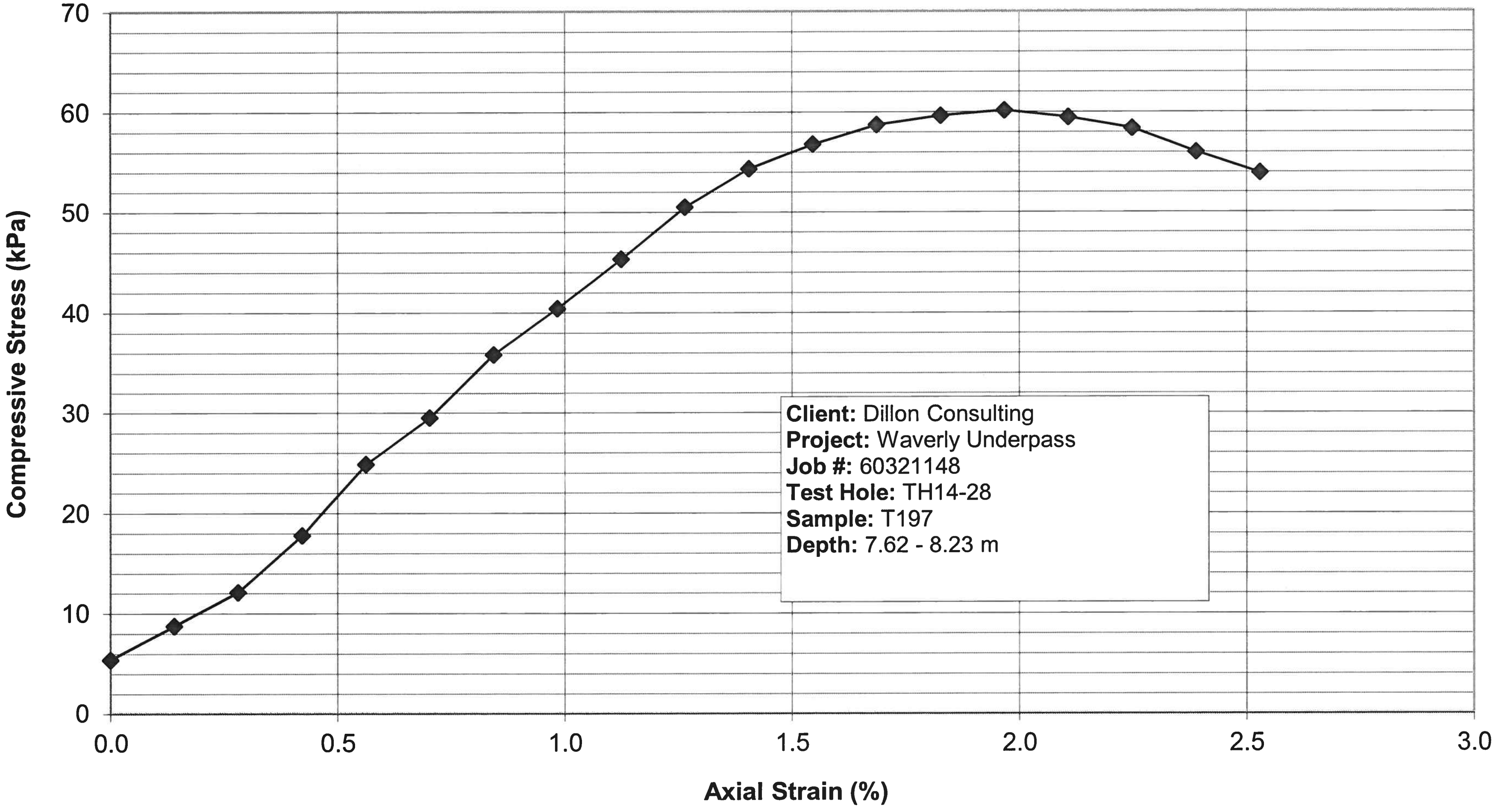
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_C**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0005	0.00	6.33	4.97	0.78	0.113	5.4
0.02	0.0009	0.14	6.34	8.06	1.27	0.183	8.8
0.03	0.0012	0.28	6.35	11.15	1.76	0.253	12.1
0.03	0.0018	0.42	6.36	16.40	2.58	0.372	17.8
0.04	0.0025	0.56	6.36	22.96	3.61	0.519	24.9
0.05	0.0029	0.70	6.37	27.27	4.28	0.616	29.5
0.06	0.0035	0.84	6.38	33.17	5.20	0.748	35.8
0.07	0.0040	0.98	6.39	37.48	5.86	0.844	40.4
0.08	0.0045	1.12	6.40	42.07	6.57	0.947	45.3
0.09	0.0050	1.26	6.41	46.94	7.32	1.055	50.5
0.09	0.0054	1.41	6.42	50.60	7.88	1.135	54.4
0.10	0.0057	1.55	6.43	52.94	8.24	1.186	56.8
0.11	0.0059	1.69	6.44	54.81	8.52	1.226	58.7
0.12	0.0060	1.83	6.45	55.75	8.65	1.245	59.6
0.13	0.0060	1.97	6.46	56.31	8.72	1.256	60.1
0.14	0.0060	2.11	6.46	55.75	8.62	1.242	59.5
0.14	0.0059	2.25	6.47	54.81	8.47	1.219	58.4
0.15	0.0056	2.39	6.48	52.66	8.12	1.170	56.0
0.16	0.0054	2.53	6.49	50.79	7.82	1.126	53.9

UNCONFINED COMPRESSIVE STRENGTH, q_u:	60.15	kPa
(based on maximum q_u value)	1.256	ksf
UNDRAINED SHEAR STRENGTH, S_u:	30.07	kPa
(based on maximum q_u value)	0.628	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-28
Sample: T197
Depth: 7.62 - 8.23 m

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	MLotecki
Sample Number:	Varies	Date Tested:	December 4, 2014

Group Index (ASTM D3282-09)

Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes

Location	Sample	Depth (m)	% Passing No. 200	Liquid Limit	Plasticity Index	Group Classification
TH14-07	G62	0.76 - 0.91 m	88.6	49.2	31.6	A-7-6(29)
TH14-16	G105	0.76 - 0.91 m	96.0	66.9	46.5	A-7-6(50)
TH14-18	G121	1.83 - 1.98 m	88.4	23.2	6.5	A-4(4)
TH14-21	G143	0.76 - 0.91 m	92.6	41.9	26.4	A-7-6(25)
TH14-25	G171	0.76 - 0.91 m	94.6	37.8	23.9	A-6(22)

AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
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204 477 5381 tel
204 284 2040 fax

Memorandum

To Saba Ibrahim

Page 1

CC

Subject Waverly Underpass

From Jared Baldwin

Date September 22, 2014

Project Number 60321148

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Twenty-six (26) Moisture Content tests.
- Two (2) Atterberg Limits (3 points) tests.
- Three (3) Torvane, Pocket Penetrometer, Moisture Content, Bulk Density and Visual Description with Unconfined Compressive Strength, on Shelby tube samples.
- Four (4) Waxed Shelby tube Samples.

If you have any questions, please contact the undersigned.

Sincerely,

Jared Baldwin, M.Sc., P.Eng.
Geotechnical Engineer

Att.

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	CMahe
Sample Number:	Varies	Date Tested:	August 19, 2014

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	**Moisture Content (%)**
TH14-01	G1	0.61 - 0.76 m	**38.8**
-	G3	2.13 - 2.29 m	**40.4**
-	G5	4.27 - 4.42 m	**54.7**
-	G10	8.23 - 8.38 m	**49.8**
-	G13	11.28 - 11.43 m	**44.2**
-	S15	12.19 - 12.34 m	**6.5**
TH14-02	G16	2.59 - 2.74 m	**39.8**
-	G19	5.64 - 5.79 m	**49.5**
-	T21	7.62 - 8.23 m	**48.5**
-	G23	10.06 - 10.21 m	**57.3**
-	G25	11.58 - 11.73 m	**39.7**
-	S28	13.41 - 13.87 m	**9.1**
-	S29	14.33 - 14.78 m	**13.1**
-	S30	15.24 - 15.70 m	**12.6**
TH14-03	G31	2.44 - 2.59 m	**35.6**
-	G33	5.33 - 5.49 m	**43.5**
-	G35	7.32 - 7.47 m	**46.8**
-	G38	10.97 - 11.13 m	**44.9**
-	G41	13.41 - 13.56 m	**9.7**
-	S42	13.72 - 14.17 m	**9.9**
TH14-04	G44	3.66 - 3.81 m	**49.0**
-	G46	6.40 - 6.55 m	**49.3**
-	G50	10.06 - 10.21 m	**37.0**
-	G52	12.80 - 12.95 m	**12.3**
-	S53	13.72 - 14.17 m	**7.6**
-	G47	7.32 - 7.47 m	**45.4**

Location	Sample	Depth (m)	**Moisture Content (%)**

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

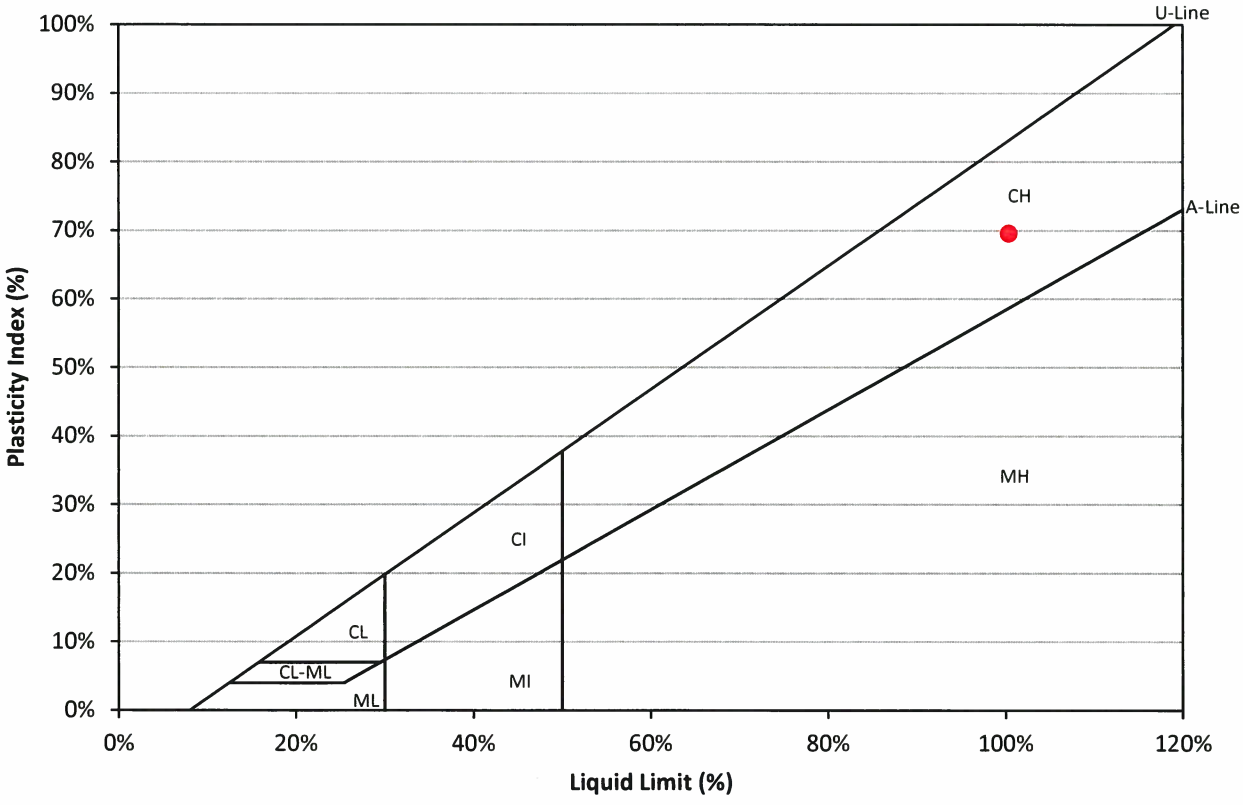
Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	14-01	Sample Date:	July 1, 2014
Sample Depth:	4.27	Lab Technician:	RDagg
Sample Number:	G5	Date Tested:	August 22, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	34	25	22
Wet Sample (g)	17.1	19.4	17.8
Dry Sample (g)	8.7	9.7	8.8
Water Content (%)	95.9%	100.0%	102.0%

Plastic Limit		
Trial	1	2
Wet Sample (g)	4.6	4.0
Dry Sample (g)	3.5	3.0
Water Content (%)	30.7%	30.7%



Liquid Limit (%): 100.3%	Plastic Limit (%): 30.7%	Plasticity Index (%): 69.6%

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

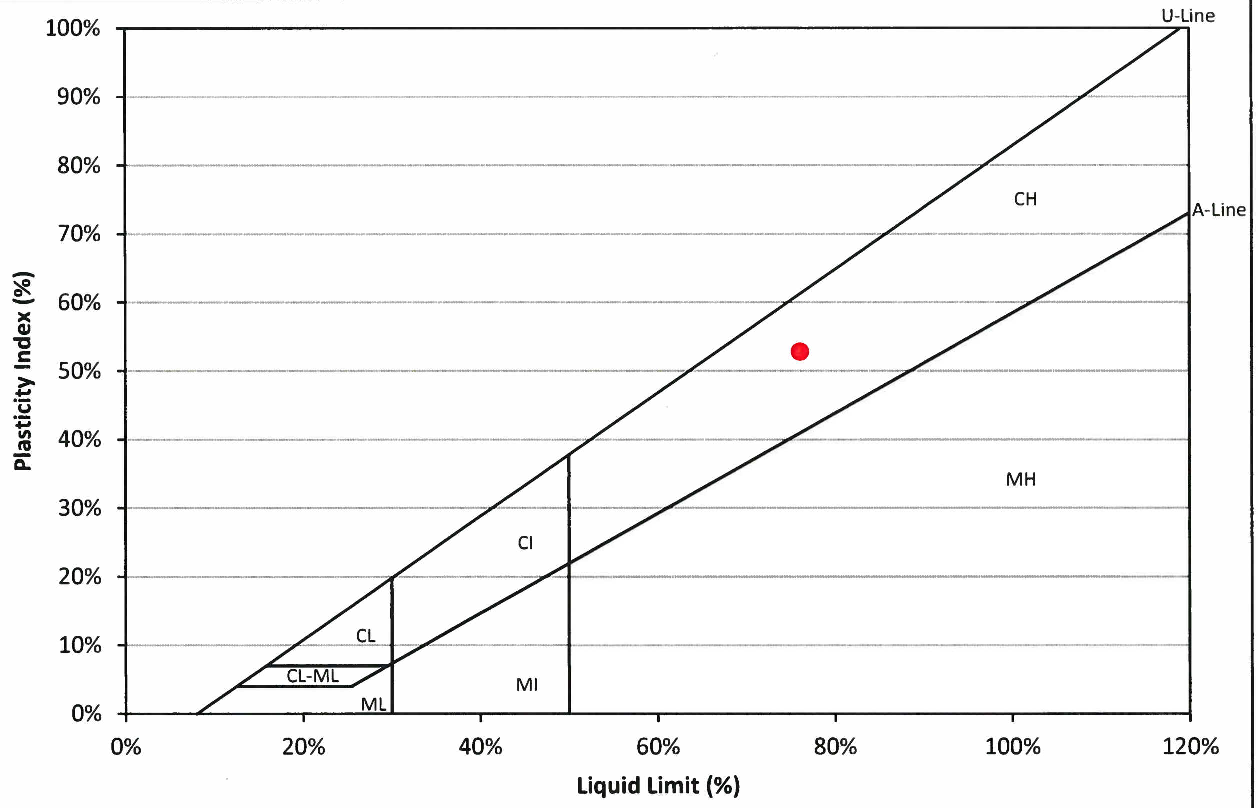
Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	14-02	Sample Date:	July 1, 2014
Sample Depth:	7.62	Lab Technician:	ML
Sample Number:	T21	Date Tested:	September 2, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	19	24	31
Wet Sample (g)	14.4	12.4	13.2
Dry Sample (g)	8.1	7.0	7.6
Water Content (%)	77.8%	76.1%	74.5%

Plastic Limit		
Trial	1	2
Wet Sample (g)	9.0	7.9
Dry Sample (g)	7.3	6.4
Water Content (%)	23.2%	23.1%



Liquid Limit (%): 76.0%	Plastic Limit (%): 23.2%	Plasticity Index (%): 52.9%

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-01**
SAMPLE NO.:	**T4**
SAMPLE DEPTH:	3.05 - 3.66 m
DATE TESTED:	2-Sep-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.55
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**53.9**
Undrained Shear Strength (ksf)	1.13
POCKET PENETROMETER	
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**69.5**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**34.8**
Undrained Shear Strength (ksf)	**0.726**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	442.0
Wt. Sample dry + tare (g)	303.6
Wt. Tare (g)	8.3
Moisture Content %	**46.9**
BULK DENSITY	
Sample Wt. (g)	1065.8
Diameter 1 (cm)	7.23
Diameter 2 (cm)	7.24
Diameter 3 (cm)	7.24
Avg. Diameter (cm)	**7.24**
Length 1 (cm)	15.34
Length 2 (cm)	15.35
Length 3 (cm)	15.36
Avg. Length (cm)	**15.35**
Volume (cm^3)	631.4
Moisture content (%)	46.9
Bulk Density (g/cm^3)	1.688
Bulk Density (kN/m^3)	**16.6**
Bulk Density (pcf)	**105.4**
Dry Density (kN/m^3)	**11.27**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

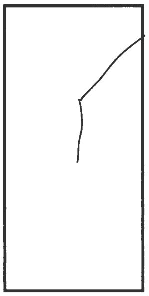
AECOM

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-01
SAMPLE NO.:	T4
SAMPLE DEPTH:	3.05 - 3.66 m
SAMPLE DATE:	February, 2014
TEST DATE:	2-Sep-14

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, brown, moist, firm, high plasticity,	
MOISTURE CONTENT:	46.9

SAMPLE DIAM.(Do):	72.37	**(mm)**	**INITIAL AREA, Ao:**	4113.1	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.50	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.84	**(0.5<R<2 % / minute)**



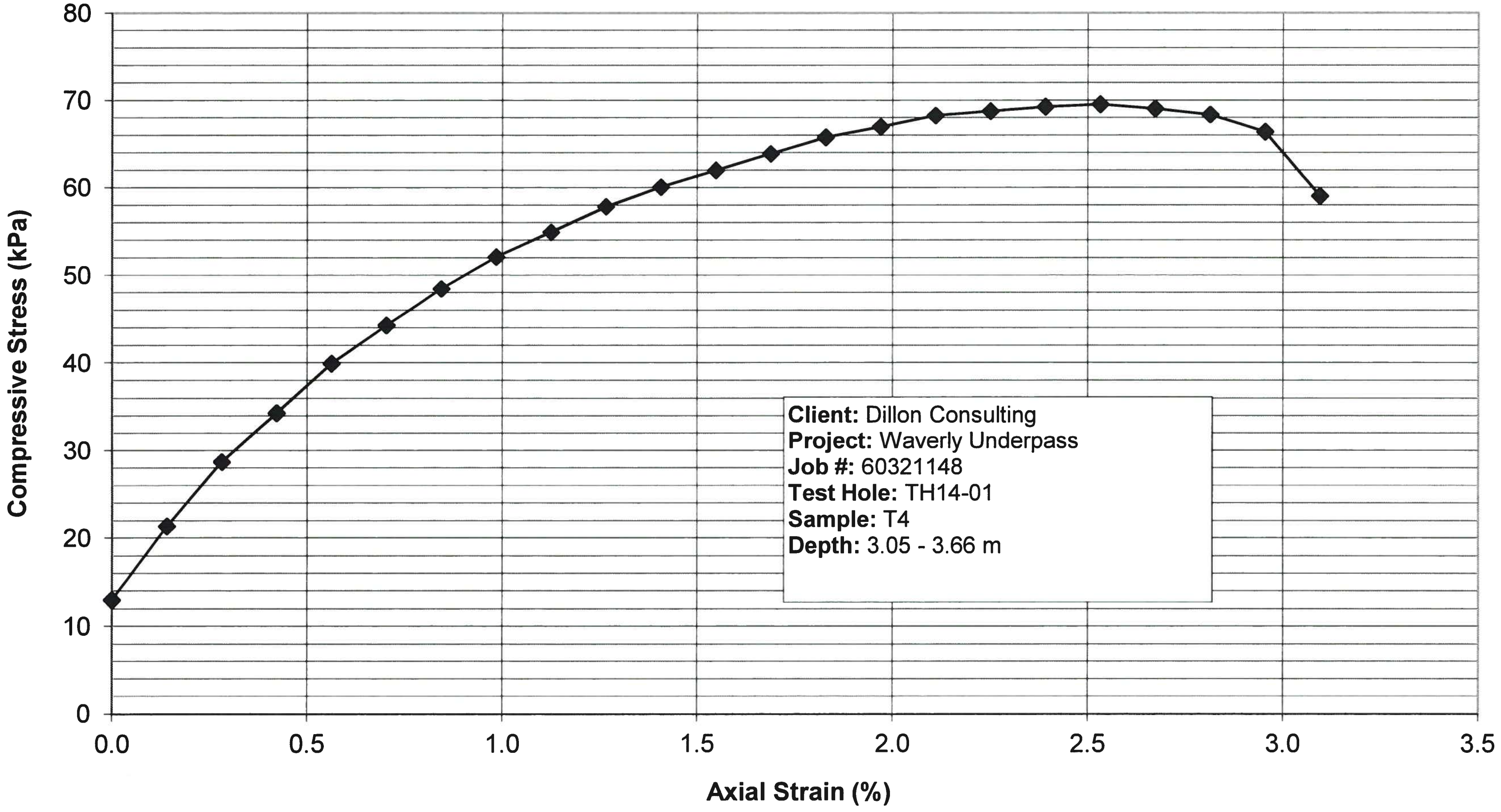
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_c**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0013	0.00	6.38	11.99	1.88	0.271	13.0
0.02	0.0021	0.14	6.38	19.77	3.10	0.446	21.4
0.03	0.0028	0.28	6.39	26.61	4.16	0.599	28.7
0.03	0.0034	0.42	6.40	31.86	4.98	0.717	34.3
0.04	0.0040	0.56	6.41	37.11	5.79	0.833	39.9
0.05	0.0044	0.70	6.42	41.23	6.42	0.925	44.3
0.06	0.0048	0.84	6.43	45.16	7.02	1.012	48.4
0.07	0.0052	0.98	6.44	48.63	7.55	1.088	52.1
0.08	0.0055	1.13	6.45	51.35	7.96	1.147	54.9
0.09	0.0058	1.27	6.46	54.16	8.39	1.208	57.8
0.09	0.0060	1.41	6.47	56.31	8.71	1.254	60.0
0.10	0.0062	1.55	6.48	58.19	8.99	1.294	62.0
0.11	0.0064	1.69	6.48	60.06	9.26	1.334	63.9
0.12	0.0066	1.83	6.49	61.94	9.54	1.373	65.8
0.13	0.0067	1.97	6.50	63.15	9.71	1.398	67.0
0.14	0.0069	2.11	6.51	64.47	9.90	1.425	68.2
0.14	0.0069	2.25	6.52	65.03	9.97	1.436	68.7
0.15	0.0070	2.39	6.53	65.59	10.04	1.446	69.2
0.16	0.0070	2.53	6.54	65.96	10.09	1.452	69.5
0.17	0.0070	2.67	6.55	65.59	10.01	1.442	69.0
0.18	0.0069	2.81	6.56	65.03	9.91	1.427	68.3
0.19	0.0068	2.95	6.57	63.25	9.63	1.386	66.4
0.20	0.0060	3.09	6.58	56.31	8.56	1.233	59.0

UNCONFINED COMPRESSIVE STRENGTH, q_u:	69.53	kPa
(based on maximum q_u value)	1.452	ksf
UNDRAINED SHEAR STRENGTH, S_u:	34.77	kPa
(based on maximum q_u value)	0.726	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-01
Sample: T4
Depth: 3.05 - 3.66 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-01**
SAMPLE NO.:	**T11**
SAMPLE DEPTH:	9.14 - 9.75 m
DATE TESTED:	2-Sep-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.55
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**53.9**
Undrained Shear Strength (ksf)	1.13
POCKET PENETROMETER	
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**69.9**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**35.0**
Undrained Shear Strength (ksf)	**0.730**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	372.8
Wt. Sample dry + tare (g)	270.7
Wt. Tare (g)	8.3
Moisture Content %	**38.9**
BULK DENSITY	
Sample Wt. (g)	1072.3
Diameter 1 (cm)	7.22
Diameter 2 (cm)	7.23
Diameter 3 (cm)	7.23
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	15.33
Length 2 (cm)	15.34
Length 3 (cm)	15.32
Avg. Length (cm)	**15.33**
Volume (cm^3)	628.8
Moisture content (%)	38.9
Bulk Density (g/cm^3)	1.705
Bulk Density (kN/m^3)	**16.7**
Bulk Density (pcf)	**106.5**
Dry Density (kN/m^3)	**12.04**

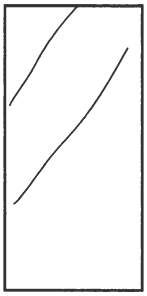
AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-01
SAMPLE NO.:	T11
SAMPLE DEPTH:	9.14 - 9.75 m
SAMPLE DATE:	February, 2014
TEST DATE:	2-Sep-14

SOIL DESCRIPTION:	
CLAY; trace sand, trace silt inclusions, trace gravel (5mm), brown, moist, firm	
high plasticity,	
MOISTURE CONTENT:	38.9

SAMPLE DIAM.(Do):	72.27	**(mm)**	**INITIAL AREA, Ao:**	4101.7	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.30	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.85	**(0.5<R<2 % / minute)**



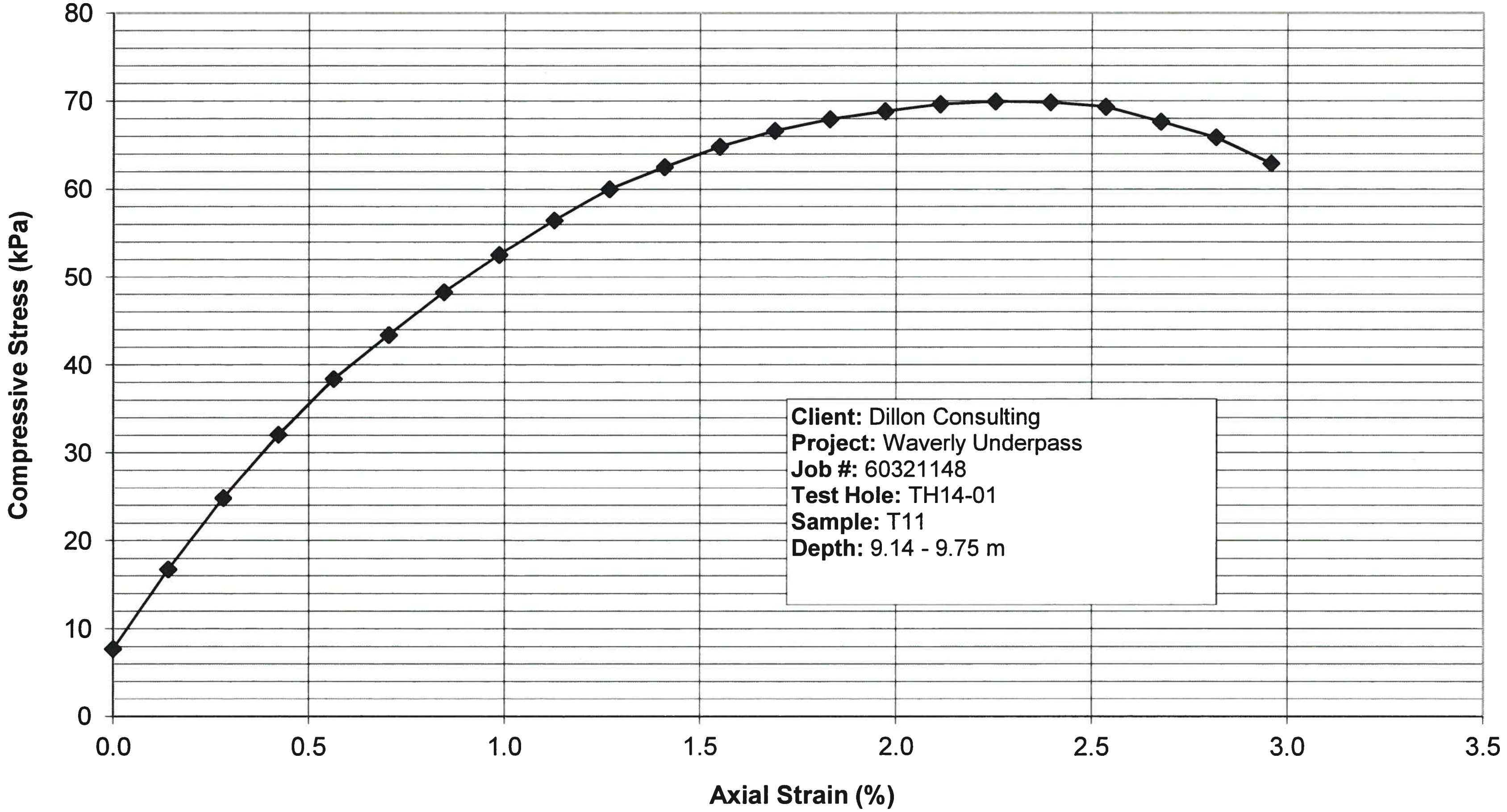
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_c**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0008	0.00	6.36	7.12	1.12	0.161	7.7
0.02	0.0017	0.14	6.37	15.46	2.43	0.350	16.7
0.03	0.0025	0.28	6.38	22.96	3.60	0.518	24.8
0.03	0.0032	0.42	6.38	29.70	4.65	0.670	32.1
0.04	0.0038	0.56	6.39	35.61	5.57	0.802	38.4
0.05	0.0043	0.70	6.40	40.29	6.29	0.906	43.4
0.06	0.0048	0.85	6.41	44.88	7.00	1.008	48.3
0.07	0.0052	0.99	6.42	48.91	7.62	1.097	52.5
0.08	0.0056	1.13	6.43	52.66	8.19	1.179	56.5
0.09	0.0060	1.27	6.44	56.03	8.70	1.253	60.0
0.09	0.0062	1.41	6.45	58.47	9.07	1.306	62.5
0.10	0.0065	1.55	6.46	60.72	9.40	1.354	64.8
0.11	0.0067	1.69	6.47	62.50	9.66	1.392	66.6
0.12	0.0068	1.83	6.48	63.81	9.85	1.419	67.9
0.13	0.0069	1.97	6.49	64.75	9.98	1.438	68.8
0.14	0.0070	2.11	6.49	65.59	10.10	1.454	69.6
0.14	0.0070	2.25	6.50	65.96	10.14	1.460	69.9
0.15	0.0070	2.39	6.51	65.96	10.13	1.458	69.8
0.16	0.0070	2.54	6.52	65.59	10.06	1.448	69.3
0.17	0.0068	2.68	6.53	64.09	9.81	1.413	67.6
0.18	0.0067	2.82	6.54	62.50	9.55	1.376	65.9
0.19	0.0064	2.96	6.55	59.78	9.12	1.314	62.9

UNCONFINED COMPRESSIVE STRENGTH, q_u:	69.93	kPa
(based on maximum q_u value)	1.460	ksf
UNDRAINED SHEAR STRENGTH, S_u:	34.96	kPa
(based on maximum q_u value)	0.730	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-01
Sample: T11
Depth: 9.14 - 9.75 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-02**
SAMPLE NO.:	**T18**
SAMPLE DEPTH:	4.57 - 5.18 m
DATE TESTED:	2-Sep-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.80
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**78.5**
Undrained Shear Strength (ksf)	1.64
POCKET PENETROMETER	
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**82.3**
Unconfined compressive strength (ksf)	**1.7**
Undrained Shear Strength (kPa)	**41.2**
Undrained Shear Strength (ksf)	**0.860**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	416.1
Wt. Sample dry + tare (g)	285.3
Wt. Tare (g)	9.3
Moisture Content %	**47.4**
BULK DENSITY	
Sample Wt. (g)	1080.9
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.24
Diameter 3 (cm)	7.21
Avg. Diameter (cm)	**7.22**
Length 1 (cm)	15.34
Length 2 (cm)	15.33
Length 3 (cm)	15.35
Avg. Length (cm)	**15.34**
Volume (cm^3)	627.5
Moisture content (%)	47.4
Bulk Density (g/cm^3)	1.723
Bulk Density (kN/m^3)	**16.9**
Bulk Density (pcf)	**107.5**
Dry Density (kN/m^3)	**11.46**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

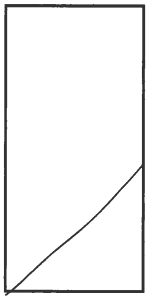
AECOM

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-02
SAMPLE NO.:	T18
SAMPLE DEPTH:	4.57 - 5.18 m
SAMPLE DATE:	February, 2014
TEST DATE:	2-Sep-14

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, brown, moist, firm, high plasticity,	
MOISTURE CONTENT:	47.4

SAMPLE DIAM.(Do):	72.17	**(mm)**	**INITIAL AREA, Ao:**	4090.4	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.40	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.13	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.84	**(0.5<R<2 % / minute)**



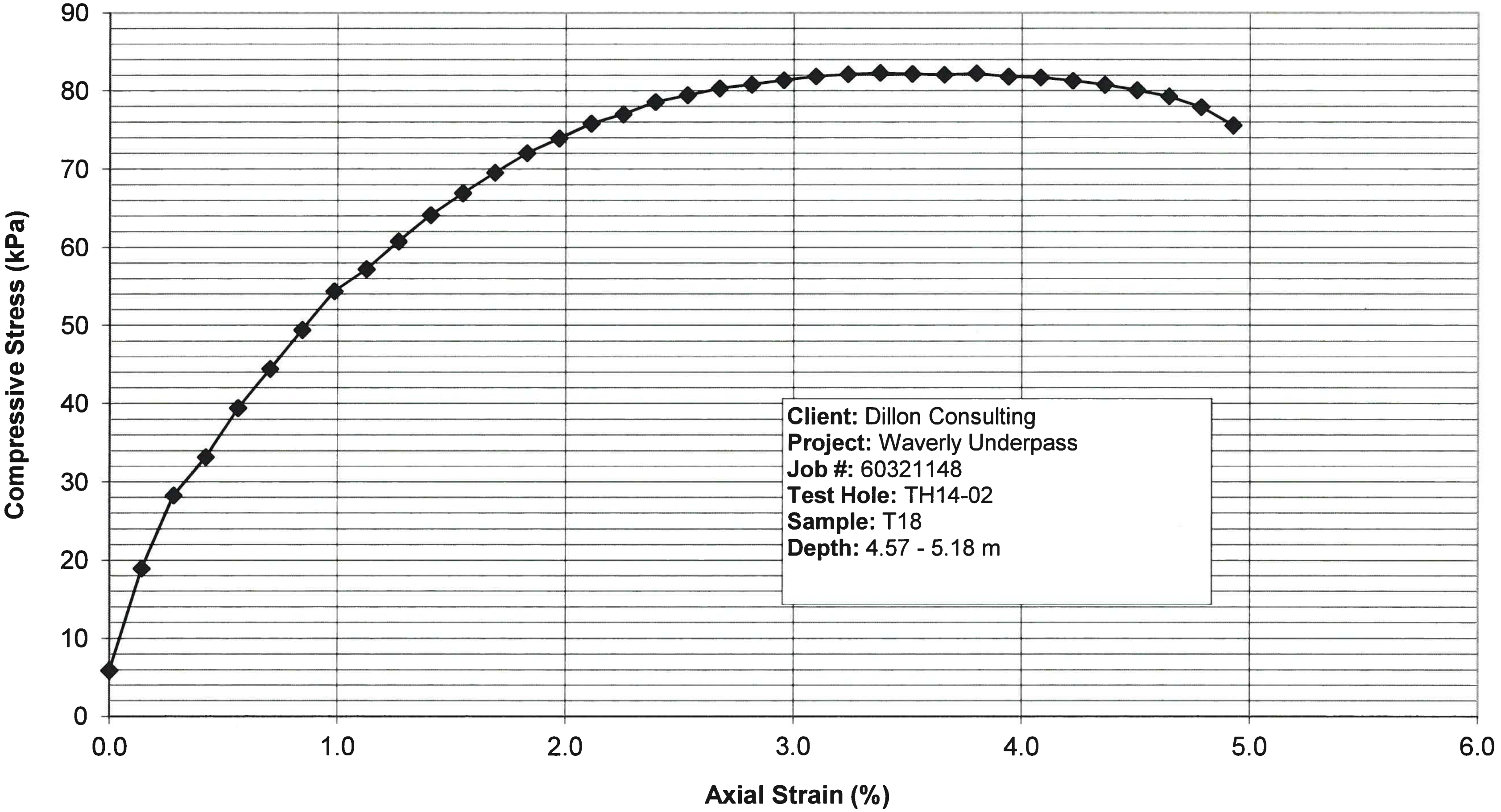
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_c**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0006	0.00	6.34	5.43	0.86	0.123	5.9
0.02	0.0019	0.14	6.35	17.43	2.75	0.395	18.9
0.03	0.0028	0.28	6.36	26.05	4.10	0.590	28.2
0.03	0.0033	0.42	6.37	30.64	4.81	0.693	33.2
0.04	0.0039	0.56	6.38	36.45	5.72	0.823	39.4
0.05	0.0044	0.70	6.39	41.13	6.44	0.928	44.4
0.06	0.0049	0.84	6.39	45.82	7.17	1.032	49.4
0.07	0.0054	0.99	6.40	50.50	7.89	1.136	54.4
0.08	0.0057	1.13	6.41	53.22	8.30	1.195	57.2
0.09	0.0060	1.27	6.42	56.59	8.81	1.269	60.8
0.09	0.0064	1.41	6.43	59.78	9.30	1.339	64.1
0.10	0.0067	1.55	6.44	62.50	9.70	1.398	66.9
0.11	0.0069	1.69	6.45	65.03	10.08	1.452	69.5
0.12	0.0072	1.83	6.46	67.46	10.45	1.504	72.0
0.13	0.0074	1.97	6.47	69.34	10.72	1.544	73.9
0.14	0.0076	2.11	6.48	71.21	10.99	1.583	75.8
0.14	0.0077	2.25	6.49	72.43	11.17	1.608	77.0
0.15	0.0079	2.39	6.50	74.02	11.40	1.641	78.6
0.16	0.0080	2.53	6.50	74.96	11.52	1.659	79.5
0.17	0.0081	2.67	6.51	75.90	11.65	1.678	80.3
0.18	0.0082	2.81	6.52	76.46	11.72	1.688	80.8
0.19	0.0082	2.96	6.53	77.12	11.80	1.700	81.4
0.20	0.0083	3.10	6.54	77.68	11.87	1.710	81.9
0.20	0.0083	3.24	6.55	78.05	11.91	1.715	82.1
0.21	0.0084	3.38	6.56	78.33	11.94	1.719	82.3
0.22	0.0084	3.52	6.57	78.33	11.92	1.717	82.2
0.23	0.0084	3.66	6.58	78.33	11.90	1.714	82.1
0.24	0.0084	3.80	6.59	78.61	11.93	1.718	82.2
0.25	0.0084	3.94	6.60	78.33	11.87	1.709	81.8
0.26	0.0084	4.08	6.61	78.33	11.85	1.707	81.7
0.26	0.0083	4.22	6.62	78.05	11.79	1.698	81.3
0.27	0.0083	4.36	6.63	77.68	11.72	1.687	80.8
0.28	0.0082	4.50	6.64	77.12	11.62	1.673	80.1
0.29	0.0082	4.64	6.65	76.46	11.50	1.656	79.3
0.30	0.0080	4.79	6.66	75.24	11.30	1.627	77.9
0.31	0.0078	4.93	6.67	73.09	10.96	1.578	75.6

UNCONFINED COMPRESSIVE STRENGTH, q_u:	82.31	kPa
(based on maximum q_u value)	1.719	ksf
UNDRAINED SHEAR STRENGTH, S_u:	41.15	kPa
(based on maximum q_u value)	0.860	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-02
Sample: T18
Depth: 4.57 - 5.18 m

Appendix D

Analysis of Pile Axial Capacity

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\USERS\ADMINI~1\DESKTOP\WAVERL~1\NEWPPC~1\WPPC1.DVN
Project Name: Waverley UP Project Date: 01/01/2015
Project Client: Dillon
Computed By: SI
Project Manager: FK

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 m
Length of Square Side: 279.00 mm

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	2.00 m
	- Driving/Restrike	2.00 m
	- Ultimate:	2.00 m
Ultimate Considerations:	- Local Scour:	0.00 m
	- Long Term Scour:	0.00 m
	- Soft Soil:	0.00 m

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	12.00 m	0.00%	17.00 kN/m^3	0.05 kPa	T-79 Concrete
2	Cohesionless	1.00 m	0.00%	18.00 kN/m^3	28.0/28.0	Nordlund
3	Cohesionless	2.00 m	0.00%	21.00 kN/m^3	36.0/39.3	Nordlund

ULTIMATE - SKIN FRICTION

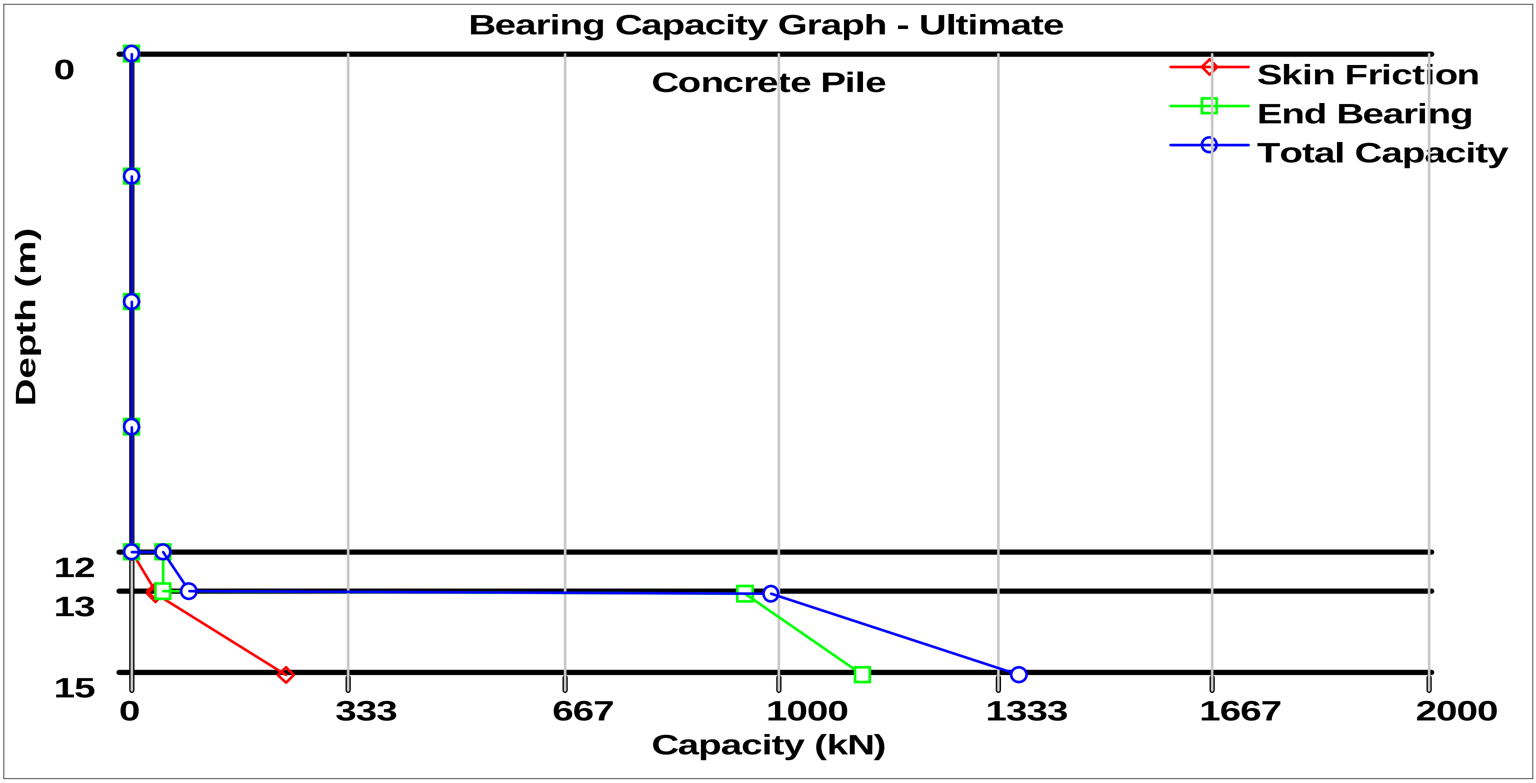
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 m	Cohesive	N/A	N/A	0.06 kPa	0.00 kN
3.01 m	Cohesive	N/A	N/A	0.06 kPa	0.19 kN
6.01 m	Cohesive	N/A	N/A	0.05 kPa	0.36 kN
9.01 m	Cohesive	N/A	N/A	0.05 kPa	0.52 kN
11.99 m	Cohesive	N/A	N/A	0.05 kPa	0.67 kN
12.01 m	Cohesionless	106.03 kPa	20.15	N/A	1.04 kN
12.99 m	Cohesionless	110.04 kPa	20.15	N/A	38.84 kN
13.01 m	Cohesionless	114.24 kPa	25.91	N/A	40.15 kN
14.99 m	Cohesionless	125.33 kPa	25.91	N/A	238.64 kN

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
3.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
6.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
9.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
11.99 m	Cohesive	N/A	N/A	N/A	0.04 kN
12.01 m	Cohesionless	106.07 kPa	22.80	49.64 kN	49.64 kN
12.99 m	Cohesionless	114.10 kPa	22.80	49.64 kN	49.64 kN
13.01 m	Cohesionless	114.30 kPa	143.28	1358.13 kN	943.99 kN
14.99 m	Cohesionless	136.47 kPa	143.28	1358.13 kN	1127.12 kN

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 m	0.00 kN	0.04 kN	0.04 kN
3.01 m	0.19 kN	0.04 kN	0.22 kN
6.01 m	0.36 kN	0.04 kN	0.40 kN
9.01 m	0.52 kN	0.04 kN	0.55 kN
11.99 m	0.67 kN	0.04 kN	0.70 kN
12.01 m	1.04 kN	49.64 kN	50.68 kN
12.99 m	38.84 kN	49.64 kN	88.48 kN
13.01 m	40.15 kN	943.99 kN	984.14 kN
14.99 m	238.64 kN	1127.12 kN	1365.76 kN



Bearing Capacity Graph for Precast-Prestressed Concrete Pile - HEX 300 mm

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\USERS\ADMINI~1\DESKTOP\WAVERL~1\NEWPPC~1\WPPC2.DVN
Project Name: Waverley UP Project Date: 01/01/2015
Project Client: Dillon
Computed By: SI
Project Manager: FK

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 m
Length of Square Side: 326.00 mm

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	2.00 m
	- Driving/Restrike	2.00 m
	- Ultimate:	2.00 m
Ultimate Considerations:	- Local Scour:	0.00 m
	- Long Term Scour:	0.00 m
	- Soft Soil:	0.00 m

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	12.00 m	0.00%	17.00 kN/m^3	0.05 kPa	T-79 Concrete
2	Cohesionless	1.00 m	0.00%	18.00 kN/m^3	28.0/28.0	Nordlund
3	Cohesionless	2.00 m	0.00%	21.00 kN/m^3	36.0/39.3	Nordlund

ULTIMATE - SKIN FRICTION

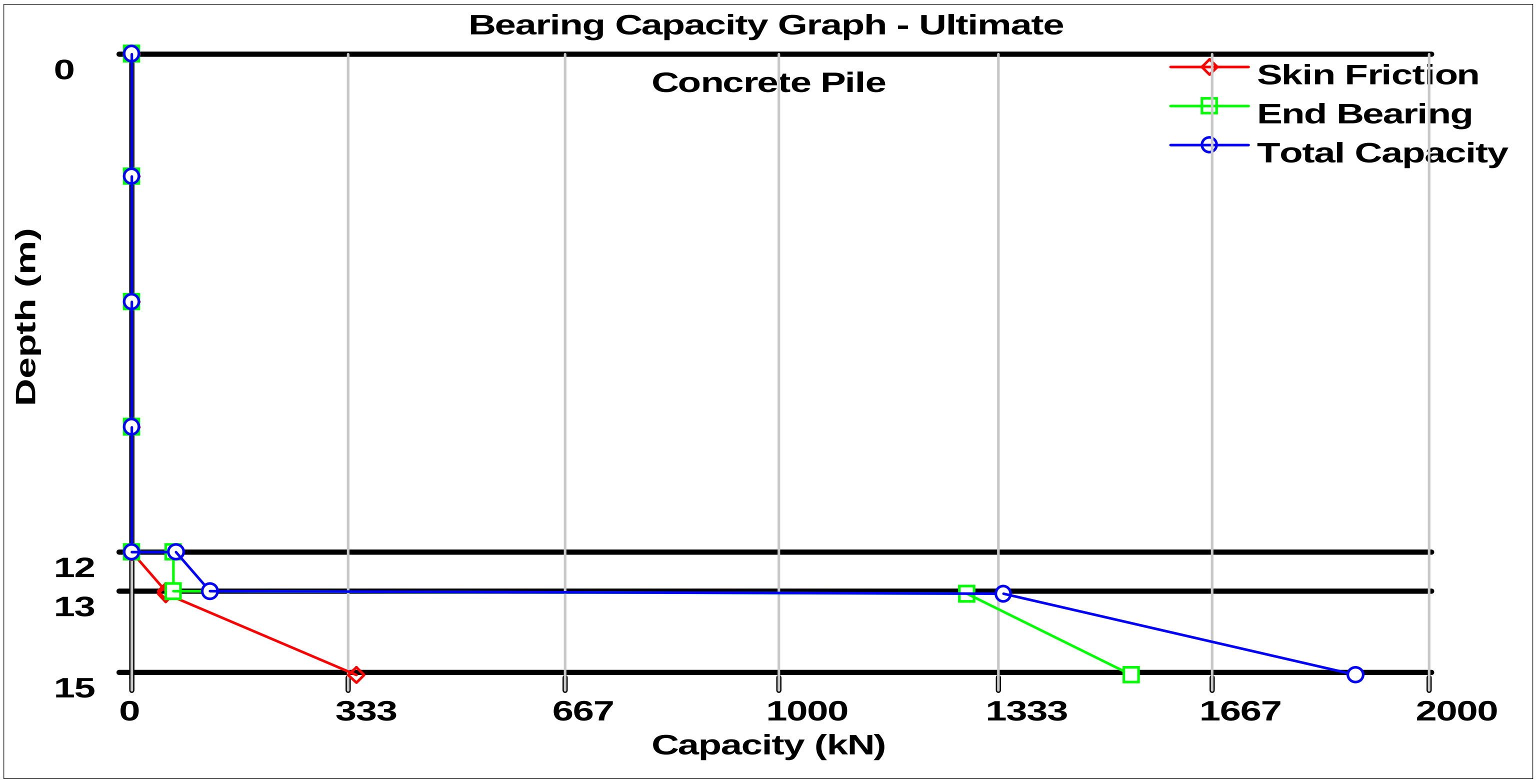
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 m	Cohesive	N/A	N/A	0.06 kPa	0.00 kN
3.01 m	Cohesive	N/A	N/A	0.06 kPa	0.22 kN
6.01 m	Cohesive	N/A	N/A	0.05 kPa	0.43 kN
9.01 m	Cohesive	N/A	N/A	0.05 kPa	0.62 kN
11.99 m	Cohesive	N/A	N/A	0.05 kPa	0.79 kN
12.01 m	Cohesionless	106.03 kPa	22.39	N/A	1.31 kN
12.99 m	Cohesionless	110.04 kPa	22.39	N/A	54.16 kN
13.01 m	Cohesionless	114.24 kPa	28.78	N/A	56.05 kN
14.99 m	Cohesionless	125.33 kPa	28.78	N/A	345.75 kN

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 m	Cohesive	N/A	N/A	N/A	0.05 kN
3.01 m	Cohesive	N/A	N/A	N/A	0.05 kN
6.01 m	Cohesive	N/A	N/A	N/A	0.05 kN
9.01 m	Cohesive	N/A	N/A	N/A	0.05 kN
11.99 m	Cohesive	N/A	N/A	N/A	0.05 kN
12.01 m	Cohesionless	106.07 kPa	22.80	67.78 kN	67.78 kN
12.99 m	Cohesionless	114.10 kPa	22.80	67.78 kN	67.78 kN
13.01 m	Cohesionless	114.30 kPa	143.28	1854.24 kN	1288.82 kN
14.99 m	Cohesionless	136.47 kPa	143.28	1854.24 kN	1538.85 kN

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 m	0.00 kN	0.05 kN	0.05 kN
3.01 m	0.22 kN	0.05 kN	0.27 kN
6.01 m	0.43 kN	0.05 kN	0.48 kN
9.01 m	0.62 kN	0.05 kN	0.67 kN
11.99 m	0.79 kN	0.05 kN	0.84 kN
12.01 m	1.31 kN	67.78 kN	69.09 kN
12.99 m	54.16 kN	67.78 kN	121.93 kN
13.01 m	56.05 kN	1288.82 kN	1344.87 kN
14.99 m	345.75 kN	1538.85 kN	1884.60 kN



Bearing Capacity Graph for Precast-Prestressed Concrete Pile - HEX 350 mm

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\USERS\ADMINI~1\DESKTOP\WAVERL~1\NEWPPC~1\WPPC3.DVN
Project Name: Waverley UP Project Date: 01/01/2015
Project Client: Dillon
Computed By: SI
Project Manager: FK

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 m
Length of Square Side: 372.00 mm

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	2.00 m
	- Driving/Restrike	2.00 m
	- Ultimate:	2.00 m
Ultimate Considerations:	- Local Scour:	0.00 m
	- Long Term Scour:	0.00 m
	- Soft Soil:	0.00 m

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	12.00 m	0.00%	17.00 kN/m^3	0.05 kPa	T-79 Concrete
2	Cohesionless	1.00 m	0.00%	18.00 kN/m^3	28.0/28.0	Nordlund
3	Cohesionless	2.00 m	0.00%	21.00 kN/m^3	36.0/39.3	Nordlund

ULTIMATE - SKIN FRICTION

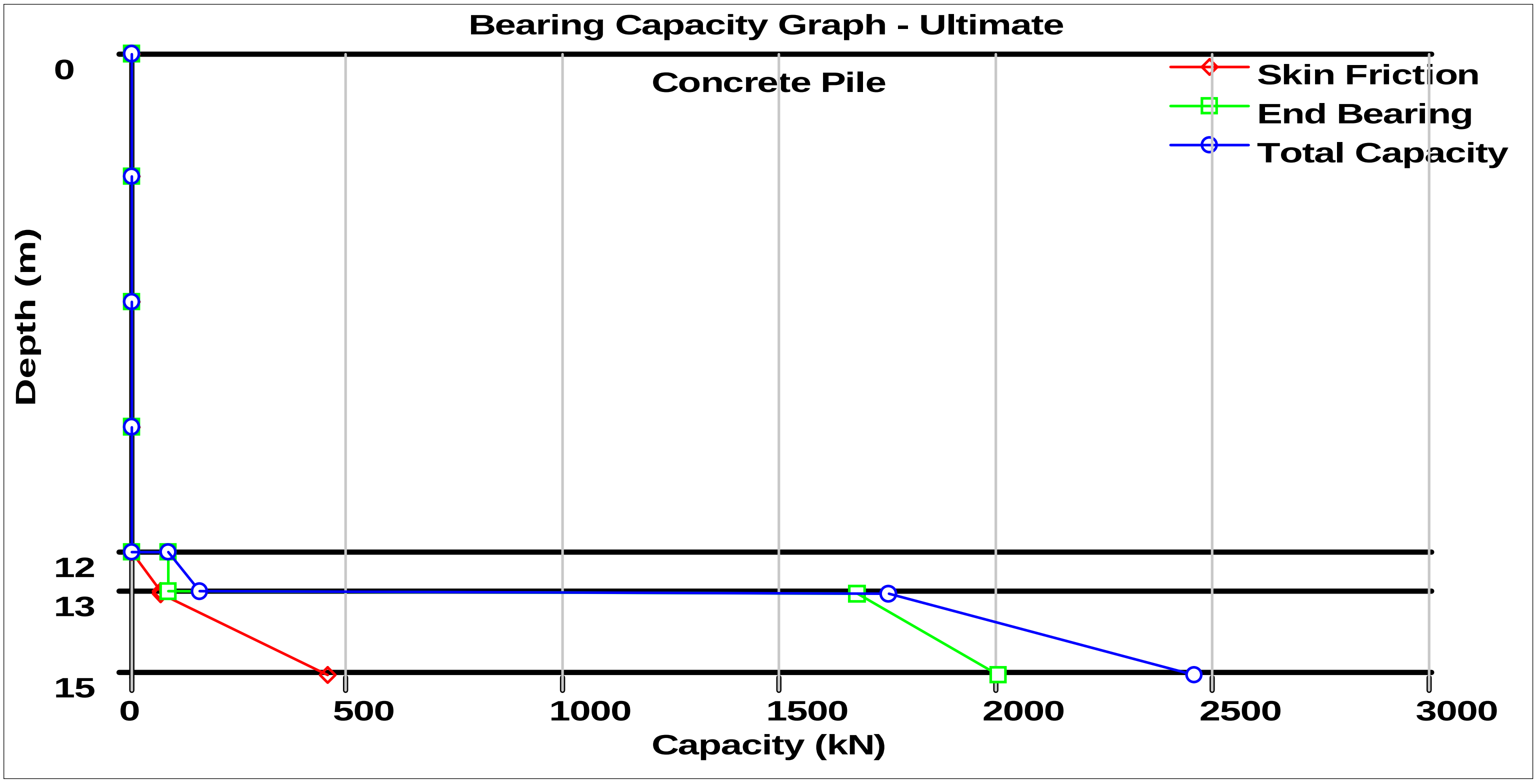
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 m	Cohesive	N/A	N/A	0.06 kPa	0.00 kN
3.01 m	Cohesive	N/A	N/A	0.06 kPa	0.25 kN
6.01 m	Cohesive	N/A	N/A	0.06 kPa	0.49 kN
9.01 m	Cohesive	N/A	N/A	0.05 kPa	0.72 kN
11.99 m	Cohesive	N/A	N/A	0.05 kPa	0.92 kN
12.01 m	Cohesionless	106.03 kPa	24.15	N/A	1.59 kN
12.99 m	Cohesionless	110.04 kPa	24.15	N/A	69.28 kN
13.01 m	Cohesionless	114.24 kPa	31.04	N/A	71.75 kN
14.99 m	Cohesionless	125.33 kPa	31.04	N/A	452.02 kN

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 m	Cohesive	N/A	N/A	N/A	0.06 kN
3.01 m	Cohesive	N/A	N/A	N/A	0.06 kN
6.01 m	Cohesive	N/A	N/A	N/A	0.06 kN
9.01 m	Cohesive	N/A	N/A	N/A	0.06 kN
11.99 m	Cohesive	N/A	N/A	N/A	0.06 kN
12.01 m	Cohesionless	106.07 kPa	22.80	88.25 kN	88.25 kN
12.99 m	Cohesionless	114.10 kPa	22.80	88.25 kN	88.25 kN
13.01 m	Cohesionless	114.30 kPa	143.28	2414.45 kN	1678.20 kN
14.99 m	Cohesionless	136.47 kPa	143.28	2414.45 kN	2003.77 kN

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 m	0.00 kN	0.06 kN	0.06 kN
3.01 m	0.25 kN	0.06 kN	0.32 kN
6.01 m	0.49 kN	0.06 kN	0.56 kN
9.01 m	0.72 kN	0.06 kN	0.78 kN
11.99 m	0.92 kN	0.06 kN	0.98 kN
12.01 m	1.59 kN	88.25 kN	89.84 kN
12.99 m	69.28 kN	88.25 kN	157.54 kN
13.01 m	71.75 kN	1678.20 kN	1749.95 kN
14.99 m	452.02 kN	2003.77 kN	2455.79 kN



Bearing Capacity Graph for Precast-Prestressed Concrete Pile - HEX 400 mm

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\USERS\ADMINI~1\DESKTOP\WAVERL~1\16MHPI~1\WHPILE.DVN
Project Name: Waverley UP		Project Date: 01/01/2015
Project Client: Dillon
Computed By: SI
Project Manager: FK

PILE INFORMATION

Pile Type: H Pile - HP310X110
Top of Pile: 0.00 m
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	2.00 m
	- Driving/Restrike	2.00 m
	- Ultimate:	2.00 m
Ultimate Considerations:	- Local Scour:	0.00 m
	- Long Term Scour:	0.00 m
	- Soft Soil:	0.00 m

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	12.00 m	0.00%	17.00 kN/m^3	0.05 kPa	T-79 Steel
2	Cohesionless	1.00 m	0.00%	18.00 kN/m^3	25.0/28.0	Nordlund
3	Cohesionless	3.00 m	0.00%	21.00 kN/m^3	38.0/40.3	Nordlund

ULTIMATE - SKIN FRICTION

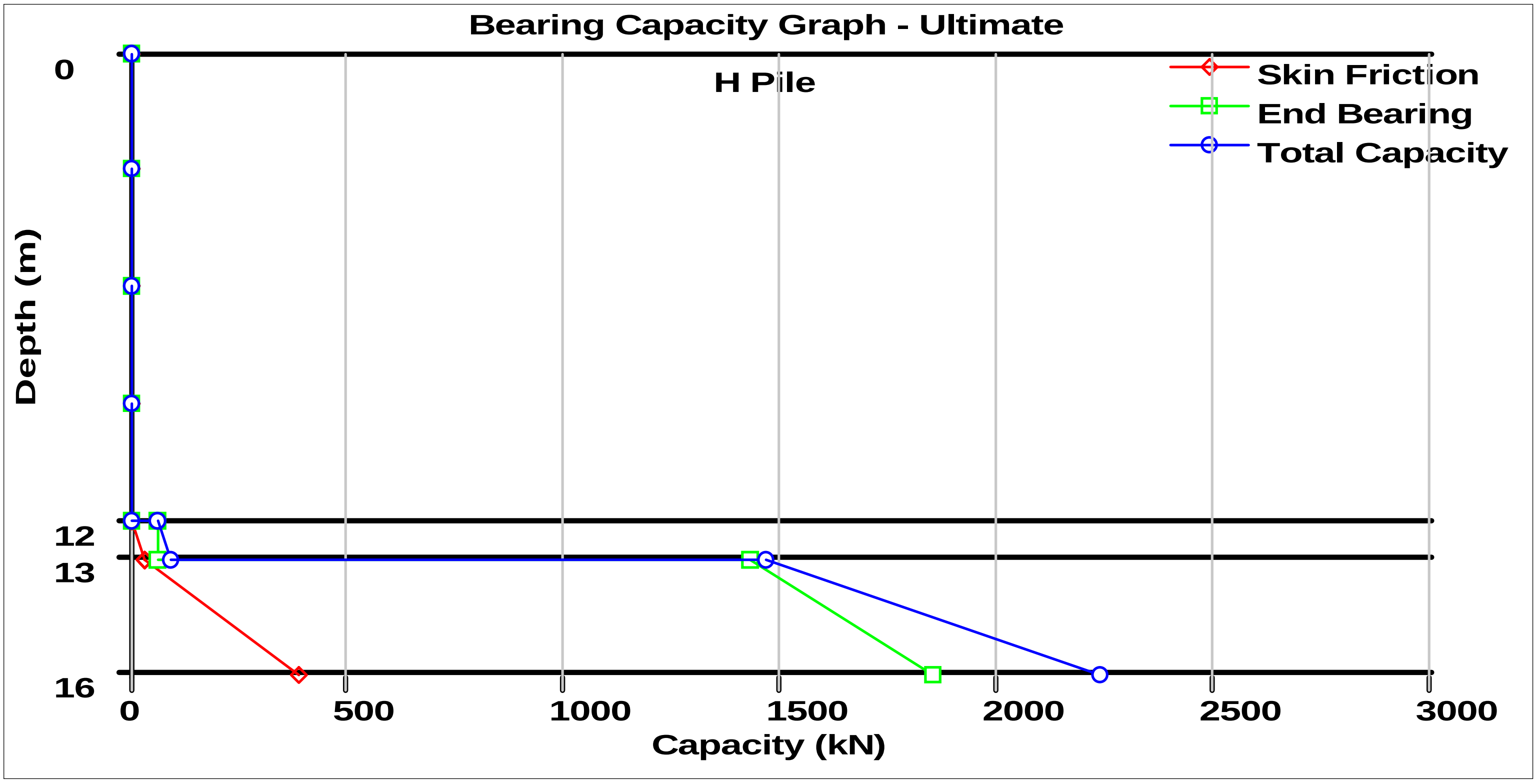
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 m	Cohesive	N/A	N/A	0.05 kPa	0.00 kN
3.01 m	Cohesive	N/A	N/A	0.05 kPa	0.19 kN
6.01 m	Cohesive	N/A	N/A	0.05 kPa	0.37 kN
9.01 m	Cohesive	N/A	N/A	0.05 kPa	0.56 kN
11.99 m	Cohesive	N/A	N/A	0.05 kPa	0.74 kN
12.01 m	Cohesionless	106.03 kPa	19.69	N/A	1.05 kN
12.99 m	Cohesionless	110.04 kPa	19.69	N/A	32.03 kN
13.01 m	Cohesionless	114.24 kPa	29.93	N/A	33.40 kN
15.99 m	Cohesionless	130.93 kPa	29.93	N/A	388.86 kN

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
3.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
6.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
9.01 m	Cohesive	N/A	N/A	N/A	0.04 kN
11.99 m	Cohesive	N/A	N/A	N/A	0.04 kN
12.01 m	Cohesionless	106.07 kPa	22.80	60.89 kN	60.89 kN
12.99 m	Cohesionless	114.10 kPa	22.80	60.89 kN	60.89 kN
13.01 m	Cohesionless	114.30 kPa	174.00	2039.93 kN	1431.69 kN
15.99 m	Cohesionless	147.67 kPa	174.00	2039.93 kN	1849.71 kN

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 m	0.00 kN	0.04 kN	0.04 kN
3.01 m	0.19 kN	0.04 kN	0.23 kN
6.01 m	0.37 kN	0.04 kN	0.41 kN
9.01 m	0.56 kN	0.04 kN	0.60 kN
11.99 m	0.74 kN	0.04 kN	0.78 kN
12.01 m	1.05 kN	60.89 kN	61.94 kN
12.99 m	32.03 kN	60.89 kN	92.92 kN
13.01 m	33.40 kN	1431.69 kN	1465.09 kN
15.99 m	388.86 kN	1849.71 kN	2238.57 kN



Bearing Capacity Graph for Steel H-Pile - HP 310X110

Appendix E

Slope Stability Figures

Waverley Underpass
Preliminary Engineering Study
Slope Stability Analysis
Open Cut Excavation - MAX. Height= 6 m; near Sta 1+790
File Name: 06-001 Waverley Underpass Slope Stability.gsz

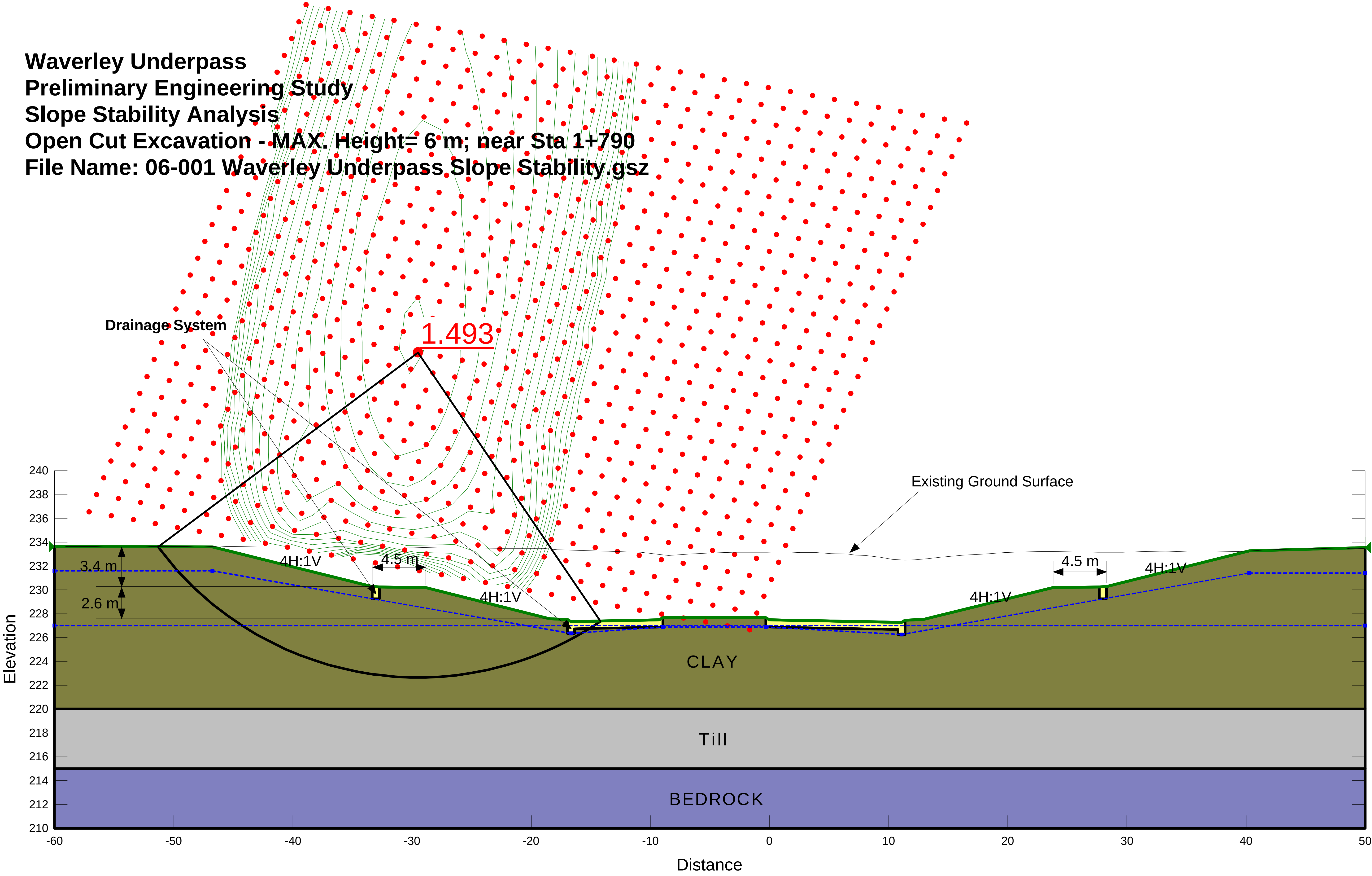


Figure 01

Waverley Underpass
Preliminary Engineering Study
Slope Stability Analysis
Open Cut Excavation - Sta 1+830 (H= 6.0-6.9 m)
File Name: 07-011 Waverley Underpass Slope Stability.gsz

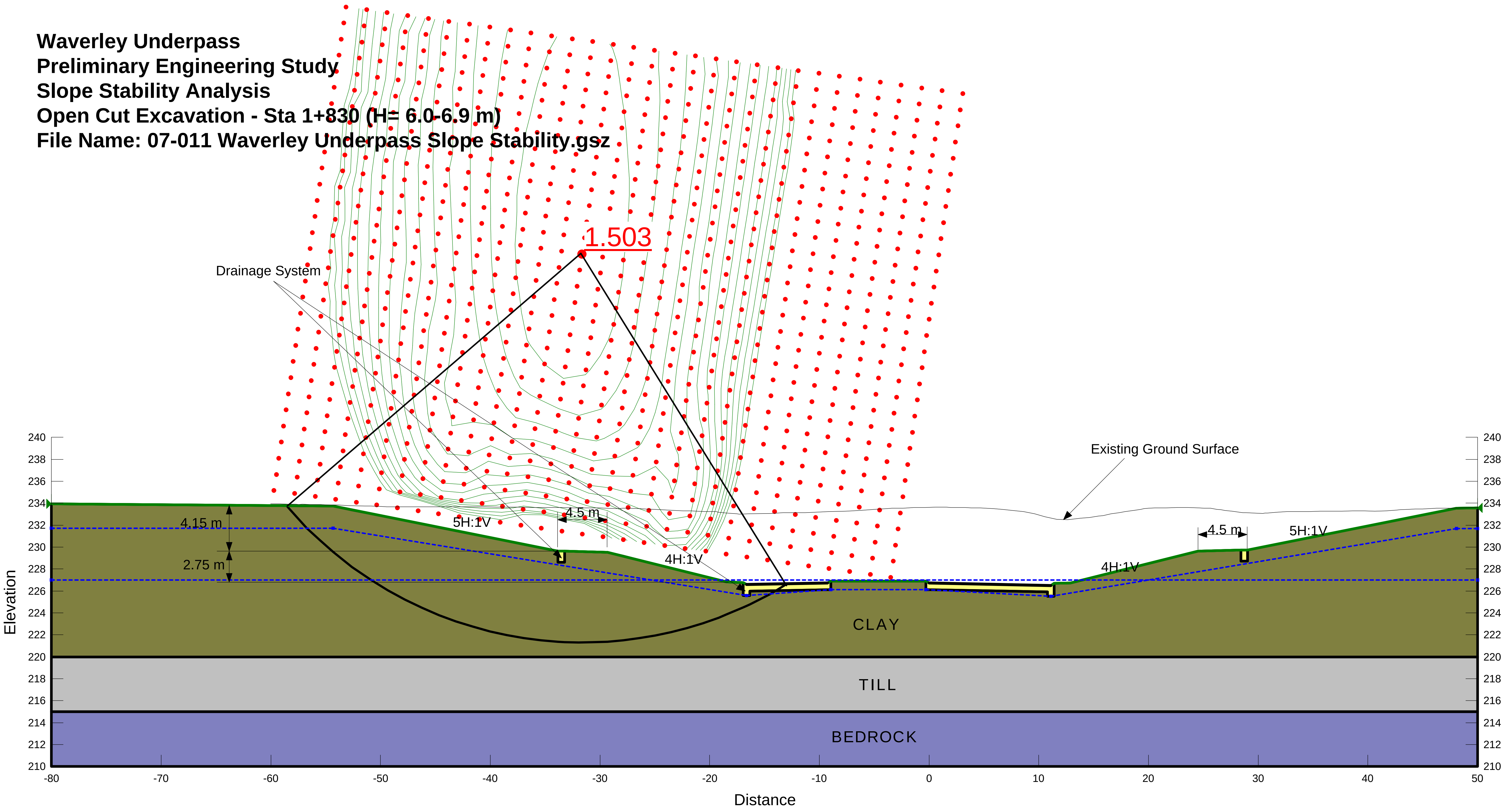


Figure 02

AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Memorandum

To	Andy Nagy, P.Eng	Page	1
CC			
Subject	Summary of Test Caisson Investigation – Waverley Underpass Project		
From	Saba Ibrahim		
Date	November 25, 2016	Project Number	60321148

A test caisson was advanced at a redundant pile to verify the design assumptions, examine the feasibility of construction, and assist in the selection of adequate equipment and proper construction practices. The drilling took place during the period between October 4th and October 7th, 2016. The test caisson was advanced on the south east side of Waverley Street intersection with existing CN railway, approximately 33 m south of the existing south CN track as shown on Figure 1, Appendix A. Drilling was carried out by Subterranean (Manitoba) Ltd. using a track-mounted Soilmec R-516 HD piling rig equipped with a 1200/910 mm diameter flight auger and 810 mm core barrel. Due to the size and heavy weight of the drill rig, a 0.3 m thick pad was constructed using granular rock fill to support the weight of the equipment. The test caisson was advanced through the clay overburden and till layer with augers to practical refusal into the bedrock at a depth of 17.2 m below surface. The core barrel was then employed to core into the bedrock from 17.2 m to a termination depth of 30.2 m below ground surface.

The caisson was sleeved with an outer temporary casing 4 feet (1.2 m) in diameter. The temporary safety casing extended from ground surface to a depth of 5.0 m below surface. An inner (permanent) sleeve was inserted into the test caisson to support the walls of the test hole at deeper depths. The inner (permanent) sleeve was 36 inch (0.91 m) in diameter and extended into the bedrock to a depth of 21.0 m below ground surface. The rock socket below depth of 21.0 m was advanced without the use of a sleeve (permanent) or casing to support the side walls of the hole.

The soil stratigraphy at the test caisson location consisted of a thin layer of topsoil and clay (fill) underlain by a thick lacustrine clay deposit extending to approximately 13.2 m below ground surface. The clay was soft to firm in consistency and of high plasticity. The clay was underlain by glacial till that typically contains variable amounts of clay, sand, and gravel as well as boulders and cobbles in silt matrix. Limestone bedrock was encountered at 17.2 m below ground surface. The top 4.5 m of the bedrock was highly fractured bedrock (very poor quality) and contained clay/sand infill zones and 0.8 m thick layer of fine grained shale. Limestone bedrock (poor to fair quality) was encountered at a depth between 23.8 m and 25.8 m below ground surface. Poor quality rock was encountered at depths below 25.8 and continued to the termination depth at 30.2 m below ground surface. A detailed log showing the soil stratums encountered is provided in Appendix A.

Sand layers were observed within the weathered/highly fractured rock zones at 18.7 m and between 20 to 20.8 m below ground surface. Water inflow and sand inflow in the test caisson were both observed within the weathered/highly fractured rock zones at depths ranged from 18 to 20.8 m and 18.7 to 20.8 m below ground surface, respectively.

During the course of the coring into the bedrock, and at the beginning of the rock coring, the inner (permanent) casing was inserted to a depth of 17.2 m. Subsequently, core barrel and the driving shoes of the inner (permanent) casing were both damaged within the weathered zone of the bedrock at depths between 17.2 and 23 m below ground surface. The drilling was suspended at the depth of 23 m and the static water depth was approximately at 12.0 m below ground surface (measured the next day prior to the commencement of first video inspection). Static water in the test caisson hole has been pumped out to the surface prior to conducting the first downhole video inspection. The first downhole video inspection up to 23 m below ground surface was performed to confirm that the proposed new depth of the inner (permanent) casing (21 m) is sufficient to maintain a stable hole excavation.

Subsequently, the damaged inner (permanent) casing was retrieved and replaced with new inner casing prior to proceeding with rock coring from 23 m until the termination depth at 30.2 m below ground surface. The depth of static water at the end of the rock coring was about 9.5 m below ground surface (measured three days later prior to the commencement of second video inspection). Water in the test caisson hole was pumped out again prior to conducting the second downhole video inspection. The second downhole video inspection was performed to aid in assessing the competency of the bedrock from 23 m to the termination depth at 30.2 m below ground surface.

Following the second video inspection, the test caisson hole was backfilled with concrete/bentonite mixture, from termination depth of 30.2 m up to 1.0 m below ground surface and with granular fill to ground surface.

Core barrel was utilized for coring into the bedrock and retrieving the rock cores from the bottom of the test caisson.

Caisson advancement was completed in approximately 24 hours of drilling including drilling into clay/till overburden which completed in about 2 hours and coring into bedrock which completed in about 22 hour. Additional time was required for site preparation including a granular pad placement at the caisson location, camera inspection and backfilling the caisson with concrete/bentonite mixture.

To summarize, based on observations from the test caisson drilling, the following practices are recommended for the installation of the bridge caissons:

- Permanent sleeve from ground surface into the weathered/highly fractured bedrock will be required to maintain a stable excavation.

- Video inspection of the test caisson is recommended to confirm the quality of the rock socket. However, if pumping of groundwater to inspect the socket would tend to de-stabilize the excavation due to pumping of fine sand from the fractured zones, an alternate method to confirm the quality of the socket core should be utilized. This should be combined with maintaining an extra water head inside the inner casing and probing the base of the socket with a weighted steel probe bar after cleaning and immediately before tremie concrete placement.

- The Soilmec R-516 HD or equivalent drill rig is capable of drilling deep caissons to the required depth in an efficient time manner.
- Tremie placement of concrete will be required due to the large amount of water seepage from the bedrock aquifer.
- The depth to the bedrock (poor to fair quality bedrock) is expected to vary across the site and it should be recognized that the test holes advanced at the bridge abutment and pier locations are more representative of expected ground conditions at those locations.

Closure

The findings and recommendations of this memorandum were based on the results of field investigations, combined with an interpolation of soil and groundwater conditions between the test hole locations. If conditions are encountered that appear to be different from those shown by the test hole drilled at this site and described in this memorandum, or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and adjusted, if necessary.

Soil conditions, by their nature, can be highly variable across a site. The placement of fill and prior construction activities on a site can contribute to the variability especially near surface soil conditions. A contingency should be included in the construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.

We trust the information provided herein is sufficient for your purposes.

Please don't hesitate to contact me should you have any questions or concerns.

Submitted by:	Reviewed by:
Saba Ibrahim, M.Sc, P.Eng. Geotechnical Engineer	Faris Alobaidy, M.Sc, P.Eng. Senior Geotechnical Engineer

Appendix A

Figure 01 / Log

Last saved by: MAHEC (2016-11-14) Last Plotted: never
Filename: P:\60321148\900-WORK\920-929 (GIS-GRAPHICS)\B\G007_60321148_01V0_CAISSONDRILLING.MXD

Project Management Initials: Designer: Checked: Approved: ANSI B 279.4mm x 431.8mm

Waverly Underpass Detailed Design

Test Caisson Location Plan

Dillon Consulting Limited
Project No.: 60321148

AECOM

Figure: 01

AECOM Canada Ltd.

GENERAL STATEMENT

NORMAL VARIABILITY OF SUBSURFACE CONDITIONS

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability for the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of earth work, foundations and similar. In the event of any changes in the basic design or location of the structures as outlined in this report or plan, we should be given the opportunity to review the changes and to modify or reaffirm in writing the conclusions and recommendations of this report.

The analysis and recommendations presented in this report are based on the data obtained from the borings and test pit excavations made at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere are not significantly different from those disclosed by the borings and excavations. However, variations in soil conditions may exist between the excavations and, also, general groundwater levels and conditions may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions differ from those encountered in the exploratory borings and excavations, are observed or encountered during construction, or appear to be present beneath or beyond excavations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Since it is possible for conditions to vary from those assumed in the analysis and upon which our conclusions and recommendations are based, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction procedures.

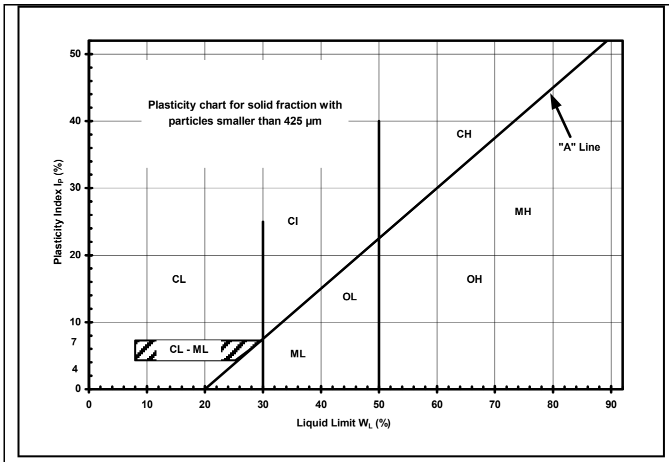
In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced soils engineer. We can be retained to provide these services for you during construction. In addition, we can be retained to review the plans and specifications that have been prepared to check for substantial conformance with the conclusions and recommendations contained in our report.

EXPLANATION OF FIELD & LABORATORY TEST DATA

Description				UMA Log Symbols	USCS Classification	Laboratory Classification Criteria			
						Fines (%)	Grading	Plasticity	Notes
COARSE GRAINED SOILS	GRAVELS (More than 50% of coarse fraction of gravel size)	CLEAN GRAVELS (Little or no fines)	Well graded gravels, sandy gravels, with little or no fines	△	GW	0-5	$C_U > 4$ $1 < C_C < 3$		Dual symbols if 5-12% fines. Dual symbols if above "A" line and $4<W_P<7$ $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10} x D_{60}}$
			Poorly graded gravels, sandy gravels, with little or no fines	▲	GP	0-5	Not satisfying GW requirements		
		DIRTY GRAVELS (With some fines)	Silty gravels, silty sandy gravels	▥	GM	> 12		Atterberg limits below "A" line or $W_P<4$	
			Clayey gravels, clayey sandy gravels	▨	GC	> 12		Atterberg limits above "A" line or $W_P<7$	
	SANDS (More than 50% of coarse fraction of sand size)	CLEAN SANDS (Little or no fines)	Well graded sands, gravelly sands, with little or no fines	◌	SW	0-5	$C_U > 6$ $1 < C_C < 3$		
			Poorly graded sands, gravelly sands, with little or no fines	○	SP	0-5	Not satisfying SW requirements		
		DIRTY SANDS (With some fines)	Silty sands, sand-silt mixtures	▥	SM	> 12		Atterberg limits below "A" line or $W_P<4$	
			Clayey sands, sand-clay mixtures	▨	SC	> 12		Atterberg limits above "A" line or $W_P<7$	
FINE GRAINED SOILS	SILTS (Below 'A' line negligible organic content)	$W_L<50$	Inorganic silts, silty or clayey fine sands, with slight plasticity	▥	ML			Classification is Based upon Plasticity Chart	
		$W_L>50$	Inorganic silts of high plasticity	▥	MH				
	CLAYS (Above 'A' line negligible organic content)	$W_L<30$	Inorganic clays, silty clays, sandy clays of low plasticity, lean clays	▨	CL				
		$30<W_L<50$	Inorganic clays and silty clays of medium plasticity	▨	CI				
		$W_L>50$	Inorganic clays of high plasticity, fat clays	▨	CH				
	ORGANIC SILTS & CLAYS (Below 'A' line)	$W_L<50$	Organic silts and organic silty clays of low plasticity	▥	OL				
		$W_L>50$	Organic clays of high plasticity	▨	OH				
HIGHLY ORGAINIC SOILS			Peat and other highly organic soils	≋	Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

■	Asphalt	▥	Till		
⬚	Concrete	▨	Bedrock (Undifferentiated)		
▩	Fill	▦	Bedrock (Limestone)		

When the above classification terms are used in this report or test hole logs, the designated fractions may be visually estimated and not measured.



<table>
<tr><th colspan="2" rowspan="2">FRACTION</th><th colspan="2">SEIVE SIZE (mm)</th><th colspan="2">DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS</th></tr>
<tr><th>Passing</th><th>Retained</th><th>Percent</th><th>Identifier</th></tr>
<tr><td rowspan="2">Gravel</td><td>Coarse</td><td>76</td><td>19</td><td rowspan="2">35-50</td><td rowspan="2">and</td></tr>
<tr><td>Fine</td><td>19</td><td>4.75</td></tr>
<tr><td rowspan="3">Sand</td><td>Coarse</td><td>4.75</td><td>2.00</td><td rowspan="2">20-35</td><td rowspan="2">“y” or “ey” *</td></tr>
<tr><td>Medium</td><td>2.00</td><td>0.425</td></tr>
<tr><td>Fine</td><td>0.425</td><td>0.075</td><td rowspan="2">10-20</td><td rowspan="2">some</td></tr>
<tr><td colspan="2" rowspan="2">Silt (non-plastic)
or Clay (plastic)</td><td colspan="2" rowspan="2">< 0.075 mm</td></tr>
<tr><td>1-10</td><td>trace</td></tr>
<tr><td colspan="6">* for example: gravelly, sandy clayey, silty</td></tr>
<tr><td colspan="6">Definition of Oversize Material
COBBLES: 76mm to 300mm diameter
BOULDERS: >300mm diameter</td></tr>
</table>

LEGEND OF SYMBOLS

Laboratory and field tests are identified as follows:

q_u - undrained shear strength (kPa) derived from unconfined compression testing.

T_v - undrained shear strength (kPa) measured using a torvane

pp - undrained shear strength (kPa) measured using a pocket penetrometer.

L_v - undrained shear strength (kPa) measured using a lab vane.

F_v - undrained shear strength (kPa) measured using a field vane.

γ - bulk unit weight (kN/m^3).

SPT - Standard Penetration Test. Recorded as number of blows (N) from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 51 mm O.D. Raymond type sampler 0.30 m into the soil.

DPPT - Drive Point Pentrometer Test. Recorded as number of blows from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 50 mm drive point 0.30 m into the soil.

w - moisture content (W_L, W_P)

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Su (kPa)	**CONSISTENCY**
<12	very soft
12 – 25	soft
25 – 50	medium or firm
50 – 100	stiff
100 – 200	very stiff
200	hard

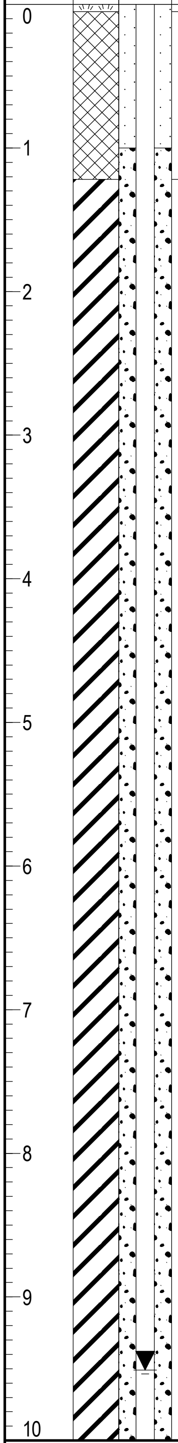
The resistance (N) of a non-cohesive soil can be related to compactness condition as follows

N – BLOWS/0.30 m	**COMPACTNESS**
0 - 4	very loose
4 - 10	loose
10 - 30	compact
30 - 50	dense
50	very dense

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **Test Caisson**
LOCATION: UTM: 14U,5523547 m N,630955 m E, South-East corner of CN/ Waverley Street Intersection		PROJECT NO.: 60321148
CONTRACTOR: Subterranean (Manitoba) LTD.	METHOD: Track Mounted Soilmec R-516 HD	ELEVATION (m): 233.10

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	◪ NO RECOVERY	▌ CORE
BACKFILL TYPE	■ BENTONITE	⁙ GRAVEL	▥ SLOUGH	⁙ GROUT	▨ CUTTINGS	⁙ SAND

DEPTH (m)	SOIL SYMBOL	BACKFILL DETAILS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ◉ Field Vane ◉ (kPa) 50 100 150 200	COMMENTS	ELEVATION



TOP SOIL -

CLAY (FILL) -
- black to dark brown, moist, soft to firm
- high plasticity

CLAY - trace silt
- brown, moist, firm to stiff
- high plasticity
- trace oxidation
- trace sulphate

- grey, soft below 7.7 m

233
232
231
230
229
228
227
226
225
224

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 30.18 m
REVIEWED BY: Faris Al-Alobaidy	COMPLETION DATE: 10/7/16
PROJECT ENGINEER: Andy Nagy	

LOG OF TEST HOLE WAVERLEY UP- TEST CAISSON.GPJ UMA WINN.GDT 11/25/16

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **Test Caisson**
LOCATION: UTM: 14U,5523547 m N,630955 m E, South-East corner of CN/ Waverley Street Intersection		PROJECT NO.: 60321148
CONTRACTOR: Subterranean (Manitoba) LTD.	METHOD: Track Mounted Soilmec R-516 HD	ELEVATION (m): 233.10

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁚ GRAVEL ▥ SLOUGH ⁚ GROUT ▨ CUTTINGS ⁚ SAND

DEPTH (m)	SOIL SYMBOL	BACKFILL DETAILS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
10										223
11										222
12										221
			- some till inclusions, very soft below 12.3 m							
13										220
			Glacial Till (SILT)- some sand to sandy, some gravel to gravelly, trace to some clay - light grey, very dense, moist - low plasticity							
14										219
			- some cobbles and boulders below 14.0 m							
15										218
			- moist to dry, hard below 15.5 m							
16										217
17										216
			- LIMESTONE - Non Intact							
18										215
			- layer of sand at 18.7 m							
19										214
			LIMESTONE - very fine to fine grained - pinkish yellow and grey - undulating to planar, smooth to rough fractures							
20										

AECOM

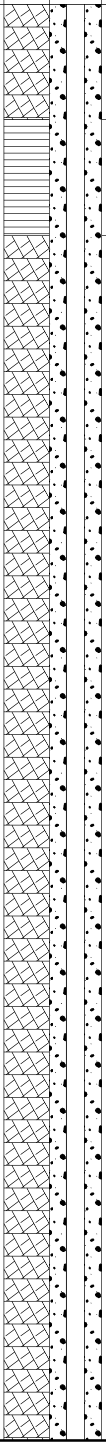
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 30.18 m
REVIEWED BY: Faris Al-Alobaidy	COMPLETION DATE: 10/7/16
PROJECT ENGINEER: Andy Nagy	

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **Test Caisson**
LOCATION: UTM: 14U,5523547 m N,630955 m E, South-East corner of CN/ Waverley Street Intersection		PROJECT NO.: 60321148
CONTRACTOR: Subterranean (Manitoba) LTD.	METHOD: Track Mounted Soilmec R-516 HD	ELEVATION (m): 233.10

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁙ GROUT ▨ CUTTINGS ⁘ SAND

DEPTH (m)	SOIL SYMBOL	BACKFILL DETAILS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION



Depth (m): 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

Elevation: 213, 212, 211, 210, 209, 208, 207, 206, 205, 204

- R2- weak below 20.0 m
- non- intact fine grained SHALE and fractured LIMESTONE between 19.5 to 20 m
- sand infill between 20 to 20.8 m

SHALE
- blue / green
- fine grained
- R0 to R1- extremely weak to very weak

LIMESTONE - very fine to fine grained
- creamish brown and white
- close to moderately closed spacing, evidence of water flow (class 3), smooth to rough fractures.
- R3- medium strong

- laminated with fine grained SHALE and hard grey CLAY from 23.3 to 24 m

- fair quality rock between 24 to 24.85 m

- non-intact fine grained SHALE between 24.85 to 25.15 m

- poor to fair quality rock between 25.15 to 25.75 m

- poor quality rock from 25.75 to 30.2 m,
- laminated with hard grey CLAY between 25.75 to 27.0 m

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 30.18 m
REVIEWED BY: Faris Al-Alobaidy	COMPLETION DATE: 10/7/16
PROJECT ENGINEER: Andy Nagy	Page 3 of 4

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PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **Test Caisson**
LOCATION: UTM: 14U,5523547 m N,630955 m E, South-East corner of CN/ Waverley Street Intersection		PROJECT NO.: 60321148
CONTRACTOR: Subterranean (Manitoba) LTD.	METHOD: Track Mounted Soilmec R-516 HD	ELEVATION (m): 233.10

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▨ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND

DEPTH (m)	SOIL SYMBOL	BACKFILL DETAILS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
30			END OF TEST CAISSON AT 30.2 m IN BEDROCK Notes: 1. Non-Intact bedrock encounterd at 17.2 m below ground surface. 2. Seepage observed at 8.7 m below ground surface, 3. 0.81 m diameter coring below 17.2 m. 4. Test caisson backfilled with concrete/bentonite up to 1.0 m below ground surface and with granular fill to ground surface. 5. Major water inflow observed at depths between 18 to 20.8 m. 6. Sand inflow observed at depths between 18.7 to 20.8 m. 7. Static water level at 9.5 m below ground surface							203
31										202
32										201
33										200
34										199
35										198
36										197
37										196
38										195
39										194
40										

AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 30.18 m
REVIEWED BY: Faris Al-Alobaidy	COMPLETION DATE: 10/7/16
PROJECT ENGINEER: Andy Nagy	Page 4 of 4

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Winnipeg, MB, Canada R3P 0Y7	204 284 2040 fax
www.aecom.com

Memorandum

To	Rados Eric, P. Eng.	Page	1
CC	Andy Nagy		
Subject	Summary of Bedrock Investigation in the Vicinity of the Proposed CN Bridge - Waverley Street Underpass Project		
From	Saba Ibrahim		
Date	November 23, 2016	Project Number	60321148 (400)

The City of Winnipeg (The City) retained Dillon Consulting Limited (Dillon) and AECOM Canada Limited (AECOM) to provide preliminary and detailed design services for the proposed Waverley Street Underpass Upgrade. The proposed Waverley Street Underpass will replace the existing at-grade CN Railway Rivers Subdivision crossing at Waverley Street with a new bridge structure.

Based on the design development during the preliminary as well as detailed design stages, driven steel H piles have been selected as the preferred foundation system to support the abutments of the proposed underpass structures while rock socketed caissons have been selected as a suitable foundation system to support the intermediate piers.

During the preliminary design stage, three deep test holes have been drilled in the close proximity of the proposed bridge. Based on the final configuration of the bridge structure, supplemental three deep test holes have been drilled during the detailed design stage in the close proximity of the bridge structure support units.

This memorandum documents the bedrock investigation and groundwater condition results obtained during preliminary and detailed design stages and provides geotechnical recommendations related to the design and construction of the proposed CN bridge foundations.

The underpass structure foundation recommendations were prepared following the guidance of AREMA 2014.

Geotechnical Investigation

1.1 Field Work

The field works for the deep test holes at the vicinity of the proposed underpass bridge structure was completed in two stages as follows:

Preliminary Design Stage

Three deep test hole (TH14-02 to 14-04) were drilled at the vicinity of the proposed underpass structure during the period from July 11 to 15, 2014 to depths of 24.4 to 25.7 m below existing grade. The test holes were located at both ends of the proposed underpass structure. The first 2.2 to 2.5 m of the test holes were advanced using hydrovac excavation to protect shallow underground utilities. The drilling was completed using a track mounted rig operated by Maple Leaf Drilling equipped with 125 mm diameter solid stem augers and HQ wireline for rock coring. The test holes were advanced more than 6 m into bedrock.

Detailed Design Stage

Drilling was completed during the period from April 13 to 19, 2016 and consisted of three test holes (TH16-01 to TH16-03). The test holes were located at both ends of the proposed bridge, in close proximity to the proposed piers and abutments. The first 2.2 to 2.5 m of the test holes were advanced by using hydrovac excavation to protect shallow underground utilities. The drilling was completed using a track mounted rig operated by Maple Leaf Drilling equipped with 125 mm diameter solid stem augers and HQ wireline for rock coring. The test holes were advanced more than 6 m into bedrock at the vicinity of the pier location and more than 3.0 m into bedrock at the vicinity of abutment location to depths of 24.5 m to 27.5 m below existing grade.

During the course of the investigation, Standard Penetration Tests (SPT) were completed at regular intervals in the clay as well as till layers. Disturbed and relatively undisturbed soil samples and rock cores were collected for further visual classification and testing.

Five standpipe piezometers were installed during preliminary design stage within the project area to monitor the groundwater conditions. These included two standpipe piezometers (SP14-02 and 14-04) installed in the bedrock unit, two standpipe piezometers (SP14-01 and 14-28) installed in the clay unit and one standpipe piezometer (SP14-29) installed in the till unit. Supplemental standpipe piezometer (SP16-04) was installed in the clay unit during the detailed design stage at the proposed CN railway/LDS pipe crossing.

Detailed logs for standpipe piezometers (SP14-01, 14-28 and 14-29) installed (within till and clay units) during preliminary design stage in intermediate test holes were documented in the AECOM report “Waverley Street Underpass-Upgrade- Preliminary Design Geotechnical Report”, dated January 2015. Detailed logs for standpipe piezometers (SP-04) installed (within clay unit) at the proposed CN railway/LDS pipe crossing were documented in the AECOM Memorandum “Geotechnical Investigation and Assessment for the proposed LDS/CN Track Crossing”, dated September 2016.

Laboratory testing was completed on selected samples and included moisture content, unit weight, gradation, Atterberg limits, undrained shear strength, consolidation test and uniaxial compressive strength for rock cores.

Drilling supervision was provided by AECOM personnel, who visually classified and logged soils, retrieved samples for laboratory testing, and supervised in-situ soil testing and standpipe piezometers installation. The approximate location of the test holes performed during preliminary and detailed design stages is shown on the Test Holes Location Plan (Figure 01) in Appendix A. Test hole logs have been prepared for each test hole to record the description and the relative position of the soil strata, location of samples obtained, seepage and sloughing conditions, field and laboratory test results, and other pertinent information. The test hole logs are attached in Appendix B. The laboratory test results are recorded on the test hole logs and are attached in Appendix C.

1.2 Subsurface Conditions

In descending order the soil profile consists of:

- Asphalt/concrete
- Fill;
- Glacio-Lacustrine Clay;
- Glacial Till; and
- Limestone Bedrock.

Each of these units is described below. Schematics of soil stratigraphy for deep test holes at the vicinity of proposed bridge based on conditions encountered during the investigation are presented on Schematic 01 and 02 in Appendix A. Soil properties from field and laboratory test results are presented on Figure 01.

Asphalt/Concrete

A layer of asphalt/ concrete was encountered within the first 2 to 2.5 m below ground surface in test holes (TH16-01 and 16-02).

Fill

Fill was encountered at the ground surface in all test holes and extended up to 1.5 m below ground surface. Two distinctive zones of fill were observed: an upper granular fill and lower clay fill.

The granular fill was 0.1 to 0.9 m thick and predominantly consisted of sand and gravel sizes, and contained variable amounts of silt, some clay and trace organic/rootlets. Cobbles and concrete debris were observed within the granular fill. The granular fill was light brown and dry to moist.

The clay fill, where encountered, was 0.2 to 1.4 m thick and contained variable amounts of silt, sand, organics, some to trace amounts of gravel and trace oxidation. The clay fill was dark grey to dark brown, moist, soft to stiff and was visually classified as of high to intermediate plasticity.

Glacio-Lacustrine Clay

In all test holes advanced past the fill zone, the fill was underlain by 10 to 11 m thick galcio-lacustrine silty clay. Generally, the clay was brown changing to grey with increasing depth, firm to stiff and becoming soft with increasing depth, moist and of high plasticity. Silt layer of about 1.0 m thick, firm to very soft, light grey to light brown and moist was observed in the upper as well as lower part of the clay unit.

Moisture contents ranged from 34 to 66 percent. The bulk unit weight of the clay was 16.9 kN/m^3 measured from one sample. Undrained shear strength measured from one unconfined compression test was 41 kPa.

Glacial Till (Silt)

In all test holes advanced past the clay, the clay was underlain by glacial till that typically contained variable amounts of clay, sand and gravel in silt matrix. Boulders and cobbles are known to be present within the till unit and were encountered during the drilling. Where the drilling advanced below the till unit, the thickness of the till layer varied from 4.8 to 8.2 m. The till was light grey, dense to very dense. Coring was necessary through very dense and boulders/cobbles in the lower zone of the till. The till was moist to wet, and of low plasticity. Measured moisture contents ranged from 7 to 21 percent.

Limestone Bedrock

The drilling was advanced past the till into the underlying limestone bedrock, which forms an artesian aquifer. The bedrock formation is a Paleozoic Carbonate rock formation known as the Upper Carbonate Aquifer. The following observations were recorded during the bedrock coring:

- Depth to bedrock surface ranged from 18 to 21 m below existing grade (Elev. 215.7 to 212.9 m).
- A layer of fine grained shale was encountered within the bedrock at depths ranged from 1.2 to 3.8 m below bedrock surface (Elev. 212.6 to 211.6) m in TH16-02, 16-03, 14-03 and 14-04. The thickness of the observed fine grained shale infill layers ranged from 0.3 to 0.8 m.
- Non intact zones and rock cores laminated with fine grained shale and hard clay were observed along the top 5.0 m of the bedrock deposit (Elev. 215.8 to 211.0 m).
- The top 5 m of the bedrock formation (Elev. 215.8 to 211.0 m) was observed as highly decomposed and based on the calculated RQD (Rock Quality Designation) values for the recovered rock cores, the rock quality was very poor to fair.

Uniaxial compressive strength tests completed on five samples of rock cores and the results are illustrated in Table 01 below. Photographs of the recovered rock cores are presented on Figures 02 and 03.

Limestone bedrock can contain zones/layers of poor fractured rock, fine grained infill, cavities, and other discontinuities that would be problematic to construction. Because these features occur unpredictably, it is not possible to fully identify their frequency or distribution during a geotechnical investigation.

Table 01: Uniaxial compressive strength test results for rock core samples

Test hole	Core No.	Depth below ground surface (m)	Compressive Strength (MPa)
TH16-01	C5	25.0	107
TH16-03	C10	23.5	145
TH14-02	C7	23.5	194
TH14-03	C7	23.5	121
TH14-04	C9	25.0	115

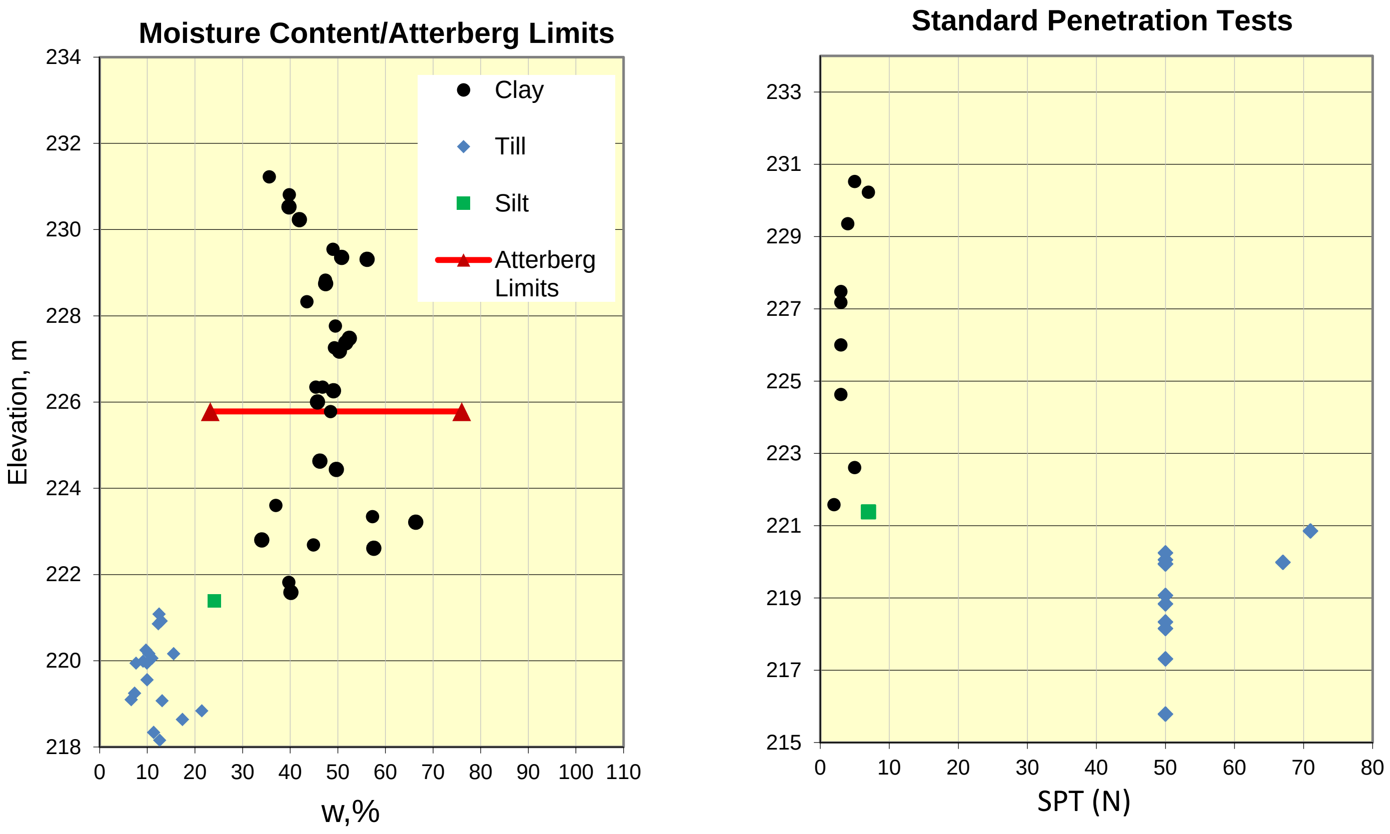

Figure 01 – Field and Laboratory Test Results

Figure 02a – Rock cores from (TH 16-01) – Detailed design stage

Figure 03b– Rock cores from (TH16-02) –Detailed design stage

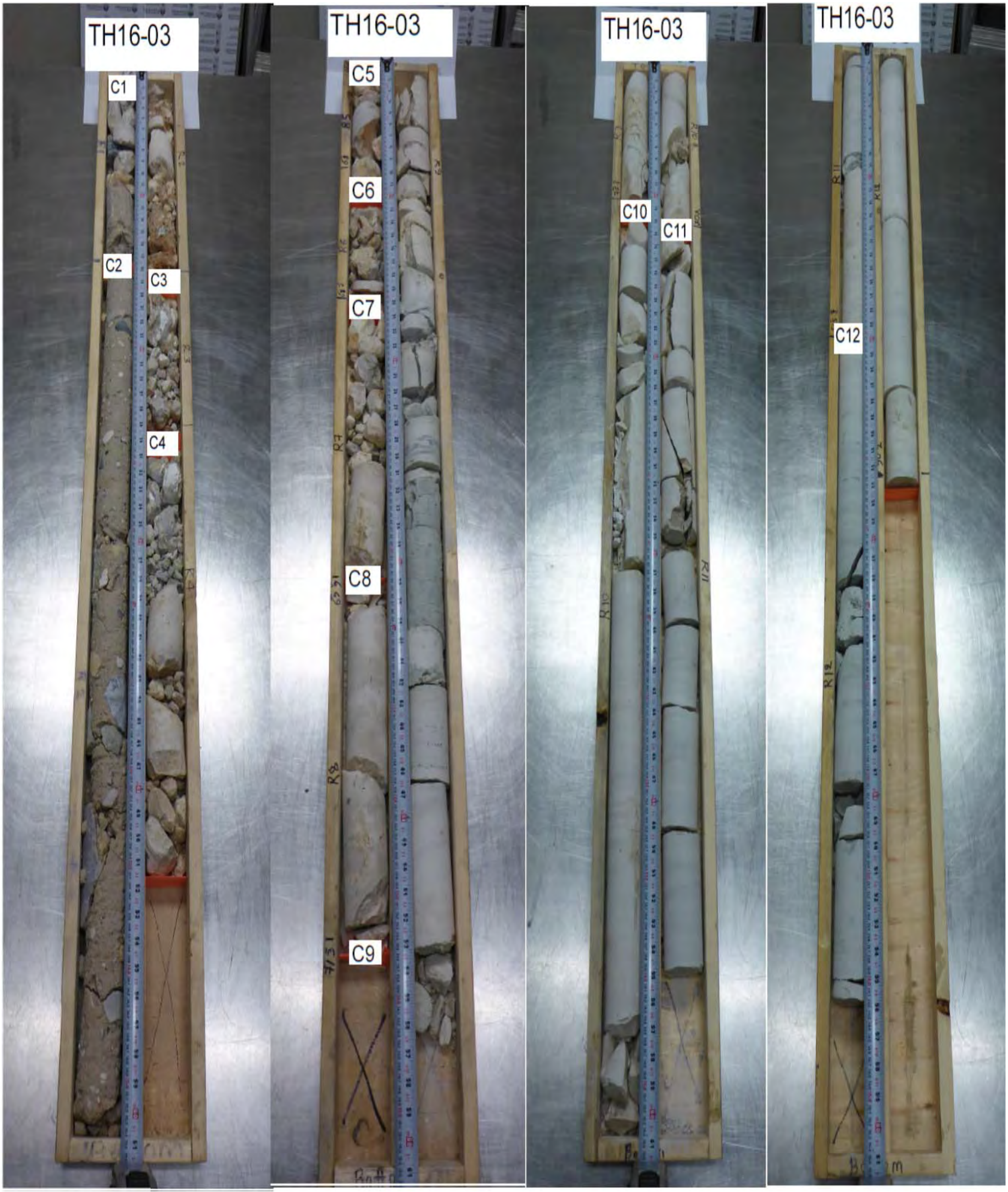

Figure 04c – Rock cores from (TH16-03) – Detailed design stage

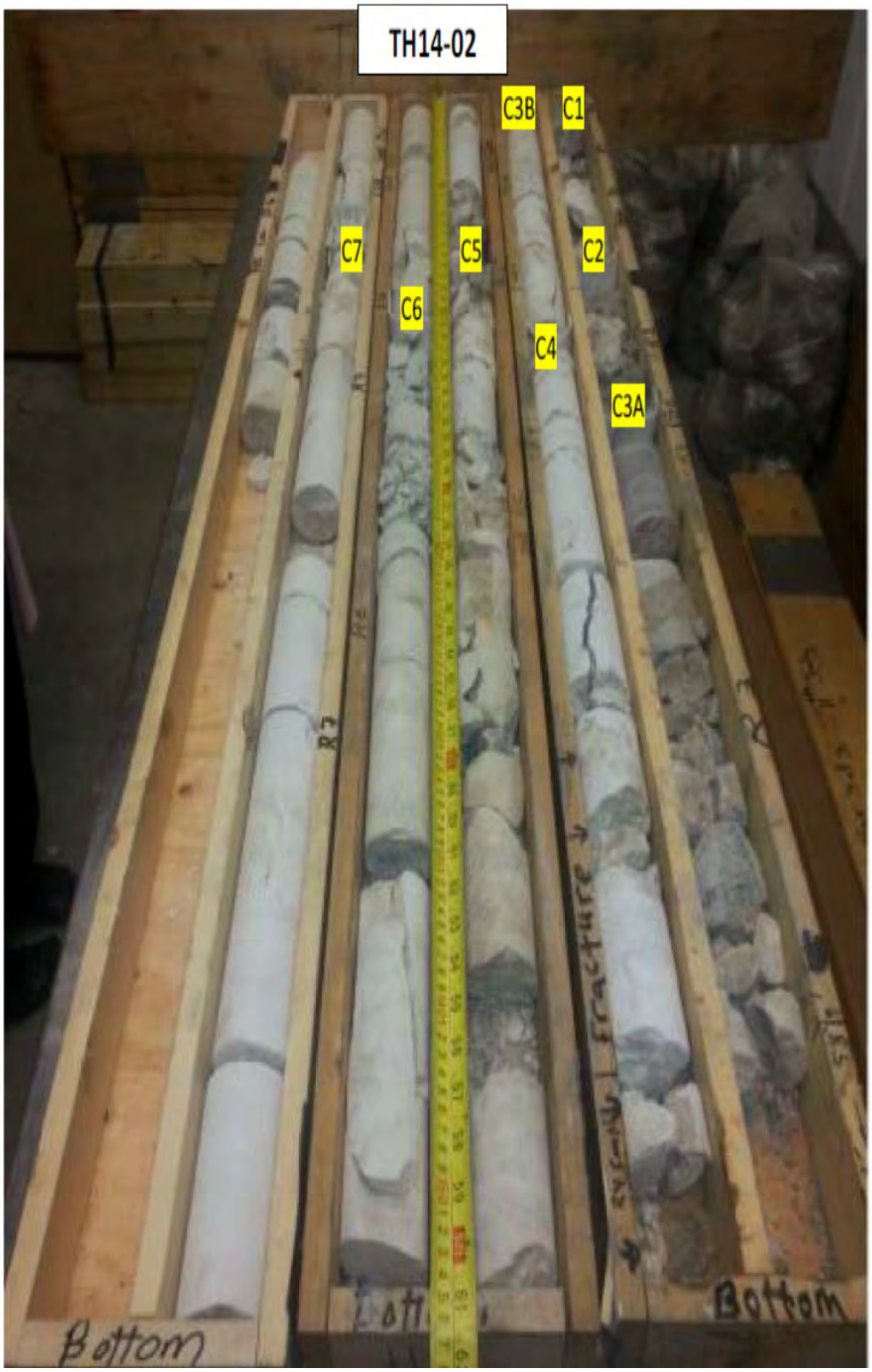

Figure 03a – Rock cores from (TH14-02) – Preliminary design stage

Figure 03b – Rock cores from (TH 14-03) – Preliminary design stage

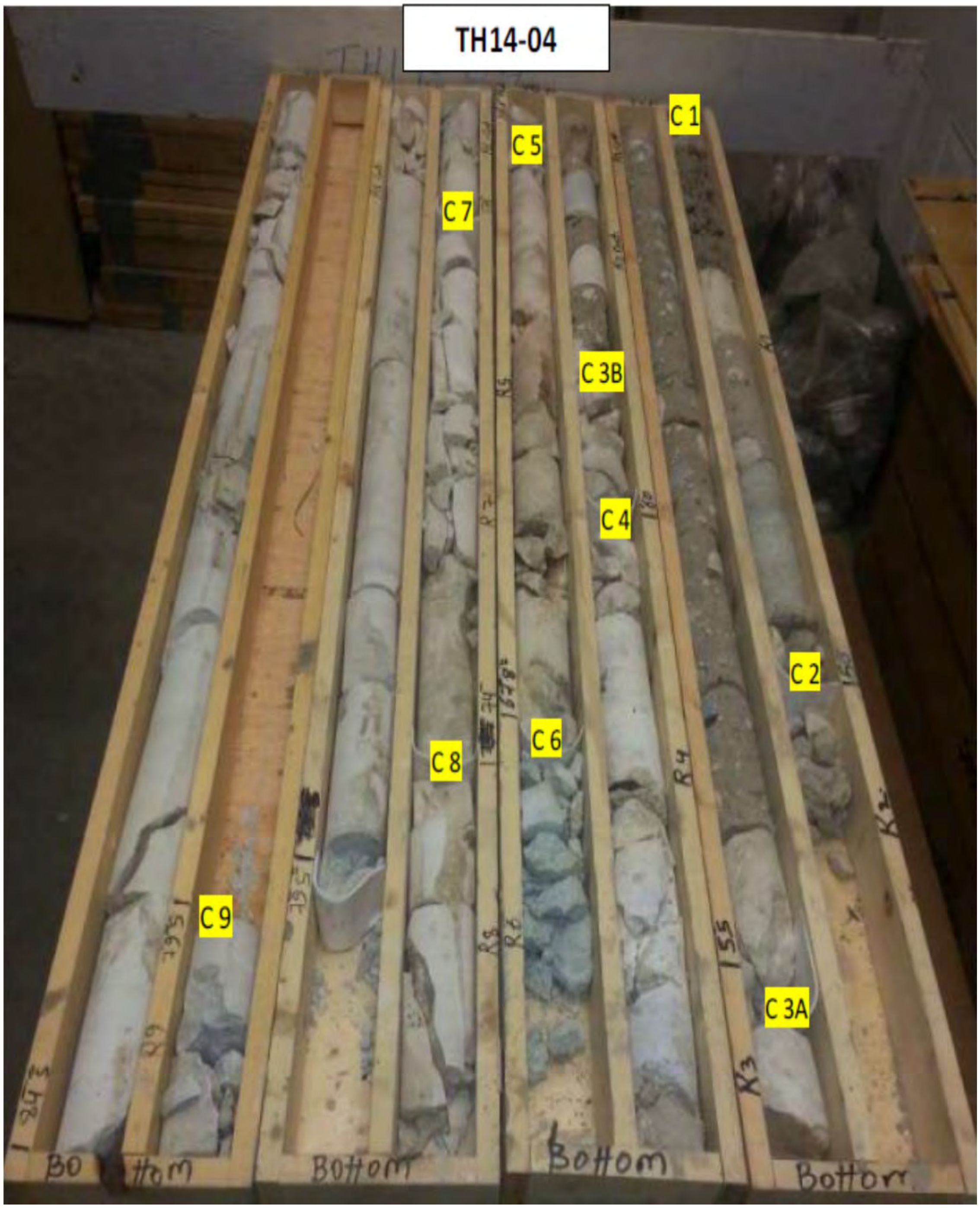

Figure 03c – Rock cores from (TH14-04) – Preliminary design stage

1.3 Groundwater Conditions

Monitoring results of the groundwater level (GWL) from the five standpipe piezometers installed at the site are presented in Table 02 and Figure 04. Groundwater levels will vary seasonally and from year to year or due to construction activities.

Based on the available monitoring results over 26 months, a GWL between elevation 224.4 and 225.9 m was recorded in the bedrock piezometers SP14-02 and 14-04. The till is considered to be hydraulically connected to the bedrock aquifer, monitoring results recorded for the till piezometer SP14-29 over 24 months ranged between elevation 224.9 and 226 m. Monitoring of clay piezometers SP14-01 and 14-28 over 26 months ranged between elevation 226.3 and 227.1 m, however a maximum GWL elevation of 229 m (i.e., GW about 4.6 m below existing grade) was recorded over a time window of approximately 3 months). GWL for the clay piezometer SP16-04 installed during the detailed design stage at the vicinity of the proposed CN Rail/LDS crossing was also monitored and recorded. Over 8 months of monitoring, the recorded groundwater elevations ranged between 230.2 and 231.1 m.

Monitoring results of two Provincial wells for bedrock aquifer GWL over the period from 2005 to 2016 are presented on Figure 05. The monitoring results from AECOM installation within the bedrock are in agreement with the data from well G05OC053 and are close to upper bound data from well G05OC008. Provincial wells G05OC008 and G05OC053 are located 0.35 km and 1.6 km away from CN Railway/Waverley Street crossing, respectively.

Table 02 – Summary of GWL Monitoring Results

Standpipe ID :	SP14-01	SP14-02	SP14-04	SP14-28	SP14-29	SP16-04
Soil/Bedrock Unit:	**Clay**	**Bedrock**	**Bedrock**	**Clay**	**Till**	**Clay**
Ground Surface Elevation (m):	**232.5**	**233.4**	**233.2**	**233.6**	**233.42**	**233.48**
12-Aug-14	225.10	225.2	225.2			
3-Sep-14	224.90	225.07	225.08			
19-Sep-14	225.55	225.5	225.55			
17-Oct-14	226.43	225.78	225.5			
6-Nov-14	226.55	225.65	225.4	226.3		
20-Nov-14	226.53	225.59	225.36	226.58		
6-Dec-14	226.4	225.4	225.23	226.6	225.27	
18-Dec-14	226.4	225.4	225.27	226.67	225.61	
9-Jan-15	226.4	225.4	225.26	226.64	225.63	
4-Feb-15	226.35	225.3	225.15	226.6	225.55	
24-Feb-15	226.32	225.25	225.13	226.53	225.51	
19-May-15	226.93	225.67	225.4	226.4	225.8	
30-Jun-15	226.55	225.28	225.05	226.5	225.65	
14-Aug-15	226.55	224.88	224.82	226.39	225.07	
28-Sep-15	226.45	225.1	225	226.4	225.25	
13-Nov-15	226.45	225.36	225.35	226.25	225.4	
23-Dec-15	226.46	225.45	225.37	226.3	225.44	
8-Feb-16	226.38	225.2	225.22	226.34	225.42	
18-Mar-16	226.5	-	225.5	226.5	225.6	
29-Apr-16	226.65	-	225.9	229	226	230.2
13-May-16	226.64	225.78	225.57	227.96	225.98	230.67
8-Jun-16	226.68	225.73	225.61	227.15	225.76	231.02
18-July-2016	226.68	225.54	225.28	226.9	225.48	231.08
30-August-2016	226.6	224.7	224.4	226.5	224.87	230.98
3-Nov-16	227.09	-	225.38	226.55	225.55	230.8

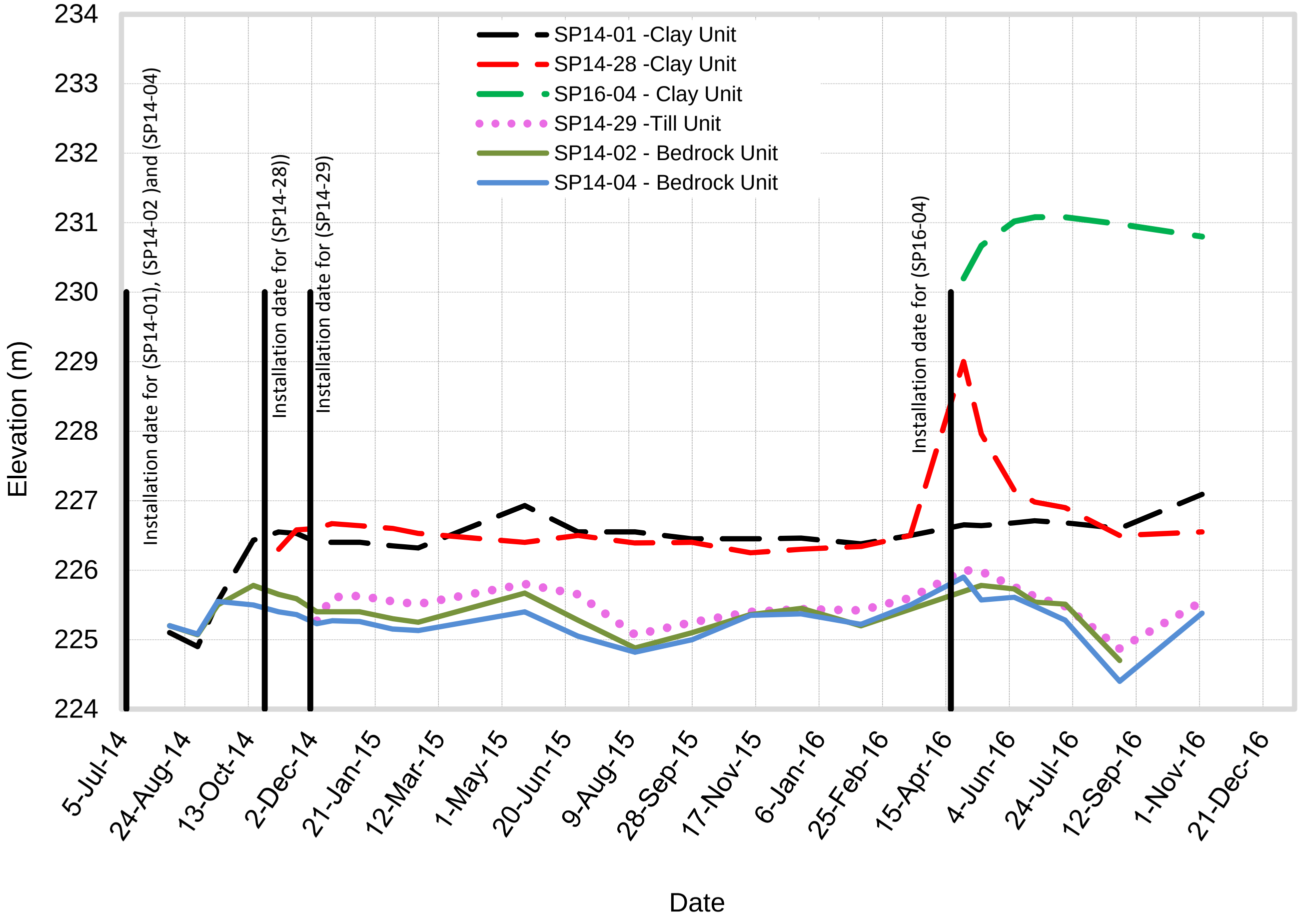

Figure 04 – Groundwater Monitoring Results for the Piezometers Installed during Preliminary and Detailed Design Stages

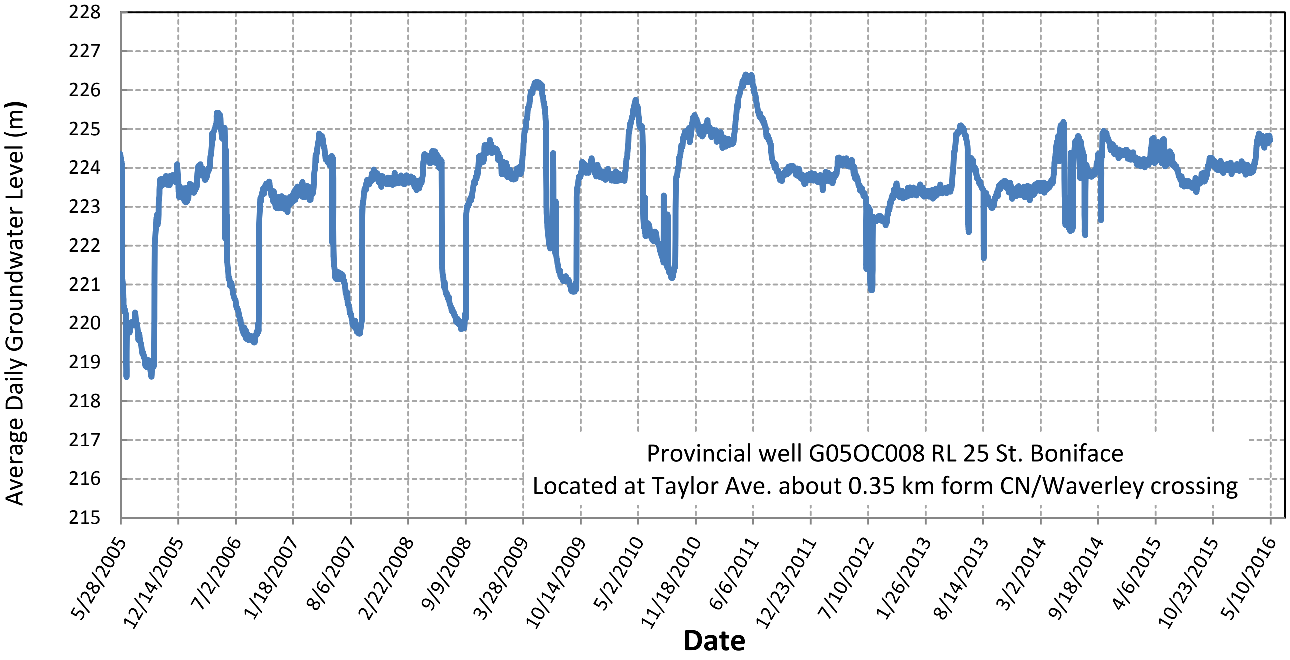

Figure 05a – Aquifer Groundwater Monitoring Results - Provincial Well G05OC008

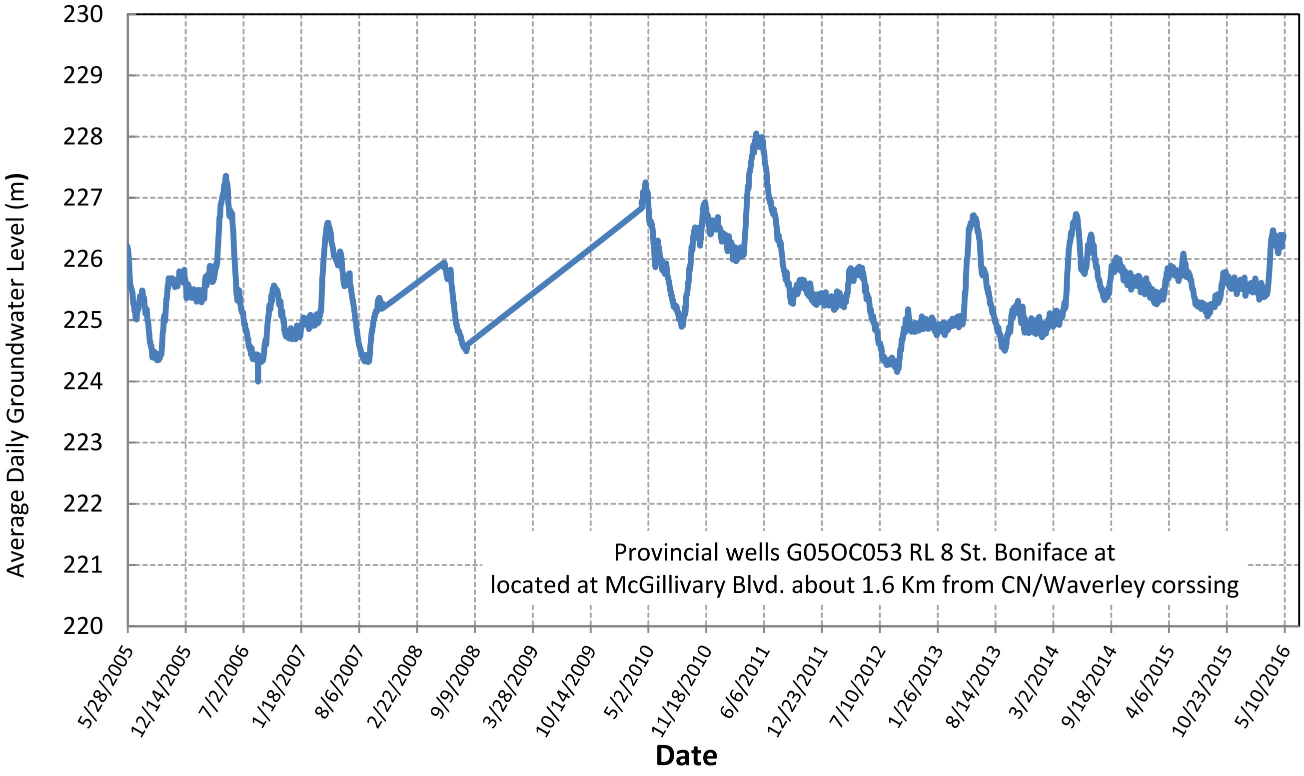

Figure 05b – Aquifer Groundwater Monitoring Results - Provincial Well G05OC053

1.4 Underpass Structure Foundations

Shallow foundations are not considered suitable to support heavily loaded structures. Deep foundations bearing on competent very dense till or bedrock will be required to support these structures. Available deep foundation alternatives include:

- Driven Pre-cast Pre-stressed Concrete Piles;
- Driven Steel Piles;
- Cast-in-Place Belled Caissons; and
- Cast-in-Place Rock Socketed Caissons.

AREMA Manual 2014 is referenced as the design code for the Underpass Structure.

Geotechnical recommendations pertaining to the design and construction of the Driven Pre-cast Pre-stressed Concrete Piles, Driven Steel Piles and Cast-in-Place Belled Caissons were provided in AECOM report “Waverley Street Underpass-Upgrade- Preliminary Design Geotechnical Report”, dated January 2015.

1.4.1 Cast-in-Place Rock Socketed Caissons

Drilled caissons socketed into competent bedrock can be designed to support the proposed piers Local practice is to design the drilled shafts based on values of maximum allowable end bearing and/or shaft adhesion of 3.0 and 1.0 MPa, respectively, provided that downhole inspection and assessment of the rock competency are undertaken. The assessment of the rock competency consists of small diameter proof drilling to a depth of 2 m below the socket base to detect the presence of voids or clay/silt layers of any significance and determine if deeper socket boring is required. In the event that the socket cannot be visually inspected, inspection of the recovered rock core and downhole video monitoring can confirm the competency of the bedrock. In this situation, caissons founded in competent bedrock should be designed on the basis of a reduced allowable shaft adhesion with no contribution from end bearing.

Safety concerns related to man entry into the hole (e.g., high level of gas) may preclude undertaking the visual inspection.

According to our knowledge, settlements of rock socketed caissons have never been measured in the Winnipeg area. However, it is anticipated that the settlements would be less than 20 mm.

Based on the finding from the six test holes (TH14-02 to 14-04 and TH16-01 to 16-03), that have been drilled during preliminary and detailed design stages, the top 5 m of the bedrock is of poor to very poor rock quality. A layer of clay/shale infill 0.3 to 0.8 m thick was encountered within the bedrock between elevation 212.6 to 211.6 m in TH16-02, 16-03, 14-03 and 14-04. The thickness of the fractured and heavily jointed bedrock is variable and could be in excess of 5 m and the clay infill may vary in thickness and could be encountered at different elevations. Socket length, should be developed below elevation 210.0 m and measures to maintain socket wall stability and groundwater control should be anticipated and undertaken. Competent bedrock was not encountered in some of the deep test holes below elevation 210.0 as the calculated RQD for the recovered rock cores ranged from 26 to 93 indicating poor to excellent rock quality. In this situation, the proposed caissons founded in fractured bedrock should be designed on the basis of a reduced allowable shaft adhesion of 0.45 MPa with no contribution from end bearing.

Inspection of the recovered rock cores by qualified and experienced geotechnical personnel and downhole video inspection will be required to aid in assessing the competency of the bedrock and determining if longer socket lengths are required. The depth to competent bedrock should be expected to vary across the site and it should be recognized that the presence of the heavily fractured rock and infill material above the socket length may require that a permanent steel casing be left in the ground so that the integrity of the shaft is maintained. In this regard, the basis for measurement and payment for the rock socket installation should be established in the contract preparation stage to recognize that the bedrock conditions at some rock socket locations may require unanticipated extra effort and materials for their completion.

The socket length should be a minimum of three socket diameter within competent bedrock. The minimum shaft diameter of the rock socket should not be less than 760 mm and the maximum diameter should be selected to suit the locally available coring equipment. The rock sockets should not be spaced closer than 3 socket diameters, centre to centre. Tremie placement of concrete is likely to be required.

1.5 Pile Lateral Capacity

Lateral forces acting on driven piles at the abutments locations should be resisted by using battered piles; battered piles can provide lateral resistance equal to the horizontal component of its axial load. Lateral resistance of vertical piles will depend on the pile head condition, the structural rigidity of the pile section and the soil strength.

Lateral pile response was analyzed using LPile software to determine pile top deflections and bending moments. The analysis considered a number of load increments between 50 and 150 kN (non-factored), the parameters used in the analysis are provided on Table 03.

The analysis was performed based on the foundation layout for the proposed bridge structure attached in Appendix A. The analysis assumed HP 360x132 and lateral force acting at the pile head. Two conditions were modeled, free head and fixed head condition. The pile length was assumed to be 16.5 m, (see Table 03) for abutments. The estimated lateral deflection and maximum moment at each condition are presented graphically on Figures 06 to 09.

Table 03: Soil Parameters for LPILE Analysis

Location	Pile Length (m)	Soil Unit	Depths (m)	LPILE Soil Type	Friction Angle (degree)	Undrained Shear Strength (kPa)	Effective Unit Weight (kN/m3)	ε_{50}
Abutments	16.5	Native Clay	0.0-11.5	Soft clay	--	25	6.5 -7.0	0.02
		Silt Till	11.5-16.5	Cemented c-phi Soil	30	50	10	0.01

The lateral capacity of individual piles in a group is primarily affected by the spacing of the piles, measured center to center in the direction of lateral load applied. Group effects diminish at a pile spacing of 6 pile diameters or greater in the direction of applied lateral load. Depending upon the pile spacing, it may be necessary to account for group effects along with other factors such as, group arrangement, as well as pile head fixity. Piles in a group may carry unequal lateral loads depending on their location within the group as well as the spacing between piles. This unequal distribution is caused by the overlap of shear zones and consequent reduction of soil resistance. As such, total lateral load applied to the pile cap should not assume to be distributed equally among the piles in a group.

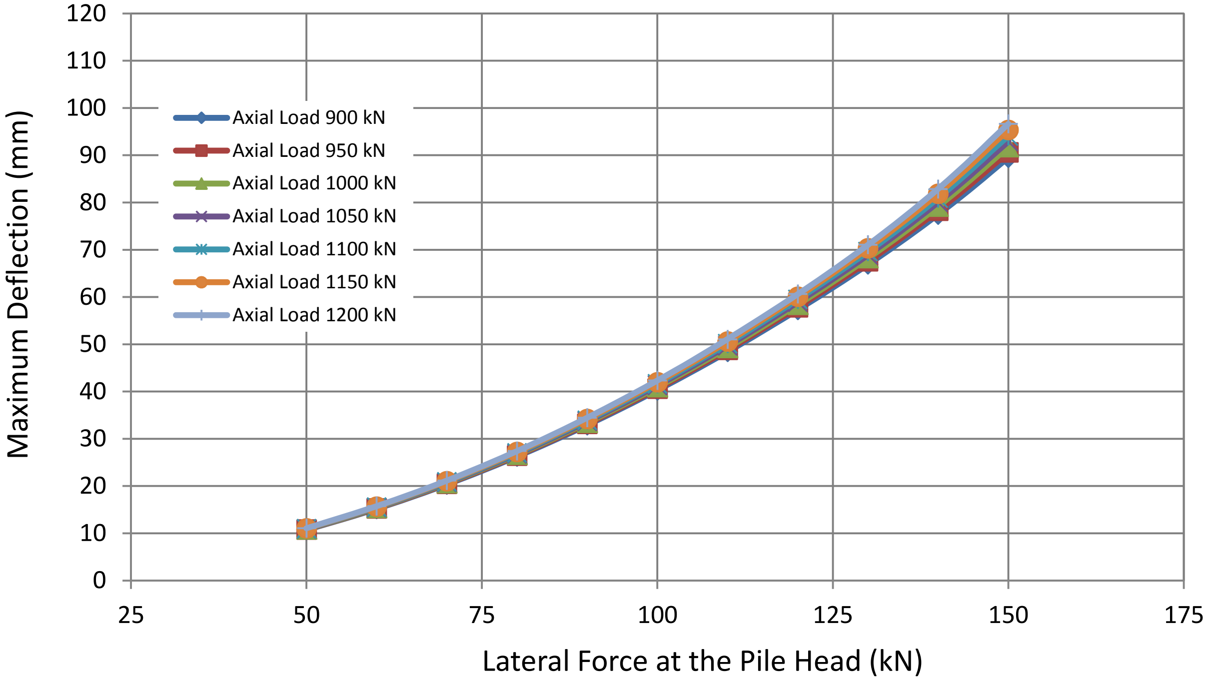

Figure 06: Maximum Lateral Deflection at Pile Head vs. Head Lateral Force (Free Head Condition)

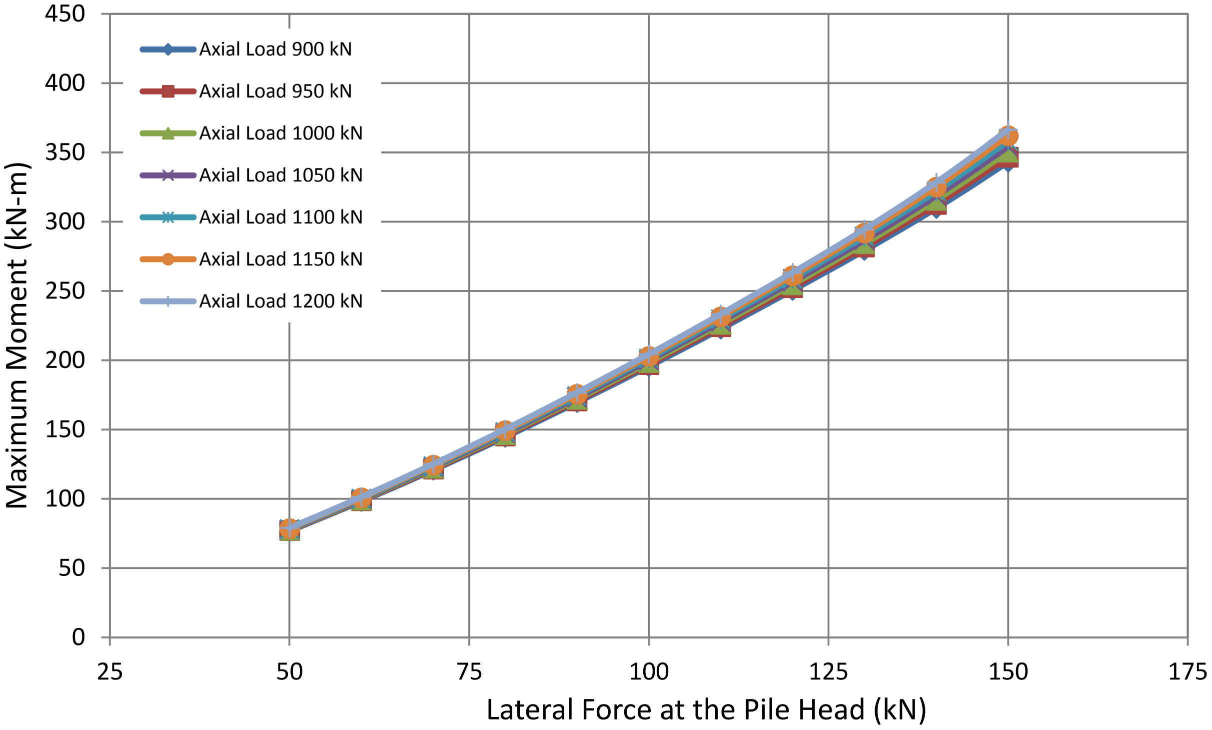

Figure 07: Maximum Bending Moment vs. Head Lateral Force (Free Head Condition)

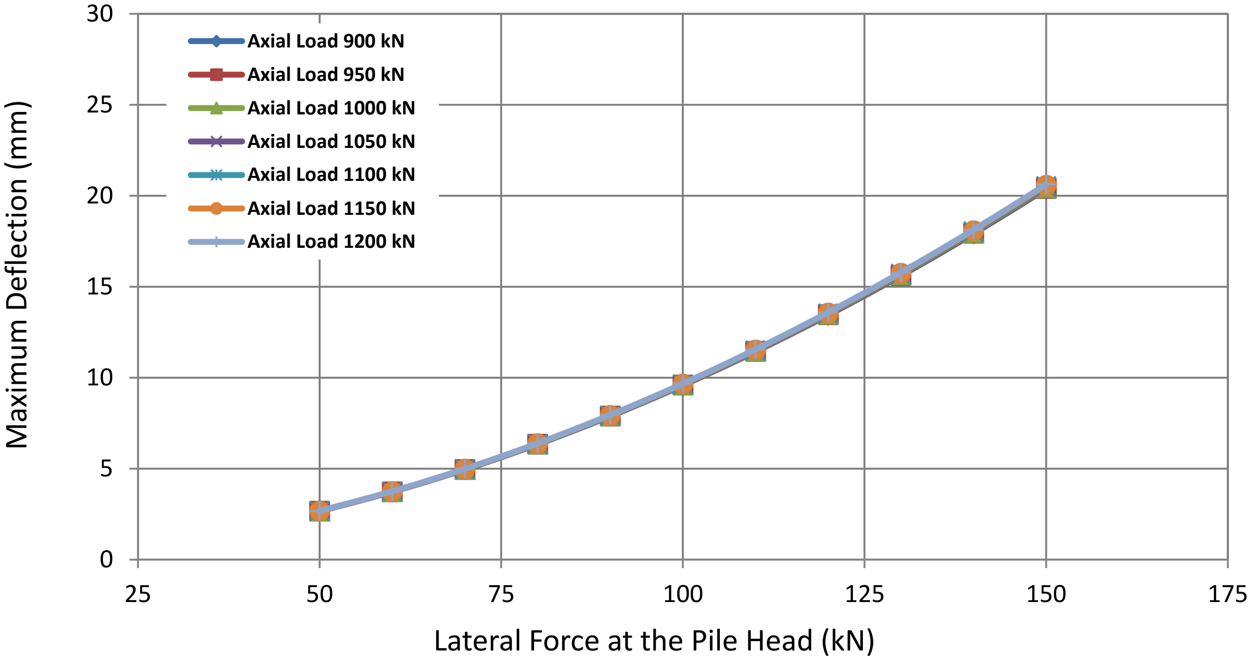

Figure 08: Maximum Lateral Deflection at Pile Head vs. Head Lateral Force (Fixed Head Condition)

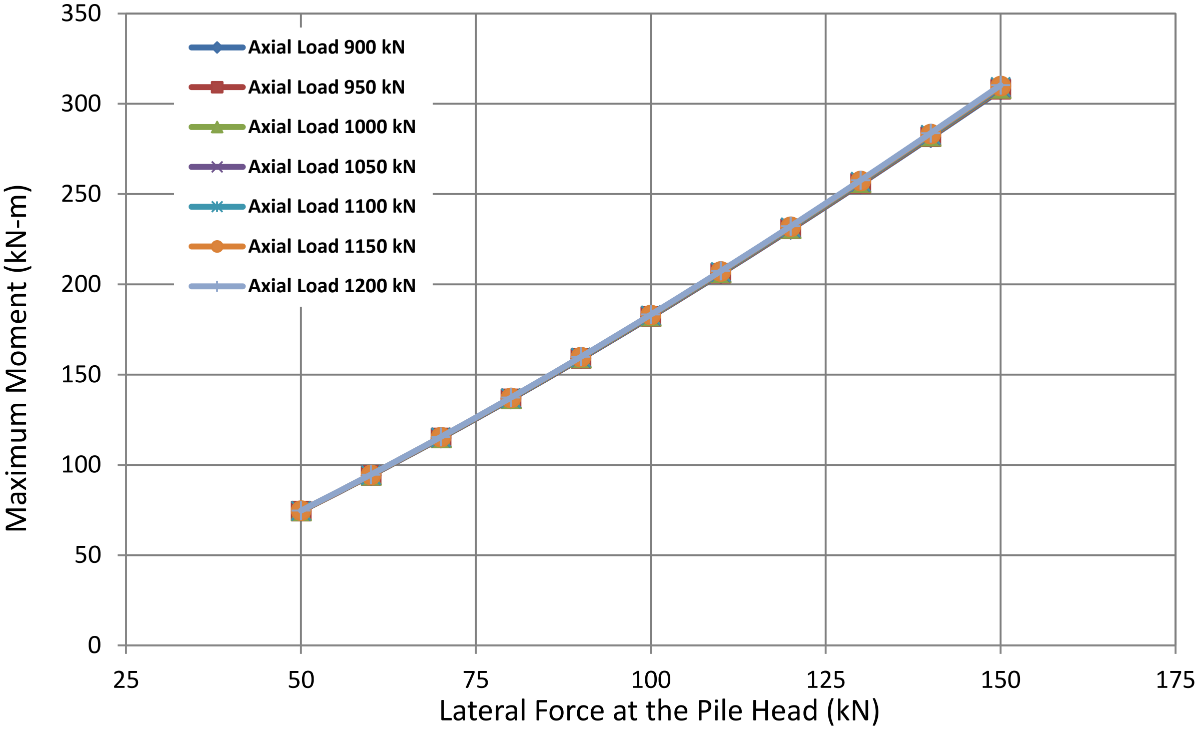

Figure 09: Maximum Bending Moment vs. Head Lateral Force (Fixed Head Condition)

Closure

The findings and recommendations of this memorandum were based on the results of field investigations, combined with an interpolation of soil and groundwater conditions between the test hole locations. If conditions are encountered that appear to be different from those shown by the test hole drilled at this site and described in this memorandum, or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and adjusted, if necessary.

Soil conditions, by their nature, can be highly variable across a site. The placement of fill and prior construction activities on a site can contribute to the variability especially near surface soil conditions. A contingency should be included in the construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.

We trust the information provided herein is sufficient for your purposes.

Please don't hesitate to contact me should you have any questions or concerns.

Submitted by:

S Ibrahim

Saba Ibrahim, M.Sc, P.Eng.
Geotechnical Engineer

Reviewed by:

Faris Alobaidy, M.Sc, P.Eng.(AB)
Senior Geotechnical Engineer

Appendix A

- Test Hole Location Plan
- Schematics Soil Stratigraphy
- Foundation Layout Figure

ANSI B 279.4mm x 431.8mm

Approved:

Checked:

Designer:

Project Management Initials:

Last saved by: MAHEC (2016-11-21) Last Plotted: never
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AECOM

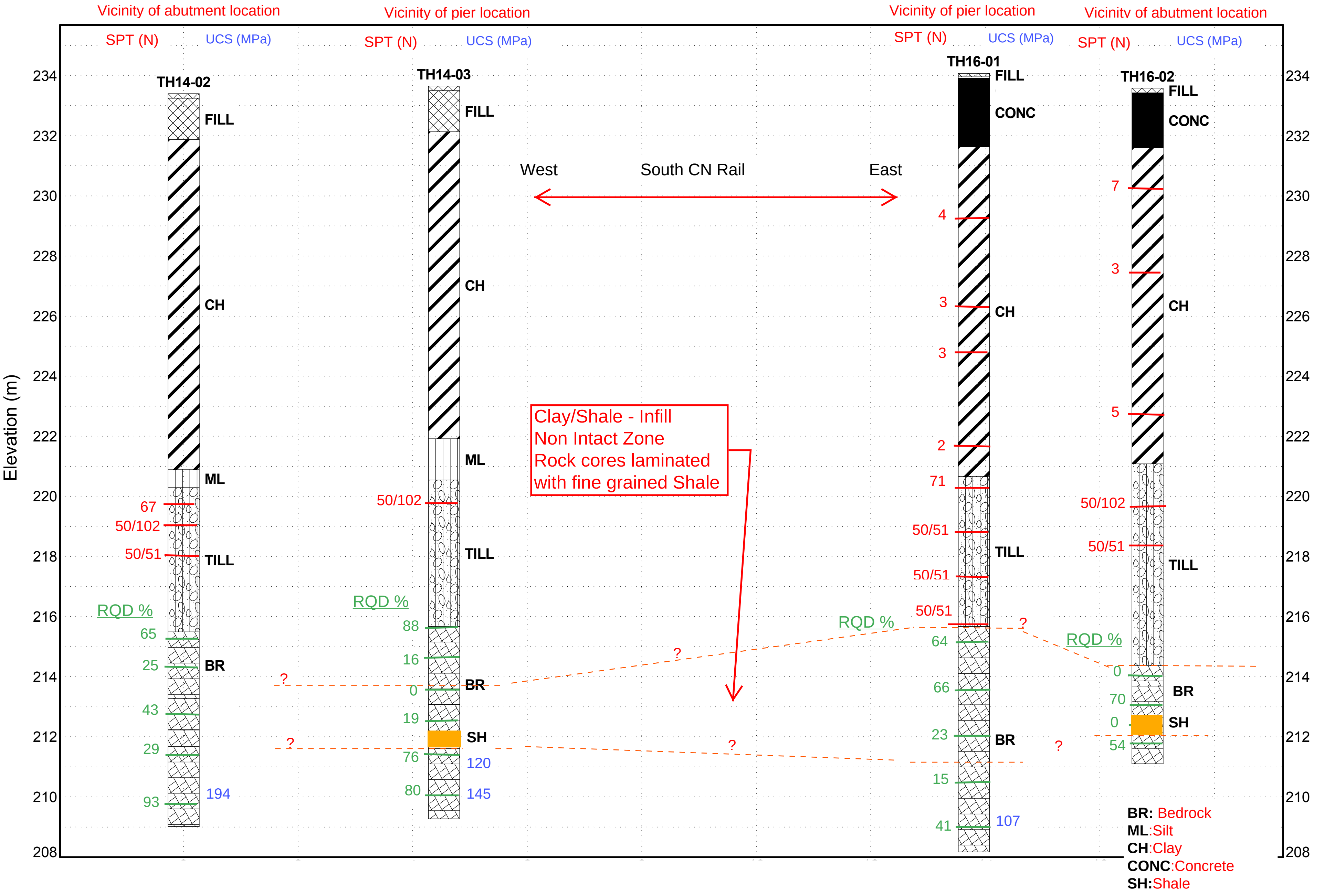
Figure: 01

Deep Test Hole and Standpipe Piezometer Location Plan

Waverley Underpass Detailed Design

Dillon Consulting Limited
Project No.: 60321148

CLIENT: Dillon Consulting Ltd. | PROJECT: Waverley Underpass - Detailed Design | PROJECT NO: 60321148

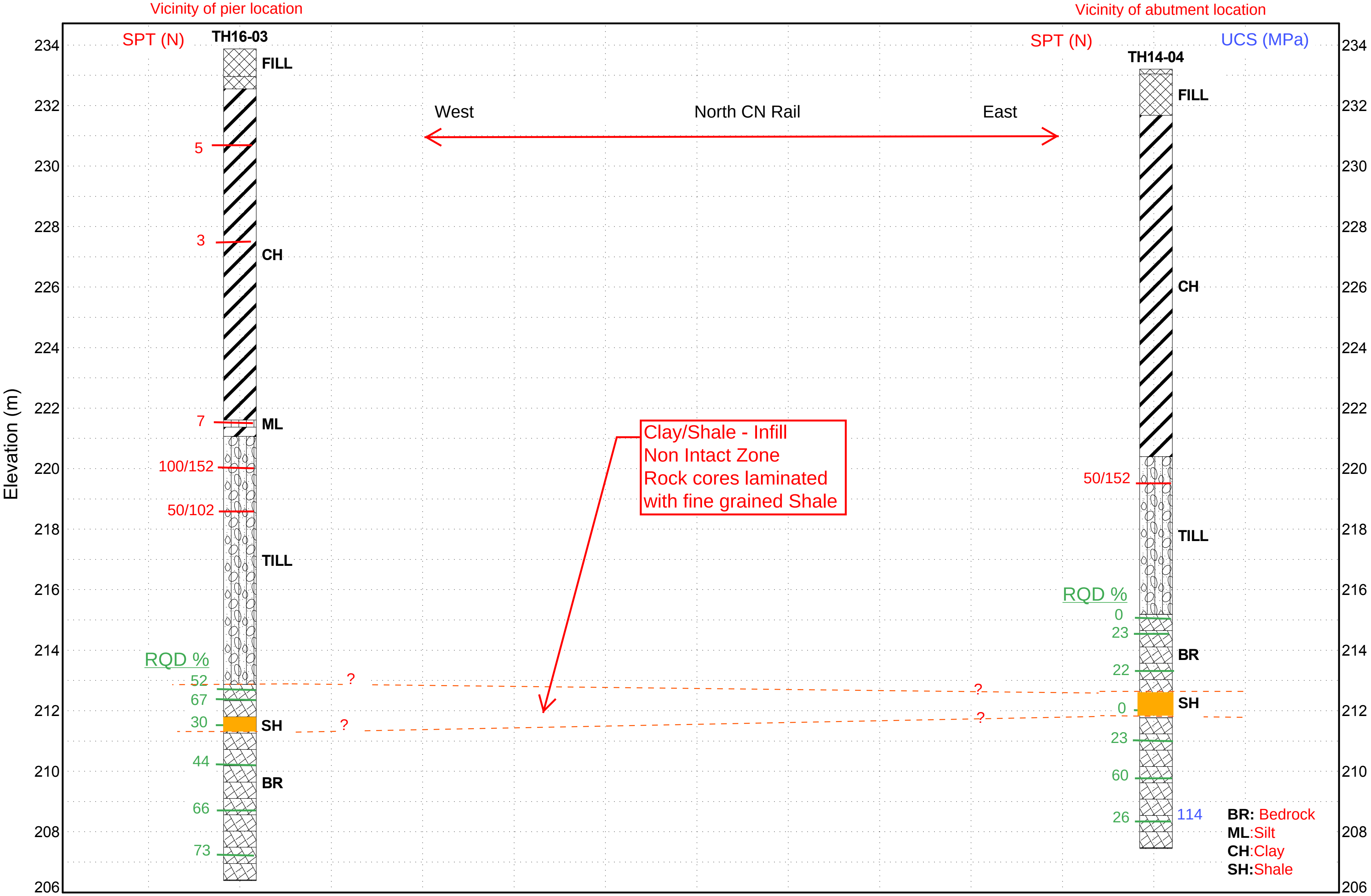


AECOM

Note: Horizontal distance not to scale

Schematic 02: Soil stratigraphy for deep test-holes

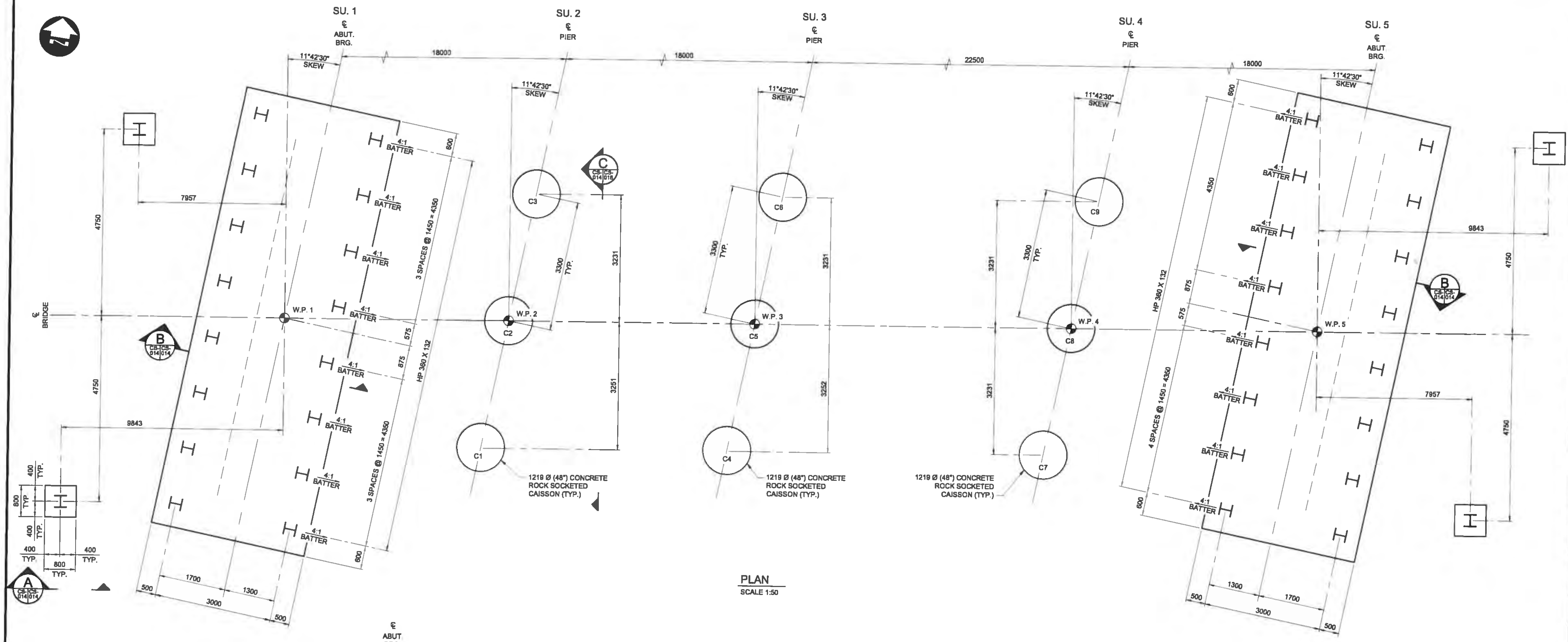
CLIENT: Dillon Consulting Ltd.	PROJECT: Waverley Underpass - Detailed Design	PROJECT NO: 60321148



Clay/Shale - Infill
Non Intact Zone
Rock cores laminated
with fine grained Shale

AECOM	Note: Horizontal distance not to scale	Schematic 01: Soil stratigraphy for deep test-holes

STRATIGRAPGHY EDIT WAVERLEY UP- TEST HOLE LOGS REV 01 - STRATIGRAPHY.GPJ UMA.GDT 6/2/16



PLAN
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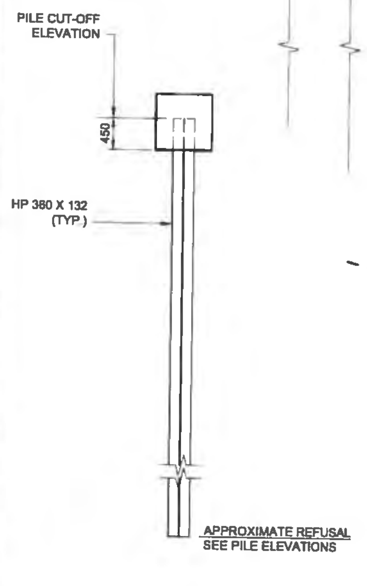

A SECTION
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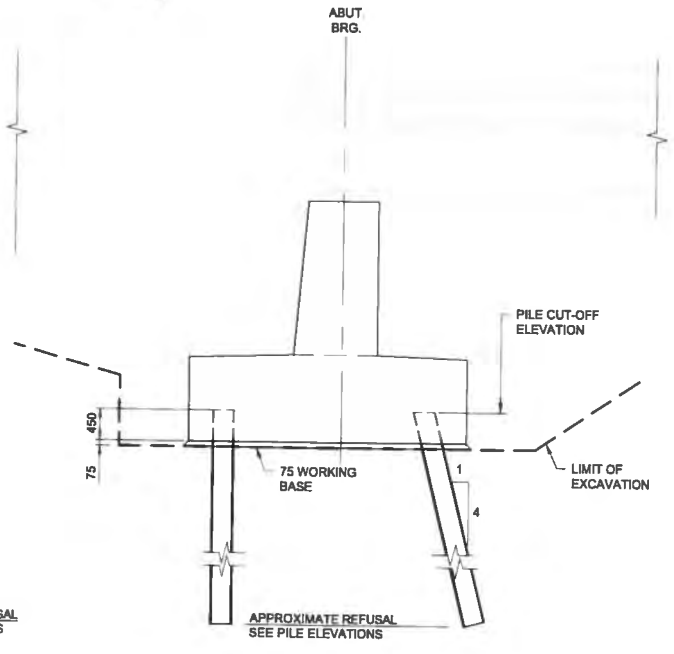
PILES AT WINGWALL



B SECTION
SCALE: 1:50
PILES AT ABUTMENT

BILL OF PILES

LOCATION	DESCRIPTION	No.	ESTIMATED LENGTH (m)	ESTIMATED TOTAL LENGTH (m)
SU. 1 (ABUTMENT)	STEEL PILES - HP 360X132 - VERTICAL	8	15.00	120.00
	STEEL PILES - HP 360X132 - BATTERED	8	16.00	128.00
SU. 1 (WINGWALL)	STEEL PILES - HP 360X132	2	18.00	36.00
SU. 2	STEEL PIPE PILES - 1219 O.D. x 19.1 THK - VERTICAL	3	23.00	69.00
SU. 3	STEEL PIPE PILES - 1219 O.D. x 19.1 THK - VERTICAL	3	23.00	69.00
SU. 4	STEEL PIPE PILES - 1219 O.D. x 19.1 THK - VERTICAL	3	23.00	69.00
SU. 5 (ABUTMENT)	STEEL PILES - HP 360X132 - VERTICAL	8	15.00	120.00
	STEEL PILES - HP 360X132 - BATTERED	8	16.00	128.00
SU. 5 (WINGWALL)	STEEL PILES - HP 360X132	2	18.00	36.00
SU. 1 & SU. 5	HARD-BITE POINT HP-77750-B FOR HP 360 x 132 PILES	36		-
SU. 2 TO SU. 4	OPEN-ENDED CUTTING SHOE FOR PIPE PILES	9		-
	TOTAL LENGTH OF HP 360 x 132 PILES			568.00 m
	TOTAL LENGTH 1219 O.D. x 19.1 THK PIPE PILES			207.00 m

PILE ELEVATIONS

LOCATION		DESCRIPTION	PILE CUT-OFF ELEVATION (m)	ESTIMATED PILE TIP ELEVATION (m)
SU. 1 (ABUTMENT)		HP 360X132	229.615	215.00
SU. 1 (WINGWALL)		HP 360X132	232.530	215.00
	C1	1219 O.D. CAISSONS	231.522	210.00
SU. 2	C2	1219 O.D. CAISSONS	231.440	210.00
	C3	1219 O.D. CAISSONS	231.522	210.00
	C4	1219 O.D. CAISSONS	231.612	210.00
SU. 3	C5	1219 O.D. CAISSONS	231.530	210.00
	C6	1219 O.D. CAISSONS	231.612	210.00
	C7	1219 O.D. CAISSONS	231.500	210.00
SU. 4	C8	1219 O.D. CAISSONS	231.418	210.00
	C9	1219 O.D. CAISSONS	231.500	210.00
SU. 5 (ABUTMENT)		HP 360X132	229.593	215.00
SU. 5 (WINGWALL)		HP 360X132	232.531	215.00

NOTE:

THE PILE AND CAISSON ELEVATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE ONLY. REFER TO THE TEST HOLE LOGS AND ALL OTHER AVAILABLE INFORMATION TO GAIN MORE KNOWLEDGE ABOUT THE SURFACE AND SUBSURFACE CONDITIONS.

PRELIMINARY ONLY
NOT FOR CONSTRUCTION

NO.	REVISIONS	DATE	BY
4	ISSUED FOR 85% UGS REVIEW	16/11/14	RE
3	ISSUED FOR 75% REVIEW	16/09/26	RE
2	ISSUED TO CN FOR 50% REVIEW	16/06/17	RE
1	ISSUED FOR 50% REVIEW	16/06/13	RE

DILLON CONSULTING

DESIGNED BY	RE	CHECKED BY	
DRAWN BY	CGC	APPROVED BY	
HOR. SCALE	AS SHOWN	RELEASED FOR CONSTRUCTION	
VERTICAL	AS SHOWN		
DATE		DATE	

ENGINEER'S SEAL

CONSULTANT PROJECT NUMBER
16-3353

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT

WAVERLEY STREET UNDERPASS AT CN MILE 3.89 RIVERS SUB
CONTRACT 2: UNDERPASS STRUCTURES, RAILWORKS, ROADWORKS, LAND DRAINAGE SEWER, PUMPING STATION AND LANDSCAPING WORKS

FOUNDATION LAYOUT

CITY DRAWING NUMBER
U-XXX-2016-C2-CS-014

SHEET 014 OF XXX

CONSULTANT DRAWING NUMBER
C2-CS-014

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Appendix B

Test Hole Logs

AECOM Canada Ltd.

GENERAL STATEMENT

NORMAL VARIABILITY OF SUBSURFACE CONDITIONS

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability for the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of earth work, foundations and similar. In the event of any changes in the basic design or location of the structures as outlined in this report or plan, we should be given the opportunity to review the changes and to modify or reaffirm in writing the conclusions and recommendations of this report.

The analysis and recommendations presented in this report are based on the data obtained from the borings and test pit excavations made at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere are not significantly different from those disclosed by the borings and excavations. However, variations in soil conditions may exist between the excavations and, also, general groundwater levels and conditions may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions differ from those encountered in the exploratory borings and excavations, are observed or encountered during construction, or appear to be present beneath or beyond excavations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Since it is possible for conditions to vary from those assumed in the analysis and upon which our conclusions and recommendations are based, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction procedures.

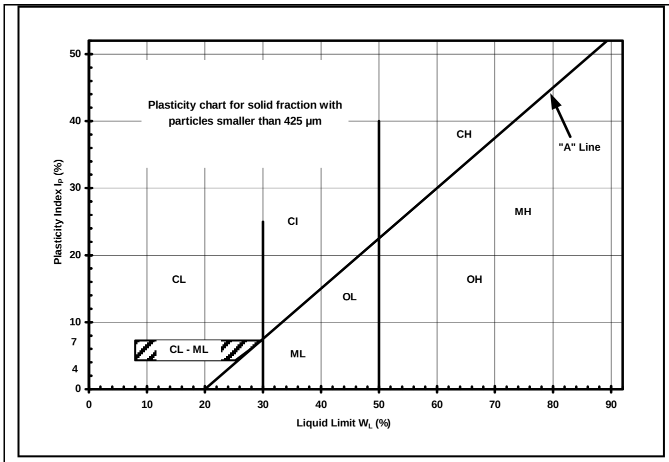
In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced soils engineer. We can be retained to provide these services for you during construction. In addition, we can be retained to review the plans and specifications that have been prepared to check for substantial conformance with the conclusions and recommendations contained in our report.

EXPLANATION OF FIELD & LABORATORY TEST DATA

Description				UMA Log Symbols	USCS Classification	Laboratory Classification Criteria			
						Fines (%)	Grading	Plasticity	Notes
COARSE GRAINED SOILS	GRAVELS (More than 50% of coarse fraction of gravel size)	CLEAN GRAVELS (Little or no fines)	Well graded gravels, sandy gravels, with little or no fines	◬	GW	0-5	$C_U > 4$ $1 < C_C < 3$		
			Poorly graded gravels, sandy gravels, with little or no fines	◭	GP	0-5	Not satisfying GW requirements		Dual symbols if 5-12% fines. Dual symbols if above “A” line and $4<W_P<7$ $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10} x D_{60}}$
		DIRTY GRAVELS (With some fines)	Silty gravels, silty sandy gravels	▥	GM	> 12		Atterberg limits below “A” line or $W_P<4$	
			Clayey gravels, clayey sandy gravels	▨	GC	> 12		Atterberg limits above “A” line or $W_P<7$	
	SANDS (More than 50% of coarse fraction of sand size)	CLEAN SANDS (Little or no fines)	Well graded sands, gravelly sands, with little or no fines	○	SW	0-5	$C_U > 6$ $1 < C_C < 3$		
			Poorly graded sands, gravelly sands, with little or no fines	○	SP	0-5	Not satisfying SW requirements		
		DIRTY SANDS (With some fines)	Silty sands, sand-silt mixtures	▥	SM	> 12		Atterberg limits below “A” line or $W_P<4$	
			Clayey sands, sand-clay mixtures	▨	SC	> 12		Atterberg limits above “A” line or $W_P<7$	
FINE GRAINED SOILS	SILTS (Below ‘A’ line negligible organic content)	$W_L<50$	Inorganic silts, silty or clayey fine sands, with slight plasticity	▥	ML			Classification is Based upon Plasticity Chart	
		$W_L>50$	Inorganic silts of high plasticity	▥	MH				
	CLAYS (Above ‘A’ line negligible organic content)	$W_L<30$	Inorganic clays, silty clays, sandy clays of low plasticity, lean clays	▨	CL				
		$30<W_L<50$	Inorganic clays and silty clays of medium plasticity	▨	CI				
		$W_L>50$	Inorganic clays of high plasticity, fat clays	▨	CH				
	ORGANIC SILTS & CLAYS (Below ‘A’ line)	$W_L<50$	Organic silts and organic silty clays of low plasticity	▥	OL				
		$W_L>50$	Organic clays of high plasticity	▨	OH				
HIGHLY ORGAINIC SOILS			Peat and other highly organic soils	≋	Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

■	Asphalt	▦	Till		
⊡	Concrete	▦	Bedrock (Undifferentiated)		
▩	Fill	▦	Bedrock (Limestone)		

When the above classification terms are used in this report or test hole logs, the designated fractions may be visually estimated and not measured.



FRACTION		SEIVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		Passing	Retained	Percent	Identifier
Gravel	Coarse	76	19	35-50	and
	Fine	19	4.75		
Sand	Coarse	4.75	2.00	20-35	“y” or “ey” *
	Medium	2.00	0.425		
	Fine	0.425	0.075	10-20	some
Silt (non-plastic) or Clay (plastic)		< 0.075 mm		1-10	trace

* for example: gravelly, sandy clayey, silty

Definition of Oversize Material

COBBLES: 76mm to 300mm diameter
BOULDERS: >300mm diameter

LEGEND OF SYMBOLS

Laboratory and field tests are identified as follows:

q_u - undrained shear strength (kPa) derived from unconfined compression testing.

T_v - undrained shear strength (kPa) measured using a torvane

pp - undrained shear strength (kPa) measured using a pocket penetrometer.

L_v - undrained shear strength (kPa) measured using a lab vane.

F_v - undrained shear strength (kPa) measured using a field vane.

γ - bulk unit weight (kN/m^3).

SPT - Standard Penetration Test. Recorded as number of blows (N) from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 51 mm O.D. Raymond type sampler 0.30 m into the soil.

DPPT - Drive Point Pentrometer Test. Recorded as number of blows from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 50 mm drive point 0.30 m into the soil.

w - moisture content (W_L, W_P)

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Su (kPa)	CONSISTENCY
<12	very soft
12 – 25	soft
25 – 50	medium or firm
50 – 100	stiff
100 – 200	very stiff
200	hard

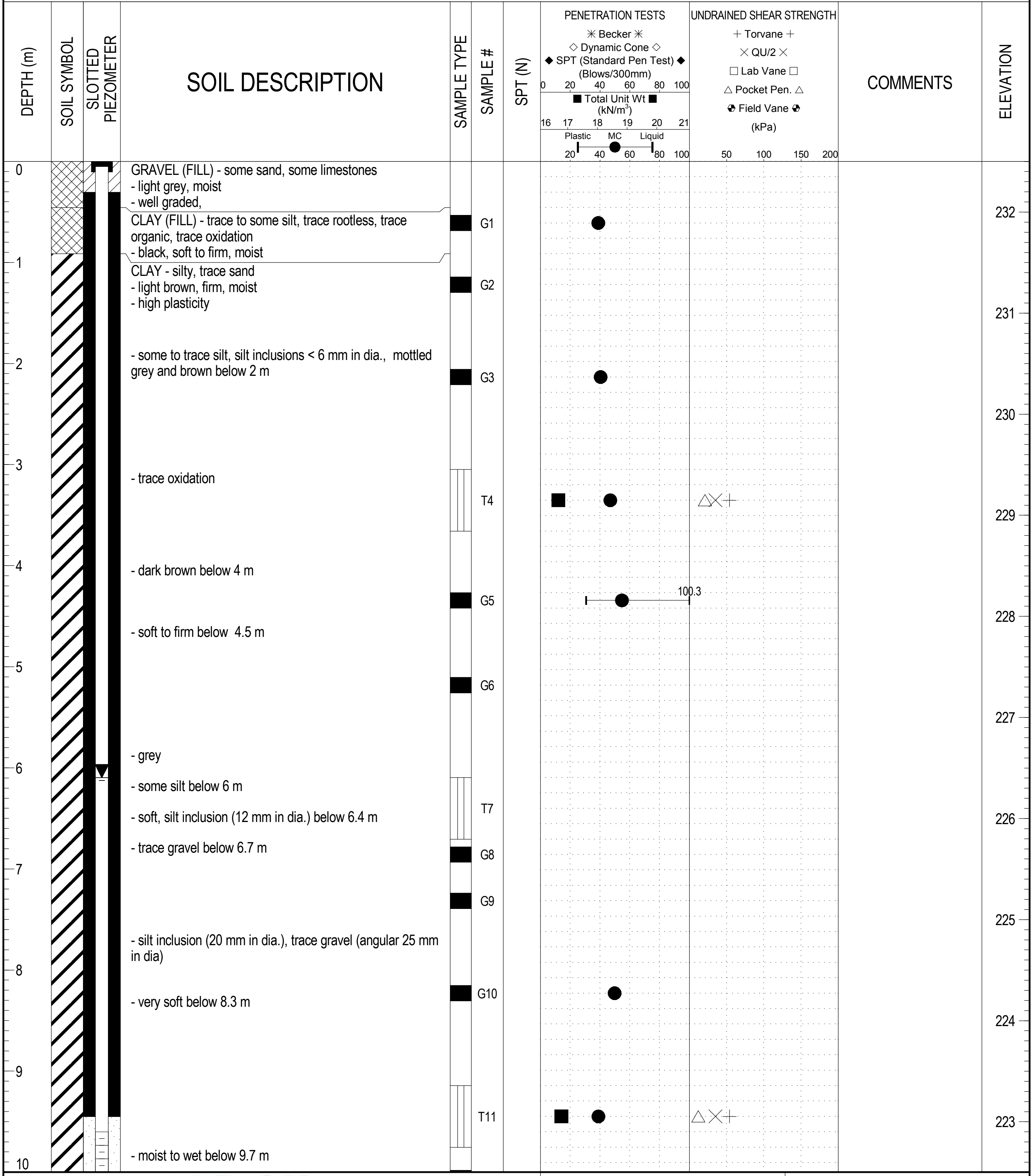
The resistance (N) of a non-cohesive soil can be related to compactness condition as follows

N – BLOWS/0.30 m	COMPACTNESS
0 - 4	very loose
4 - 10	loose
10 - 30	compact
30 - 50	dense
50	very dense

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-01**
LOCATION: UTM: 14U, 5523653 m N, 630934 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 232.50

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁚ GRAVEL ▥ SLOUGH ⁖ GROUT ▨ CUTTINGS ⁚ SAND



GRAVEL (FILL) - some sand, some limestones
- light grey, moist
- well graded,

CLAY (FILL) - trace to some silt, trace rootless, trace organic, trace oxidation
- black, soft to firm, moist

CLAY - silty, trace sand
- light brown, firm, moist
- high plasticity

- some to trace silt, silt inclusions < 6 mm in dia., mottled grey and brown below 2 m

- trace oxidation

- dark brown below 4 m

- soft to firm below 4.5 m

- grey

- some silt below 6 m

- soft, silt inclusion (12 mm in dia.) below 6.4 m

- trace gravel below 6.7 m

- silt inclusion (20 mm in dia.), trace gravel (angular 25 mm in dia)

- very soft below 8.3 m

- moist to wet below 9.7 m

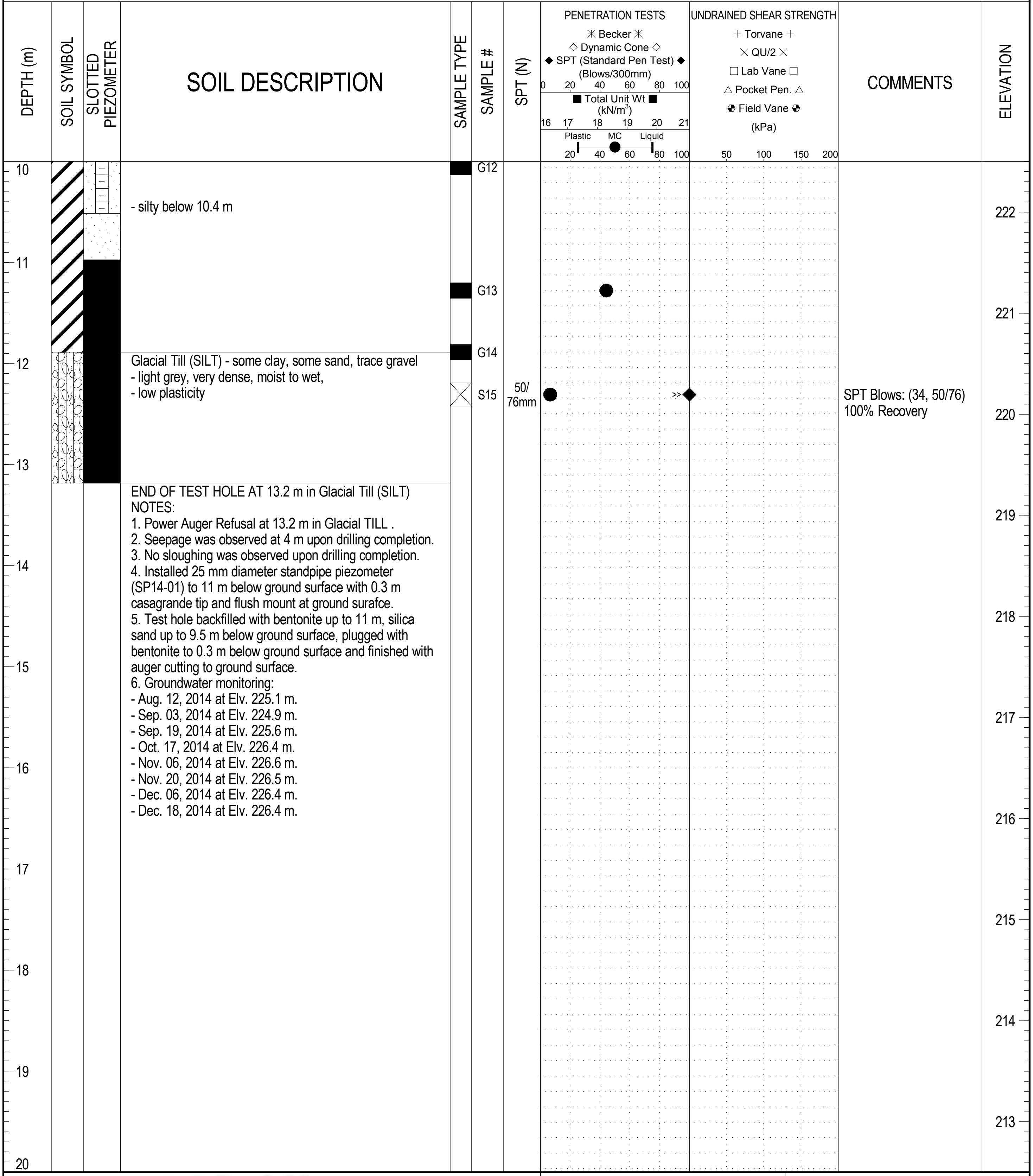
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 13.18 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/9/14
PROJECT ENGINEER: Faris Kahlil	Page 1 of 2

LOG OF TEST HOLE WAVERLEY UP- TEST HOLE LOGS - REVISION 5.GPJ UMA WINN.GDT 11/22/16

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-01**
LOCATION: UTM: 14U, 5523653 m N, 630934 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 232.50

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



- silty below 10.4 m

Glacial Till (SILT) - some clay, some sand, trace gravel
- light grey, very dense, moist to wet,
- low plasticity

SPT Blows: (34, 50/76)
100% Recovery

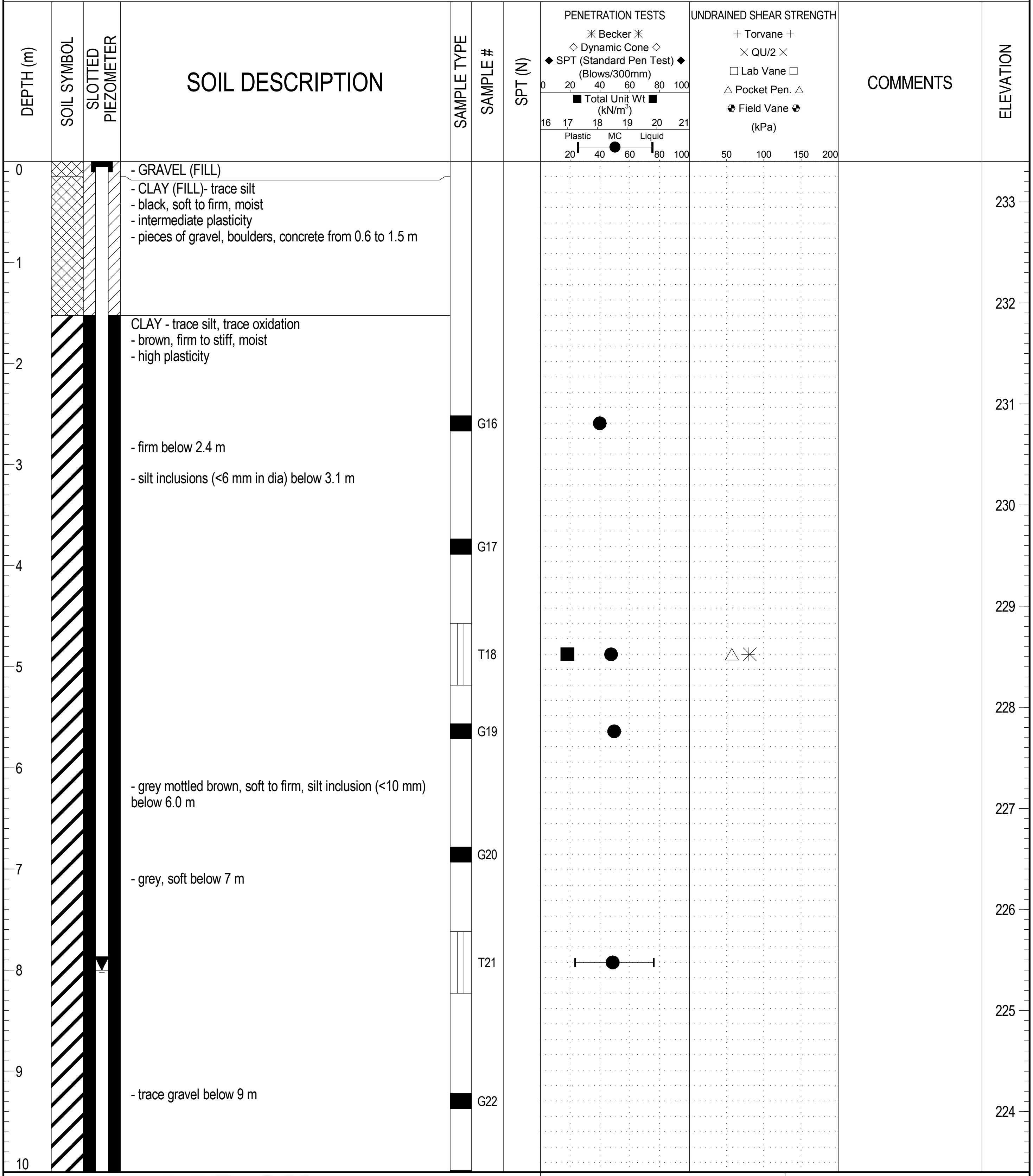
END OF TEST HOLE AT 13.2 m in Glacial Till (SILT)
NOTES:
1. Power Auger Refusal at 13.2 m in Glacial TILL .
2. Seepage was observed at 4 m upon drilling completion.
3. No sloughing was observed upon drilling completion.
4. Installed 25 mm diameter standpipe piezometer (SP14-01) to 11 m below ground surface with 0.3 m casagrande tip and flush mount at ground surafce.
5. Test hole backfilled with bentonite up to 11 m, silica sand up to 9.5 m below ground surface, plugged with bentonite to 0.3 m below ground surface and finished with auger cutting to ground surface.
6. Groundwater monitoring:
- Aug. 12, 2014 at Elv. 225.1 m.
- Sep. 03, 2014 at Elv. 224.9 m.
- Sep. 19, 2014 at Elv. 225.6 m.
- Oct. 17, 2014 at Elv. 226.4 m.
- Nov. 06, 2014 at Elv. 226.6 m.
- Nov. 20, 2014 at Elv. 226.5 m.
- Dec. 06, 2014 at Elv. 226.4 m.
- Dec. 18, 2014 at Elv. 226.4 m.

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 13.18 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/9/14
PROJECT ENGINEER: Faris Kahlil	Page 2 of 2

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-02**
LOCATION: UTM: 14U, 5523559 m N, 630870 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.40

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



- GRAVEL (FILL)

- CLAY (FILL)- trace silt
- black, soft to firm, moist
- intermediate plasticity
- pieces of gravel, boulders, concrete from 0.6 to 1.5 m

CLAY - trace silt, trace oxidation
- brown, firm to stiff, moist
- high plasticity

- firm below 2.4 m

- silt inclusions (<6 mm in dia) below 3.1 m

- grey mottled brown, soft to firm, silt inclusion (<10 mm) below 6.0 m

- grey, soft below 7 m

- trace gravel below 9 m

LOG OF TEST HOLE WAVERLEY UP- TEST HOLE LOGS - REVISION 5.GPJ UMA WINN.GDT 11/22/16

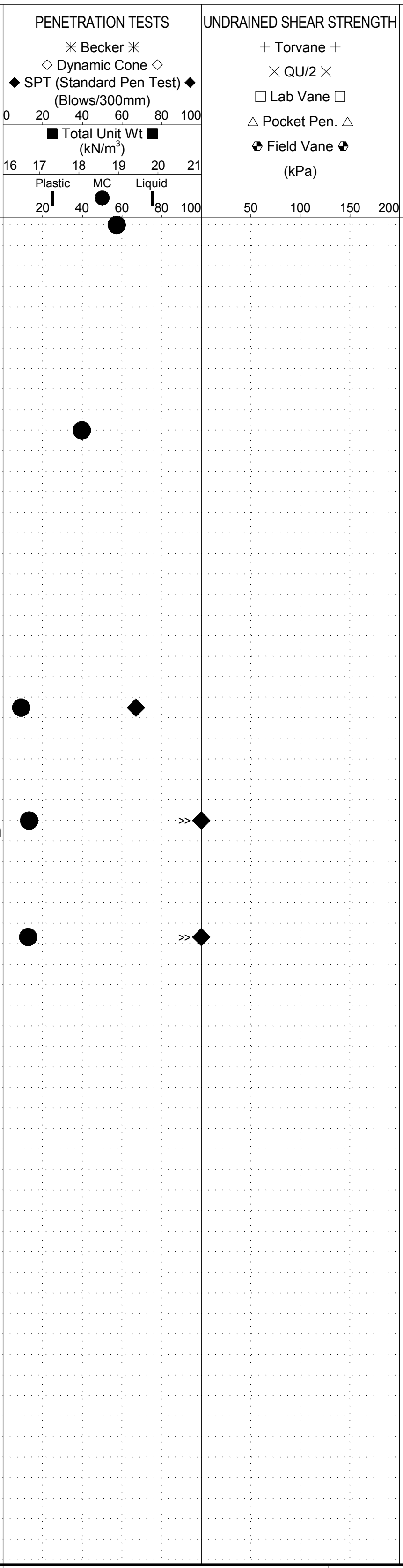
AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/11/14
PROJECT ENGINEER: Faris Kahlil	

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-02**
LOCATION: UTM: 14U, 5523559 m N, 630870 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.40

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10		■	G23			
	- silt inclusion (<30 mm in dia.) below 10.3 m					223
11		▥	T24			
	- some silt from 11.2 to 11.5 m					222
	- silty, light brown, soft , wet, low plasticity below 11.5 m	■	G25 G26			
12						
						221
	SILT -some gravel - light grey, very dense, moist to wet - low plasticity					
13		■	G27			
	Glacial Till (SILT)- some sand, some to trace gravel, trace clay - light grey, compact, moist to wet - low plasticity					220
		☒	S28	67	SPT Blows: (32, 43, 24) 61 % Recovery	
14						
	- ligth brown, some gravel below 14.4 m	☒	S29	50/ 102mm	SPT Blows: (35, 50/102) 89 % Recovery	219
	- trace gypsum					
15						
	- some gravel, some cobbles below 15.5 m	☒	S30	50/ 51mm	SPT Blows: (50/51) 100 % Recovery	218
16		▌	C1		C1 RQD: 0% C1 Recovery: 28 %	
						217
17	- sandy below 16.7 m	▌	C2		C2 RQD: 0% C2 Recovery: 100%	
		▌	C3A		C3A RQD: 0% C3A Recovery: 67%	216
18	LIMESTONE - fine grained, no foliation - creamish white - R3 - medium strong - close to moderately closed spacing,smooth, undulation, planar fractures, - no evidence of water flow (class 2) - fossiliferous - vuggy	▌	C3B		C3B RQD: 65% C3B Recovery: 100%	215
19		▌	C4		C4 RQD: 25% C4 Recovery: 90%	
						214
20						

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/11/14
PROJECT ENGINEER: Faris Kahlil	

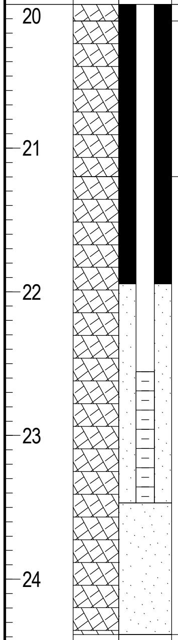
PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-02**
LOCATION: UTM: 14U, 5523559 m N, 630870 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.40

SAMPLE TYPE: ■ GRAB | ▥ SHELBY TUBE | ☒ SPLIT SPOON | ☰ BULK | ▨ NO RECOVERY | ▌ CORE

BACKFILL TYPE: ■ BENTONITE | ⁚ GRAVEL | ▥ SLOUGH | ⁖ GROUT | ▨ CUTTINGS | ⁚ SAND

PENETRATION TESTS: ✳ Becker ✳; ◇ Dynamic Cone ◇; ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane +; × QU/2 ×; □ Lab Vane □; △ Pocket Pen. △; ◉ Field Vane ◉ (kPa) 50 100 150 200



DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
20	- altered yellow and red below 20 m - extremely close to moderately closed spaced, smooth planar fractures - evidence of water flow (class 3)	▌	C5		C5 RQD: 43% C5 Recovery: 98%	213
21	- laminated below 21.2 m - close spaced to moderately closed spaced, smooth planar fractures, - no evidence of water flow (class 2)	▌	C6		C6 RQD: 29% C6 Recovery: 75 %	212 211
23	- R5- very strong	▌	C7		C7 RQD: 93% C7 Recovery: 100 %, qu = 194.4 MPa	210 209

END OF TEST HOLE AT 24.4 m IN BEDROCK
Notes:
1. Power Auger Refusal at 15.4 m in Glacial TILL.
2. HQ coring below 15.4 m.
3. Seepage observed at 3.0 m upon drilling completion.
4. Installed 25 mm diameter standpipe piezometer (SP14-02) to 23.5 m below ground surface with 0.3 m casagrande tip and flush mount at ground surface.
5.Test hole backfilled with silica sand up to 22 m below ground surface, bentonite up to 1.5 m and plugged with auger cutting to ground surface.
6. Prominent sub-vertical fracture (180 degrees to core axis), closed to gapped, smooth undulating, evidence of water flow (class 3) between 17.9 to 18.4 m.
7. Groundwater monitoring:
- Aug. 12, 2014 at Elv. 225.29 m.
- Sep. 03, 2014 at Elv. 225.0 m.
- Sep. 19, 2014 at Elv. 225.5 m.
- Oct. 17, 2014 at Elv. 225.8 m.
- Nov. 06, 2014 at Elv. 225.7 m
- Nov. 20, 2014 at Elv. 225.6 m
- Dec. 06, 2014 at Elv. 225.4 m
- Dec. 18, 2014 at Elv. 225.4 m

DEPTH (m): 25 26 27 28 29 30
ELEVATION: 208 207 206 205 204

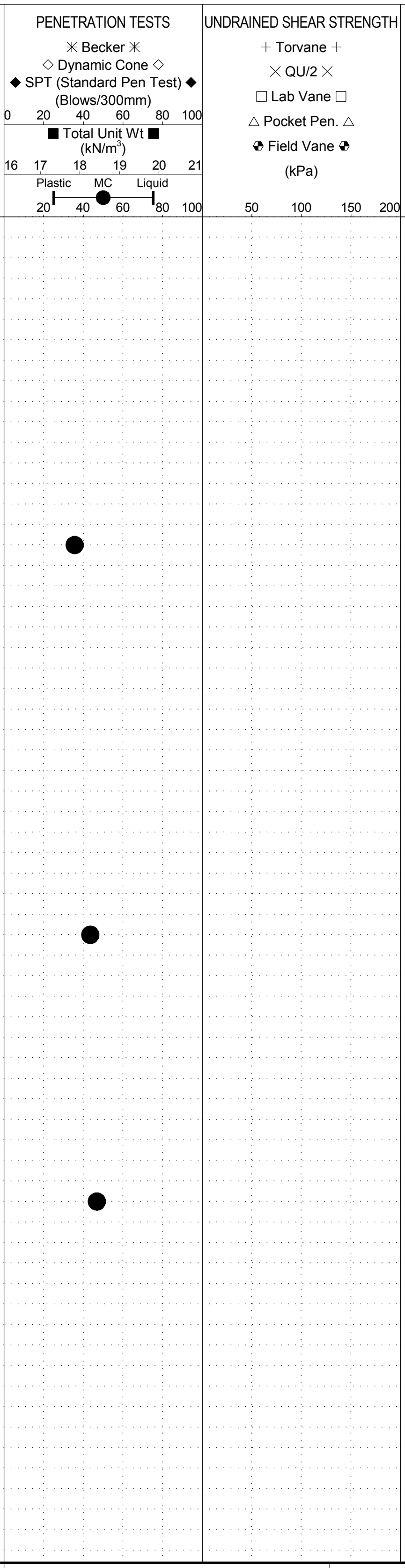
AECOM

LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 7/11/14
PROJECT ENGINEER: Faris Kahlil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-03**
LOCATION: UTM: 14U, 5523562 m N, 630895 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.66

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▐ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	ELEVATION
0	-GRAVEL (FILL)			
	- CLAY (FILL)-trace silt - black, soft to firm, moist - intermediate plasticity - pieces of gravel, boulders, concrete from 0.6 to 1.5 m			233
1				
	CLAY - some silt, trace oxidation - dark brown, firm to stiff, moist - intermediate to high plasticity - silt inclusion (<12 mm in dia.) - brown mottled grey below 2.1 m			232
2		■	G31	231
3		▥	T32	230
	- brown, high plasticity, firm below 3.7 m			
4	- dark brown below 4.6 m			229
5	- firm , trace gypsum below 5.2 m	■	G33	228
6		▥	T34	227
7	- soft to firm, dark brown, trace gravel below 7 m	■	G35	226
	- grey, soft, silt inclusion (6-30 mm in dia.) below 7.6 m			
8		■	G36	225
9		▥	T37	224
10				



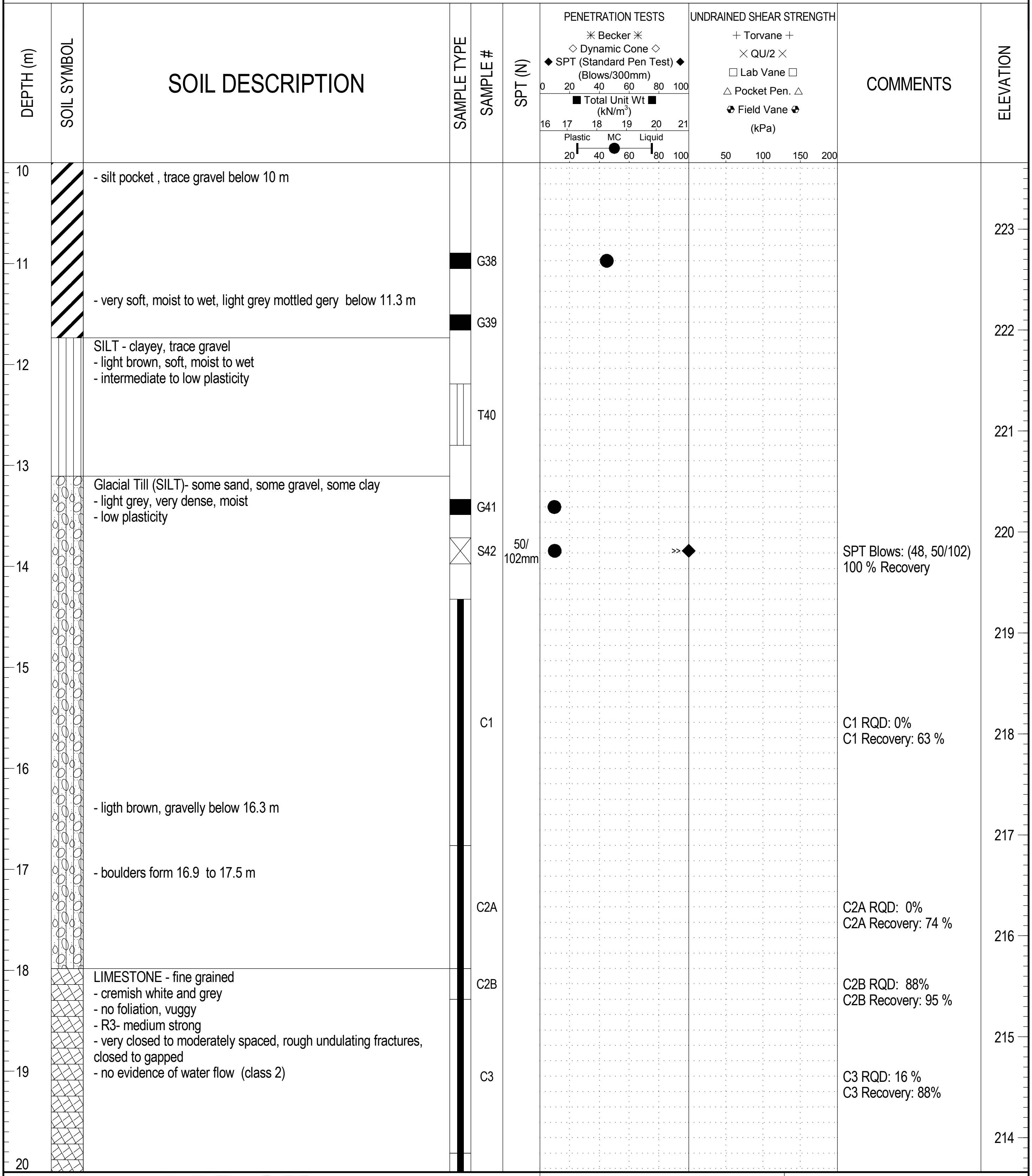
SPT (N) | COMMENTS

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/14/14
PROJECT ENGINEER: Faris Kahlil	Page 1 of 3

AECOM

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-03**
LOCATION: UTM: 14U, 5523562 m N, 630895 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.66

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▌ CORE



- silt pocket , trace gravel below 10 m

- very soft, moist to wet, light grey mottled gery below 11.3 m

SILT - clayey, trace gravel
- light brown, soft, moist to wet
- intermediate to low plasticity

Glacial Till (SILT)- some sand, some gravel, some clay
- light grey, very dense, moist
- low plasticity

- ligth brown, gravelly below 16.3 m

- boulders form 16.9 to 17.5 m

LIMESTONE - fine grained
- cremish white and grey
- no foliation, vuggy
- R3- medium strong
- very closed to moderately spaced, rough undulating fractures, closed to gapped
- no evidence of water flow (class 2)

Sample #	SPT (N)	Comments
G38		
G39		
T40		
G41		
S42	50/ 102mm	SPT Blows: (48, 50/102) 100 % Recovery
C1		C1 RQD: 0% C1 Recovery: 63 %
C2A		C2A RQD: 0% C2A Recovery: 74 %
C2B		C2B RQD: 88% C2B Recovery: 95 %
C3		C3 RQD: 16 % C3 Recovery: 88%

AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/14/14
PROJECT ENGINEER: Faris Kahlil	Page 2 of 3

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-03**
LOCATION: UTM: 14U, 5523562 m N, 630895 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.66

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ⧄ NO RECOVERY ▮ CORE

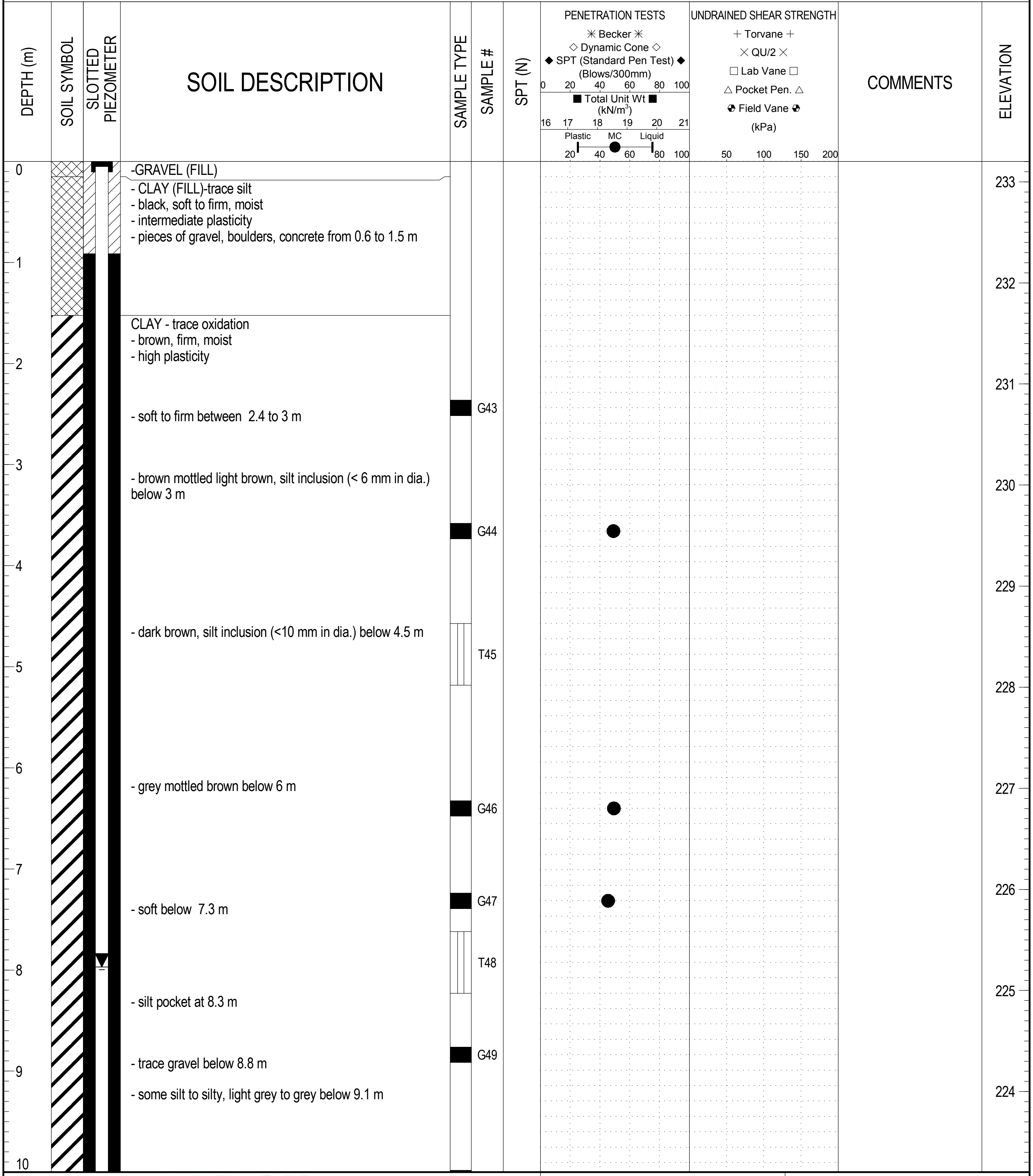
DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
20		- recovered as coarse, sub angular to sub rounded light grey gravel between 20.3 to 21.9 m	▮	C4				C4 RQD: 0% C4 Recovery: 100%	213
21			▮	C5				C5 RQD: 19% C5 Recovery: 68 %	212
22		SHALE - very fine grained - blue, green - no foliation - R1- very weak - extremely close spaced, rough undulating fractures	▮	C6				C6 RQD: 76% C6 Recovery: 100 %	211
23		LIMESTONE - white - fine grained - no foliation - R3- medium strong - close to moderately spaced, smooth fractures, closed, no evidence of water flow (class 2) - laminated below 22 m	▮	C7				C7 RQD: 80% C7 Recovery: 100 %	210
24		- R5- very strong	▮					qu =120.9 MPa	209
25		END OF TEST HOLE AT 24.4 m IN BEDROCK Notes: 1. Power Auger Refusal at 14.3 m in Glacial TILL. 2. HQ coring below 14.3 m. 3. No sloughing was observed upon drilling completion. 4. No seepage was observed upon drilling completion. 5.Test hole backfilled with bentonite up to 3 m below ground level and with auger cutting to the ground surafce.							208
26									207
27									206
28									205
29									204
30									

LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 24.38 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 7/14/14
PROJECT ENGINEER: Faris Kahlil

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-04**
LOCATION: UTM: 14U, 5523599 m N, 630952 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.20

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▱ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



-GRAVEL (FILL)

- CLAY (FILL)-trace silt
- black, soft to firm, moist
- intermediate plasticity
- pieces of gravel, boulders, concrete from 0.6 to 1.5 m

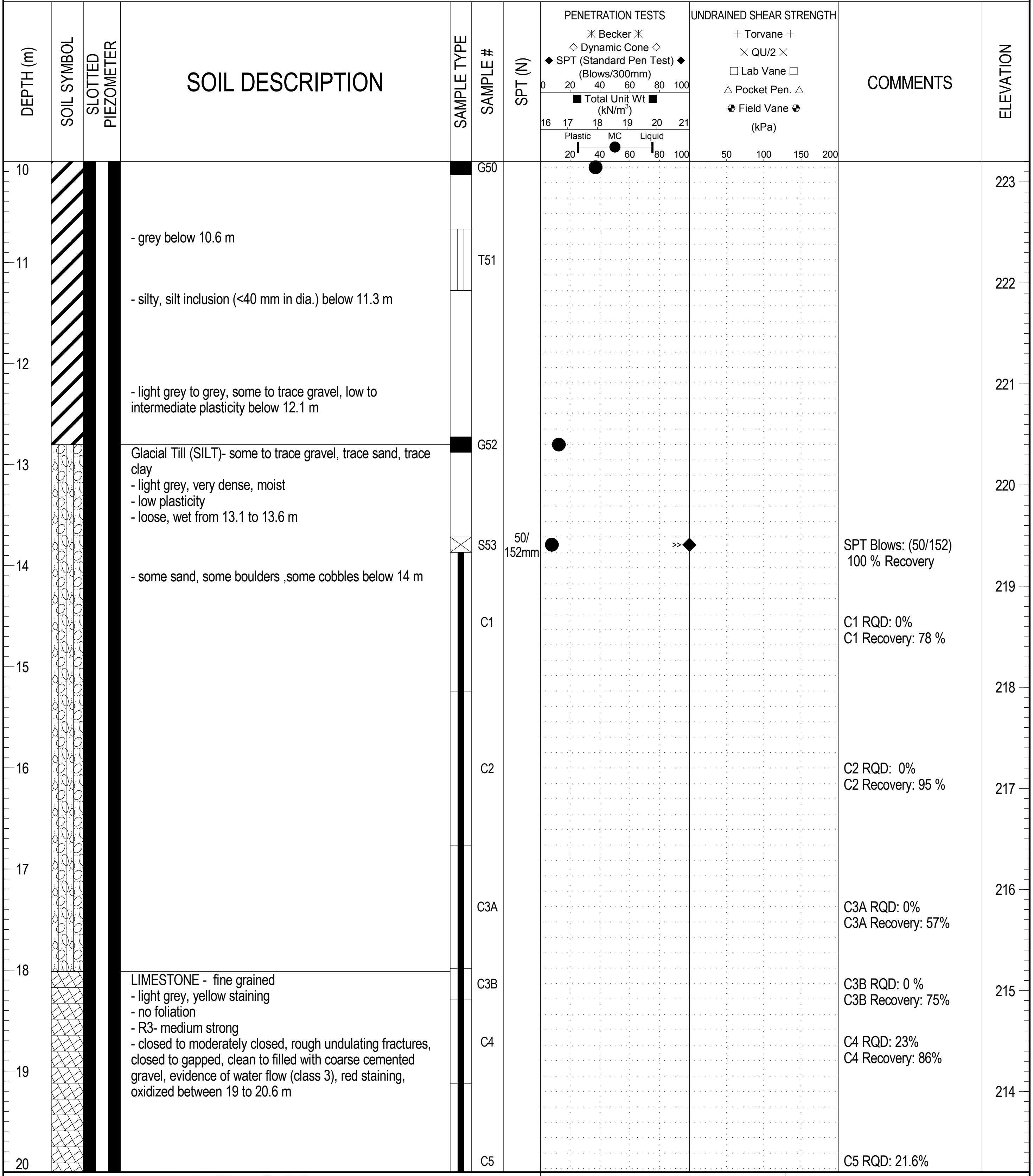
CLAY - trace oxidation
- brown, firm, moist
- high plasticity
- soft to firm between 2.4 to 3 m
- brown mottled light brown, silt inclusion (< 6 mm in dia.) below 3 m
- dark brown, silt inclusion (<10 mm in dia.) below 4.5 m
- grey mottled brown below 6 m
- soft below 7.3 m
- silt pocket at 8.3 m
- trace gravel below 8.8 m
- some silt to silty, light grey to grey below 9.1 m

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.73 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/15/14
PROJECT ENGINEER: Faris Kahlil	

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-04**
LOCATION: UTM: 14U, 5523599 m N, 630952 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.20

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



SOIL DESCRIPTION

- grey below 10.6 m
- silty, silt inclusion (<40 mm in dia.) below 11.3 m
- light grey to grey, some to trace gravel, low to intermediate plasticity below 12.1 m

Glacial Till (SILT)- some to trace gravel, trace sand, trace clay
- light grey, very dense, moist
- low plasticity
- loose, wet from 13.1 to 13.6 m
- some sand, some boulders ,some cobbles below 14 m

LIMESTONE - fine grained
- light grey, yellow staining
- no foliation
- R3- medium strong
- closed to moderately closed, rough undulating fractures, closed to gapped, clean to filled with coarse cemented gravel, evidence of water flow (class 3), red staining, oxidized between 19 to 20.6 m

SAMPLE #	SPT (N)	COMMENTS
G50		
T51		
G52		
S53	50/ 152mm	SPT Blows: (50/152) 100 % Recovery
C1		C1 RQD: 0% C1 Recovery: 78 %
C2		C2 RQD: 0% C2 Recovery: 95 %
C3A		C3A RQD: 0% C3A Recovery: 57%
C3B		C3B RQD: 0 % C3B Recovery: 75%
C4		C4 RQD: 23% C4 Recovery: 86%
C5		C5 RQD: 21.6%

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.73 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/15/14
PROJECT ENGINEER: Faris Kahlil	

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-04**
LOCATION: UTM: 14U, 5523599 m N, 630952 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.20

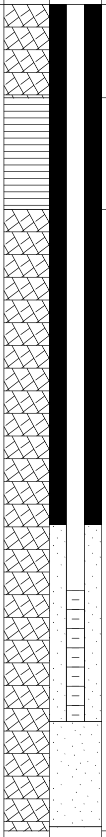
SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND

DEPTH (m) | SOIL SYMBOL | SLOTTED PIEZOMETER | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE # | SPT (N) | PENETRATION TESTS | UNDRAINED SHEAR STRENGTH | COMMENTS | ELEVATION

PENETRATION TESTS: ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ⊕ Field Vane ⊕ (kPa) 50 100 150 200



Depth (m)	Soil Description	Sample #	Comments	Elevation
20			C5 Recovery: 71 %	213
21	SHALE - blue / green - fine grained - no foliation - R1- very weak - close spacing	C6	C6 RQD: 0% C6 Recovery: 56 %	212
22	LIMESTONE - fine grained - creamish white and grey - no foliation - R3- medium strong - moderately closed too widely spaced, planar smooth features, clean, no evidence of water flow (class 2) - gapped fractures(180 degrees to core axis), rough undulating , clean between 21.6 to 22.6 m	C7	C7 RQD: 23% C7 Recovery: 81 %	211
23	- gapped fractures(180 degrees to core axis), rough undulating , clean between 23 to 23.5 m	C8	C8 RQD: 60% C7 Recovery: 100 %	210
24	- gapped fractures(180 degrees to core axis), rough undulating , clean between 24.2 to 25 m			209
25	- R5- very strong	C9	C9 RQD: 26% C7 Recovery: 100 % qu= 114.9 MPa	208

END OF TEST HOLE AT 25.7 m IN BEDROCK
NOTES:
1. Power Auger Refusal at 13.8 m in Glacial TILL.
2. HQ coring below 13.8 m.
3. Seepage observed at 3.0 m upon drilling completion.
4. Installed 25 mm diameter standpipe piezometer (SP14-04) to 23.5 m below ground surface with 0.3 m casagrande tip and flush mount at ground surface.
5. Test hole backfilled with silica sand up to 23.6 m below ground surface, bentonite up to 1 m and plugged with auger cutting to ground surface.
6. Groundwater monitoring:
- Aug. 12, 2014 at Elv. 225.2 m.
- Sep. 03, 2014 at Elv. 225.0 m.
- Sep. 19, 2014 at Elv. 225.6 m.
- Oct. 17, 2014 at Elv. 225.5 m.
- Nov. 06, 2014 at Elv. 225.4 m.
- Nov. 20, 2014 at Elv. 225.4 m.
- Dec. 06, 2014 at Elv. 225.2 m.
- Dec. 18, 2014 at Elv. 225.2 m.

Depth (m): 26 27 28 29 30

Elevation: 207 206 205 204

AECOM	LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.73 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 7/15/14
	PROJECT ENGINEER: Faris Kahlil	Page 3 of 3

PROJECT: Waverley Underpass	CLIENT: City of Winnipeg	TESTHOLE NO: **TH14-28**
LOCATION: UTM: 14U, 5523511 m N, 630871 m E		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 233.80

SAMPLE TYPE: ■ GRAB | ▥ SHELBY TUBE | ☒ SPLIT SPOON | ☰ BULK | ◿ NO RECOVERY | ▌ CORE

BACKFILL TYPE: ■ BENTONITE | ⁘ GRAVEL | ▥ SLOUGH | ⁘ GROUT | ▨ CUTTINGS | ⁘ SAND

DEPTH (m) | SOIL SYMBOL | SLOTTED PIEZOMETER | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE # | SPT (N) | PENETRATION TESTS | UNDRAINED SHEAR STRENGTH | COMMENTS | ELEVATION

SOIL DESCRIPTION

TOPSOIL

CLAY - trace gravel, trace silt
- grey, firm, moist
- high plasticity
- some silt, intermediate plasticity from 0.4 m to 0.6 m
- trace sand, dark grey, soft to firm below 0.6 m

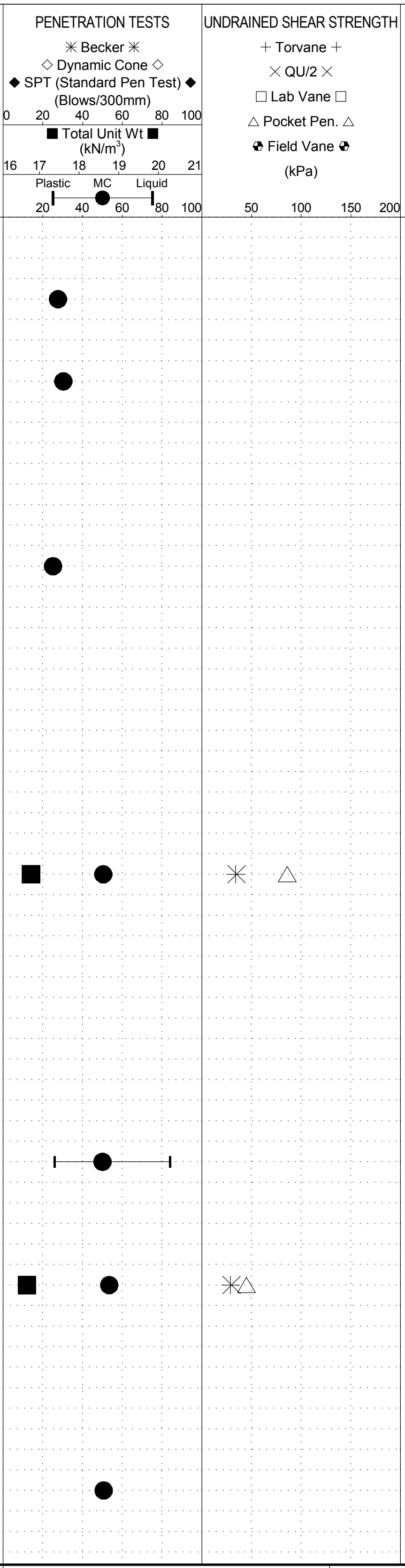
SILT - clayey, sandy
- light brown, soft, moist
- low plasticity

CLAY - silty
- grey,firm to soft, moist
- intemediate plasticity

SILT - sandy, clayey
- light brown, soft, wet to moist
- low plasticity

CLAY - silty
- brown mottled grey, firm, moist,
- high plasticity
- trace silt inclusion (< 6 mm in dia.) from 3 m to 4.6 m
- grey mottled brown, trace oxidation from 6.1 m to 7.6 m
- grey, soft to firm from 7 m to 8.2 m
- trace silt inclusion (< 6 mm in dia.) from 9.1 m to 10.7 m
- soft to firm below 9.14 m

SAMPLE TYPE	SAMPLE #	COMMENTS
■	G188	
■	G189	
■	G190	
▥	T191	
■	G192	(Gravel: 0.0%, Sand: 24.1%, Silt: 55.3%, Clay: 20.6%
■	G193	
▥	T194	
■	G195	
■	G196	Gravel: 0.0%, Sand: 0.0%, Silt: 20.7%, Clay: 79.3%, AASHTO Classification (A-7-6)
▥	T197	
■	G198	



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10

ELEVATION: 233 232 231 230 229 228 227 226 225 224

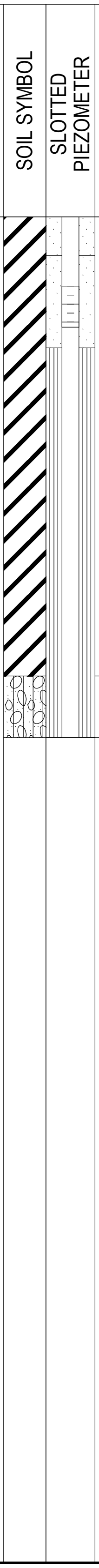
LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 13.87 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

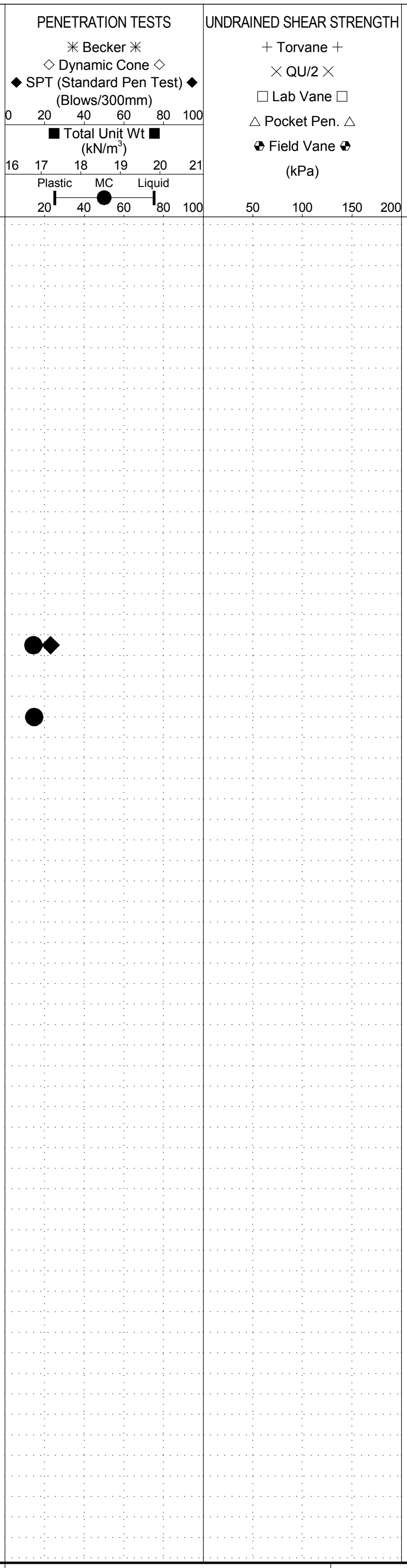
AECOM

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-28**
LOCATION: UTM: 14U, 5523511 m N, 630871 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m): 233.80

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▨ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10		GRAB	G199			
11		SHELBY TUBE	T200			223
12	- some sand, some gravel from 12 m to 13.4 m	SHELBY TUBE	T201			222
13		SPLIT SPOON	S202	23	SPT Blow Count: (10,10,13) 75 %Recovery	221
	Glacial Till (SILT) - some gravel, some sand, some to trace clay - light grey, very dense, moist, - low plasticity	GRAB	G203			220
14						
15						219
16						218
17						217
18						216
19						215
20						214



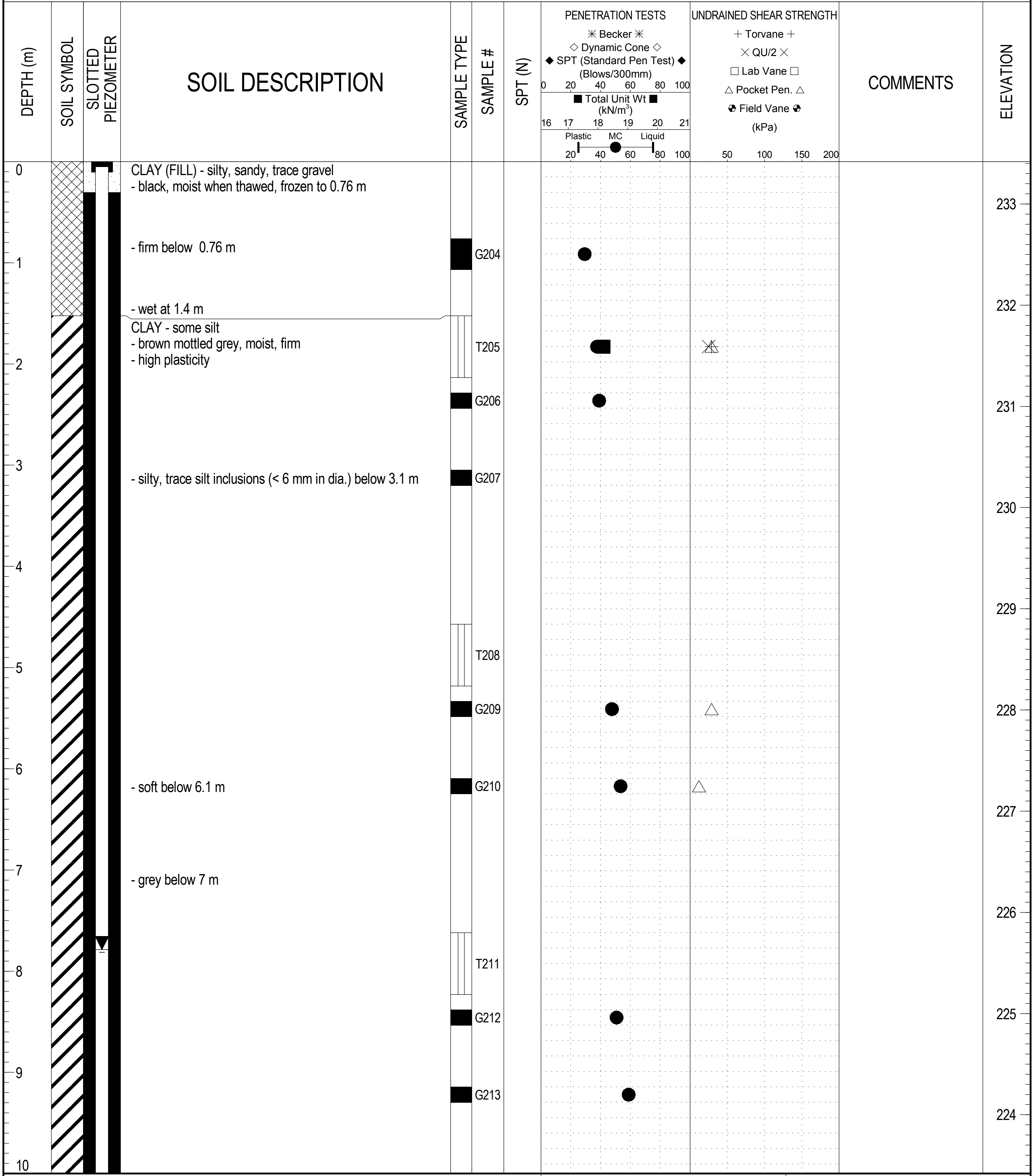


END OF TEST HOLE AT 13.87 m IN Glacial Till (SILT).
NOTES:
1. Power auger refusal at 13.87 m in Glacial Till .
2. Seepage was observed from silt layer below 2.1 m.
3. Sloughing was observed from silt layer below 2.1 m.
4. Installed 25 mm diameter standpipe piezometer (SP14-28) to 11 m below ground surface with 0.3 m casagrande tip and flush mount up to 0.3 m below ground surafce.
5. Test hole backfilled with slough up to 11 m and silica sand up to 0.3 m below ground surface and plugged with top soil to ground surface.
6. Groundwater monitoring:
- Nov. 06, 2014 at Elv. 226.3 m.
- Nov. 20, 2014 at Elv. 226.6 m.
- Dec. 06, 2014 at Elv. 226.6 m.
- Dec. 18, 2014 at Elv. 226.6 m.

LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 13.87 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 10/26/14
PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass
CLIENT: City of Winnipeg
TESTHOLE NO: **TH14-29**
LOCATION: UTM: 14U, 5523602 m N, 630869 m E
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA
ELEVATION (m): 233.42

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▨ NO RECOVERY	▌ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND



CLAY (FILL) - silty, sandy, trace gravel
- black, moist when thawed, frozen to 0.76 m

- firm below 0.76 m

- wet at 1.4 m

CLAY - some silt
- brown mottled grey, moist, firm
- high plasticity

- silty, trace silt inclusions (< 6 mm in dia.) below 3.1 m

- soft below 6.1 m

- grey below 7 m

SAMPLE TYPE	SAMPLE #
■	G204
▥	T205
■	G206
■	G207
▥	T208
■	G209
■	G210
▥	T211
■	G212
■	G213

AECOM

LOGGED BY: Mustafa Alkiki
COMPLETION DEPTH: 15.79 m
REVIEWED BY:
COMPLETION DATE: 12/1/14
PROJECT ENGINEER: Faris Khalil

LOG OF TEST HOLE WAVERLEY UP - PHASE III- TEST HOLE LOGS -.GPJ UMA WINN.GDT 11/22/16

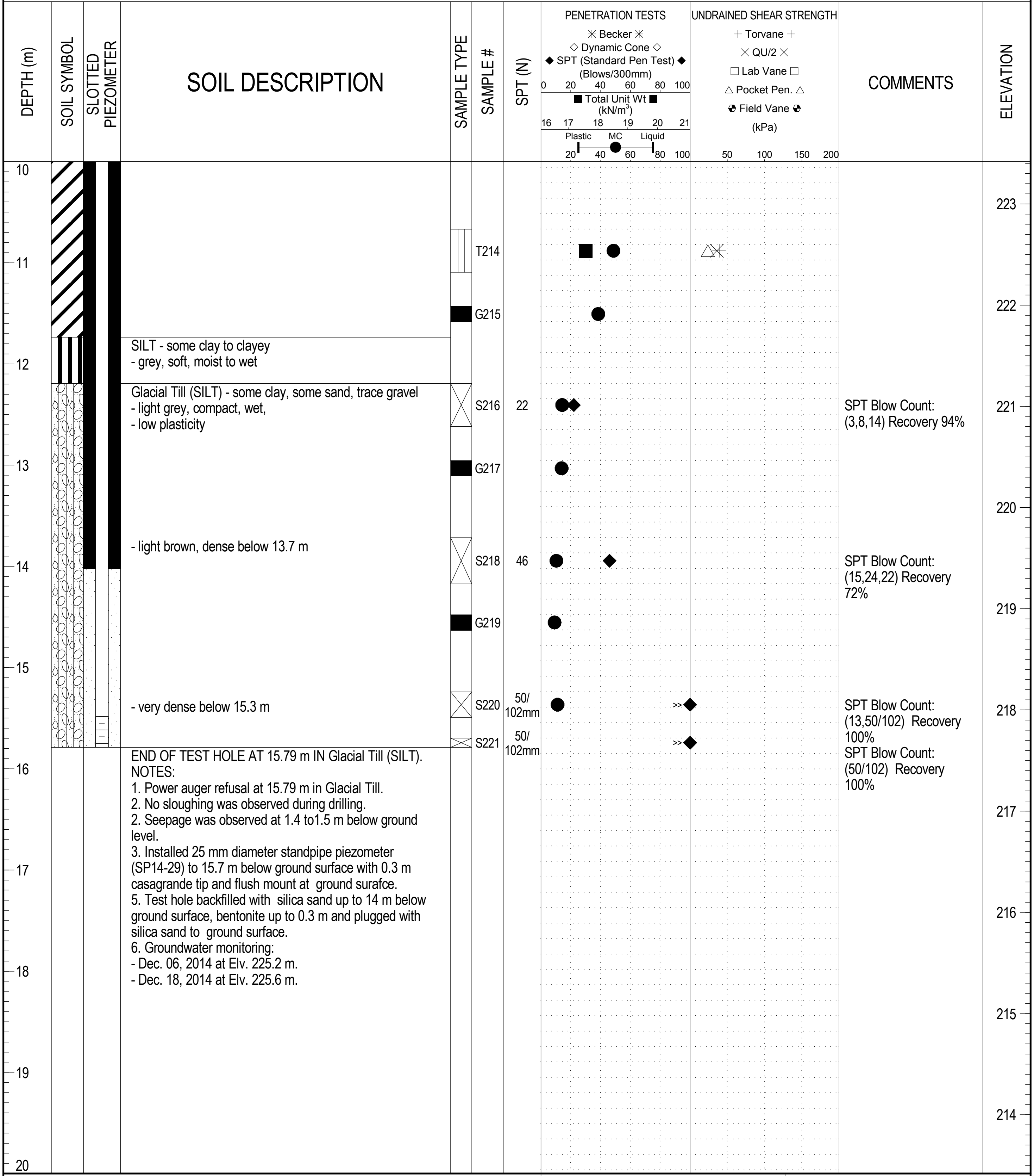
PROJECT: Waverley Underpass | CLIENT: City of Winnipeg | TESTHOLE NO: **TH14-29**

LOCATION: UTM: 14U, 5523602 m N, 630869 m E | PROJECT NO.: 60321148

CONTRACTOR: Maple Leaf Drilling Ltd. | METHOD: 125 mm SSA | ELEVATION (m): 233.42

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



SILT - some clay to clayey
- grey, soft, moist to wet

Glacial Till (SILT) - some clay, some sand, trace gravel
- light grey, compact, wet,
- low plasticity

- light brown, dense below 13.7 m

- very dense below 15.3 m

SAMPLE #	SPT (N)	COMMENTS
T214		
G215		
S216	22	SPT Blow Count: (3,8,14) Recovery 94%
G217		
S218	46	SPT Blow Count: (15,24,22) Recovery 72%
G219		
S220	50/ 102mm	SPT Blow Count: (13,50/102) Recovery 100%
S221	50/ 102mm	SPT Blow Count: (50/102) Recovery 100%

END OF TEST HOLE AT 15.79 m IN Glacial Till (SILT).
NOTES:
1. Power auger refusal at 15.79 m in Glacial Till.
2. No sloughing was observed during drilling.
2. Seepage was observed at 1.4 to1.5 m below ground level.
3. Installed 25 mm diameter standpipe piezometer (SP14-29) to 15.7 m below ground surface with 0.3 m casagrande tip and flush mount at ground surafce.
5. Test hole backfilled with silica sand up to 14 m below ground surface, bentonite up to 0.3 m and plugged with silica sand to ground surface.
6. Groundwater monitoring:
- Dec. 06, 2014 at Elv. 225.2 m.
- Dec. 18, 2014 at Elv. 225.6 m.

LOGGED BY: Mustafa Alkiki | COMPLETION DEPTH: 15.79 m

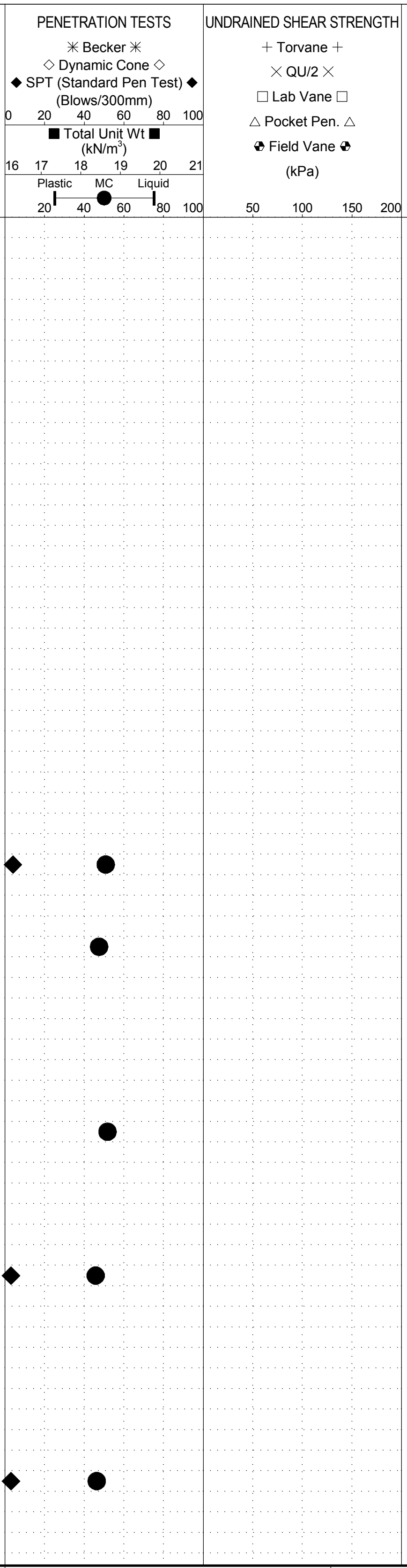
REVIEWED BY: | COMPLETION DATE: 12/1/14

PROJECT ENGINEER: Faris Khalil

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-01**
LOCATION: UTM: 14U,5523569 m N,630934 m E, 7.6 m south of CN south track, 10.0 south east of Waverley Street		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 234.08

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
0	SAND (FILL)- silty, gravelly - light brown, moist - CRUSHED LIMESTONE- - ASPHALT AND CONCRETE-					234
1						233
2						232
	CLAY - trace silt - brown mottled grey, firm to stiff, moist - high plasticity - trace oxidation - trace silt inclusions (<6 m in dia.) - trace sulphate					
3						231
4						230
	- brown to brown mottled grey below 4.7 m	☒	S1	4	SPT Blows: (1,2,2) 100 % Recovery	
5						229
		■	G2			
6	- firm below 6.2 m					228
		■	G3			
7						227
	- grey, soft below 7.7 m	☒	S4	3	SPT Blows: (2,1,2) 100 % Recovery	
8						226
9						225
	- trace till inclusions below 9.2 m	☒	S5	3	SPT Blows: (1,1,2) 100 % Recovery	
10						

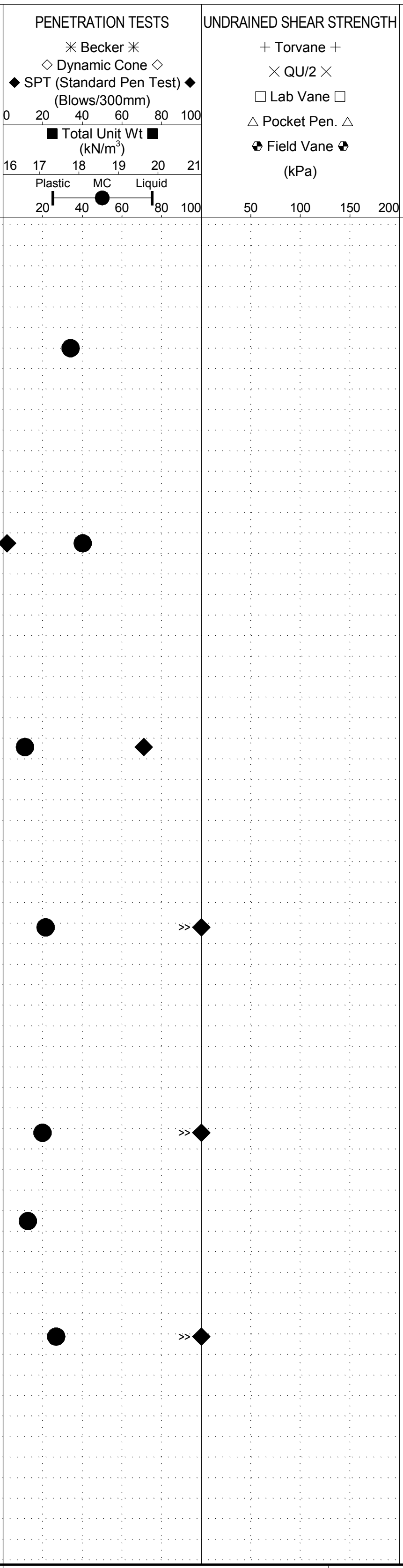


LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.91 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/13/16
PROJECT ENGINEER: Zeyad Shukri	Page 1 of 3

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-01**
LOCATION: UTM: 14U,5523569 m N,630934 m E, 7.6 m south of CN south track, 10.0 south east of Waverley Street		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 234.08

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ⊠ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10						224
11		▥	T6			223
12	- some till inclusions, very soft below 12.3 m	⊠	S7	2	SPT Blows: (1,1,1) 100 % Recovery	222
13						221
14	Glacial Till (SILT)- some sand to sandy, some gravel to gravelly, trace to some clay - light grey, very dense, moist to wet - low plasticity	⊠	S8	71	SPT Blows: (15,21,50) 44 % Recovery	220
15		⊠	S9	50/ 51mm	SPT Blows: (50/51) 33 % Recovery	219
16						218
17		⊠	S10	50/ 51mm	SPT Blows: (50/51) 66 % Recovery	217
		■	G11			
18		⊠	S12	50/ 51mm	SPT Blows: (50/51) 10 % Recovery	216
19	LIMESTONE - very fine to fine grained - creamish white - R3 - medium strong - laminated with fine grained SHALE - sub-horizontal bedding fracture - close to moderately closed spacing, gapped, clean - no evidence of water flow (class 2) - stepped to undulating, smooth to rough fractures	▮	C1		C1 RQD: 64% C1 Recovery: 100 %	215
20	- pinkish grey and white	▮				



LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.91 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/13/16
PROJECT ENGINEER: Zeyad Shukri	

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-01**
LOCATION: UTM: 14U,5523569 m N,630934 m E, 7.6 m south of CN south track, 10.0 south east of Waverley Street		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 234.08

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◺ NO RECOVERY ▌ CORE

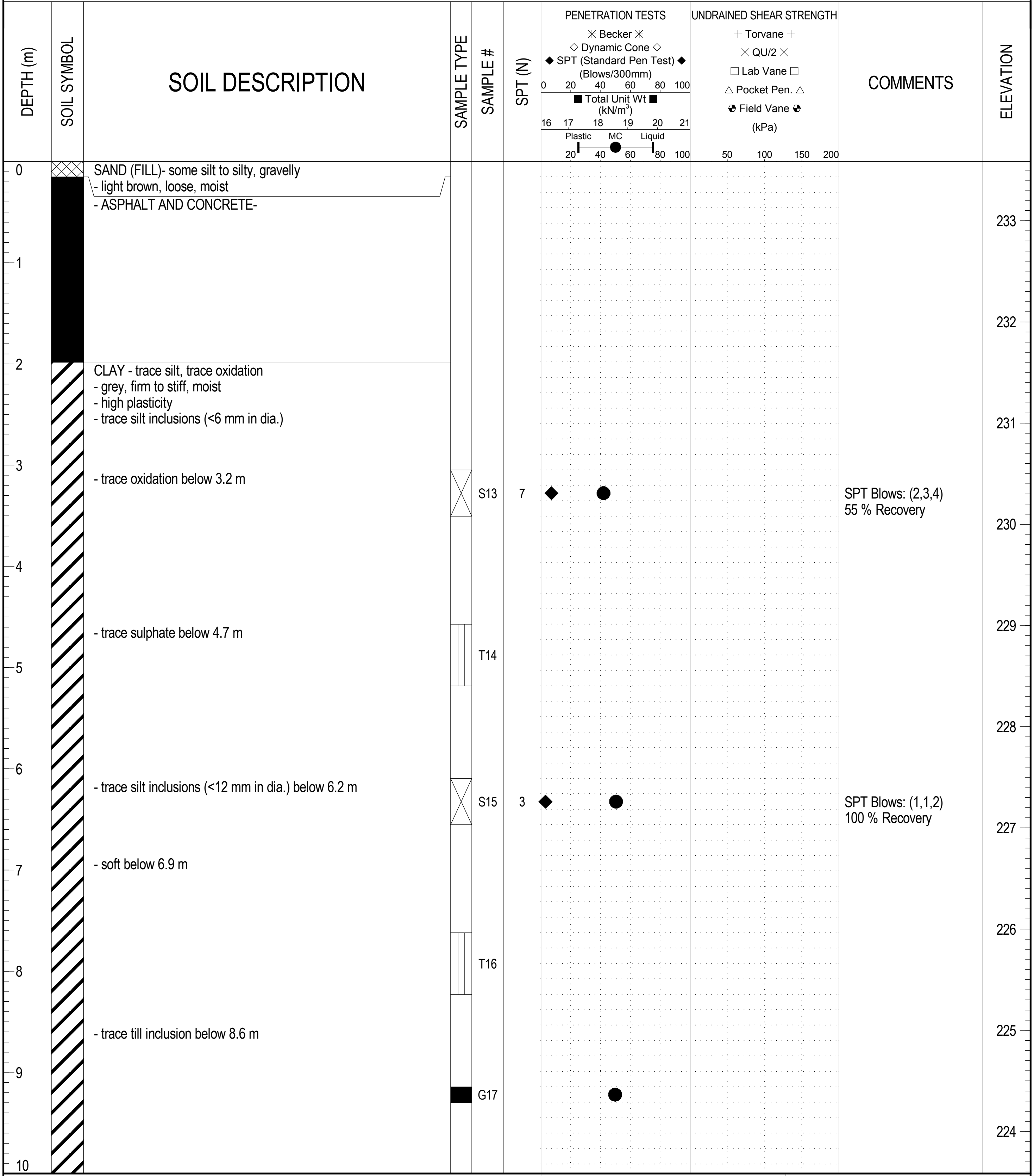
DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
20	▨	- close to widely spaced, undulating, rough, close to gapped fractures	▌						214
	▨		▌	C2				C2 RQD: 66% C2 Recovery: 100%	
21	▨		▌						213
	▨	- creamish grey - laminated with fine grained dark grey SHALE - planar fractures	▌						
22	▨		▌	C3				C3 RQD: 23% C3 Recovery: 86%	212
23	▨	- very close to closed spacing, close to gapped, clean - stepped to undulating, smooth to rough fractures	▌						211
	▨	- R2 - weak - gapped to open, evidence of water flow (class 3)	▌	C4				C4 RQD: 15% C4 Recovery: 100%	
24	▨		▌						210
	▨	- creamish white - R5 - very strong - sub-horizontal bedding fracture - close to moderately closed spacing, gapped to open , clean - no evidence of water flow (class 2) - undulating to planar	▌						
25	▨		▌	C5				C5 RQD: 41% C5 Recovery: 100%, UCS=107.7 MPa	209
	▨	- prominent joint set between 25.4 to 25.6 m (20 to 45 degrees at core axis)	▌						
26		END OF TEST HOLE AT 25.9 m IN BEDROCK Notes: 1. Power Auger Refusal at 18.3 m in Glacial TILL. 2. HQ coring below 18.3 m. 3. No sloughing was observed upon drilling completion. 4. No seepage was observed upon drilling completion. 5.Test hole backfilled with bentonite up to 1.0 m and plugged with auger cutting to ground surface.							208
27									207
28									206
29									205
30									

AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 25.91 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/13/16
PROJECT ENGINEER: Zeyad Shukri	Page 3 of 3

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-02**
LOCATION: UTM: 14U,5523572 m N,630943 m E, 6.5 m south of CN south track, 19.5 south east of Waverley Street		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.58

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE



SAND (FILL)- some silt to silty, gravelly
- light brown, loose, moist

- ASPHALT AND CONCRETE-

CLAY - trace silt, trace oxidation
- grey, firm to stiff, moist
- high plasticity
- trace silt inclusions (<6 mm in dia.)

- trace oxidation below 3.2 m

- trace sulphate below 4.7 m

- trace silt inclusions (<12 mm in dia.) below 6.2 m

- soft below 6.9 m

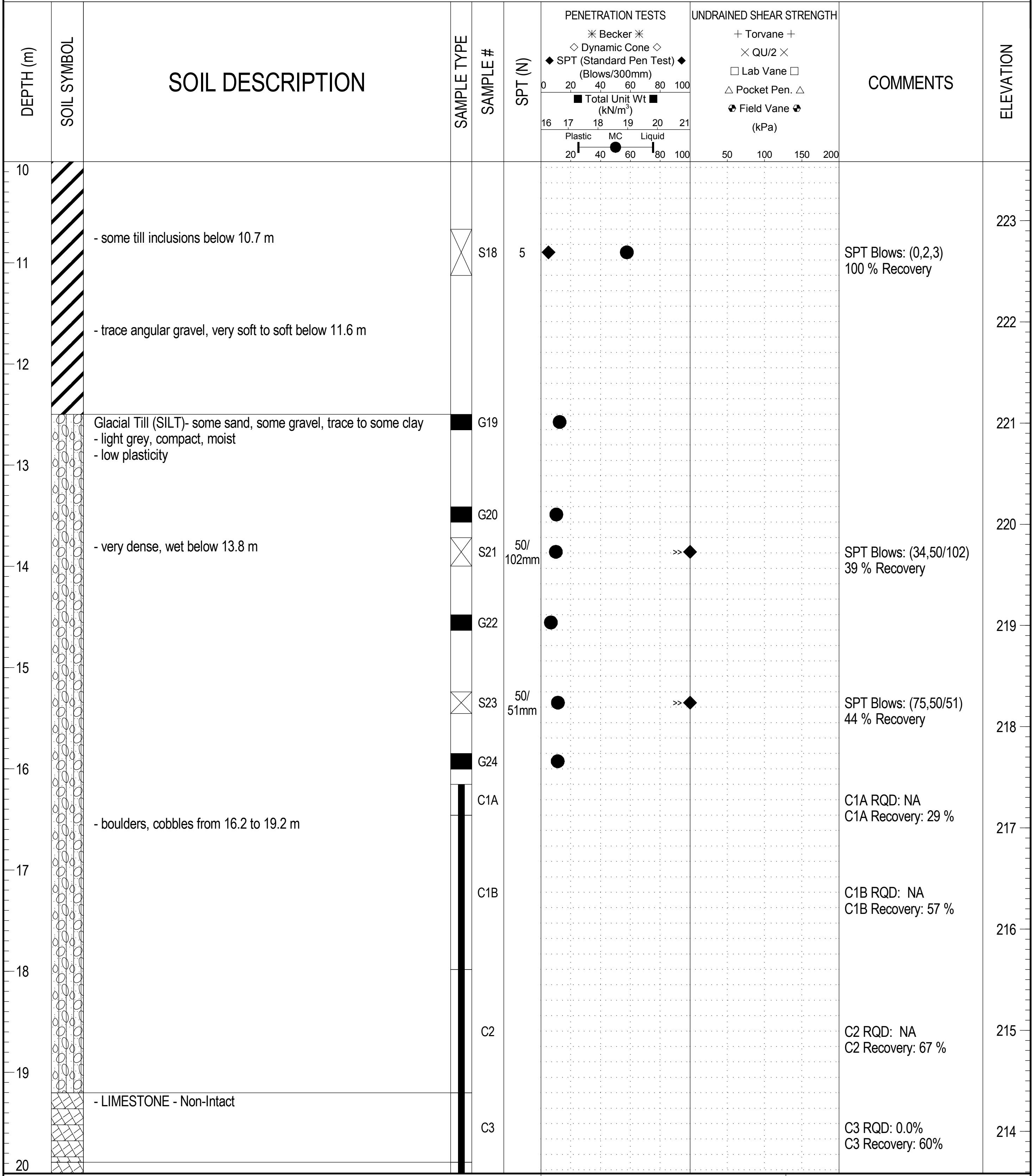
- trace till inclusion below 8.6 m

SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
☒	S13	7	SPT Blows: (2,3,4) 55 % Recovery
▥	T14		
☒	S15	3	SPT Blows: (1,1,2) 100 % Recovery
▥	T16		
■	G17		

AECOM

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 22.48 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/14/16
PROJECT ENGINEER: Zeyad Shukri	Page 1 of 3

PROJECT: Waverley Underpass - Detailed Design
CLIENT: Dillon Consulting Ltd.
TESTHOLE NO: **TH16-02**
LOCATION: UTM: 14U,5523572 m N,630943 m E, 6.5 m south of CN south track, 19.5 south east of Waverley Street
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.58
SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌CORE



- some till inclusions below 10.7 m
- trace angular gravel, very soft to soft below 11.6 m

Glacial Till (SILT)- some sand, some gravel, trace to some clay
- light grey, compact, moist
- low plasticity
- very dense, wet below 13.8 m
- boulders, cobbles from 16.2 to 19.2 m

- LIMESTONE - Non-Intact

SAMPLE #	SPT (N)	COMMENTS
S18	5	SPT Blows: (0,2,3) 100 % Recovery
G19		
G20		
S21	50/ 102mm	SPT Blows: (34,50/102) 39 % Recovery
G22		
S23	50/ 51mm	SPT Blows: (75,50/51) 44 % Recovery
G24		
C1A		C1A RQD: NA C1A Recovery: 29 %
C1B		C1B RQD: NA C1B Recovery: 57 %
C2		C2 RQD: NA C2 Recovery: 67 %
C3		C3 RQD: 0.0% C3 Recovery: 60%

LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Zeyad Shukri
COMPLETION DEPTH: 22.48 m
COMPLETION DATE: 4/14/16

PROJECT: Waverley Underpass - Detailed Design
CLIENT: Dillon Consulting Ltd.
TESTHOLE NO: **TH16-02**
LOCATION: UTM: 14U,5523572 m N,630943 m E, 6.5 m south of CN south track, 19.5 south east of Waverley Street
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.58

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × ☐ Lab Vane ☐ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
20	▨	LIMESTONE - very fine to fine grained - pinkish yellow and grey - R3 - medium strong - sub-horizontal bedding fracture, close to widely spaced, closed and clean, no evidence of water flow (class 2), planar, smooth to rough fractures	▌	C4				C4 RQD: 70% C4 Recovery: 80%	213
21	▨	- non- intact zone from 21.1 to 21.3 m, R1 to R2 - very weak to weak, greyish white - non- intact hard CLAY SHALE and fractured LIMESTONE between 21.3 to 21.6 m	▌	C5				C5 RQD: 0.0% C5 Recovery: 100 %	212
22	▨	- creamish white and grey, R3 - medium strong - laminated with fine grained SHALE and hard dark grey CLAY - close to moderately closed spaced, closed to gapped and clean to infilled with hard clay (class 2) - rough and undulating fractures	▌	C6				C6 RQD: 54% C6 Recovery: 95 %	211
23		END OF TEST HOLE AT 22.5 m IN BEDROCK Notes: 1. Power Auger Refusal at 16.2 m in Glacial TILL. 2. HQ coring below 16.2 m. 3. No sloughing was observed upon drilling completion. 4. No seepage was observed upon drilling completion. 5.Test hole backfilled with bentonite up to 1.0 m below ground level and with auger cutting to the ground surafce.							210
24									209
25									208
26									207
27									206
28									205
29									204
30									

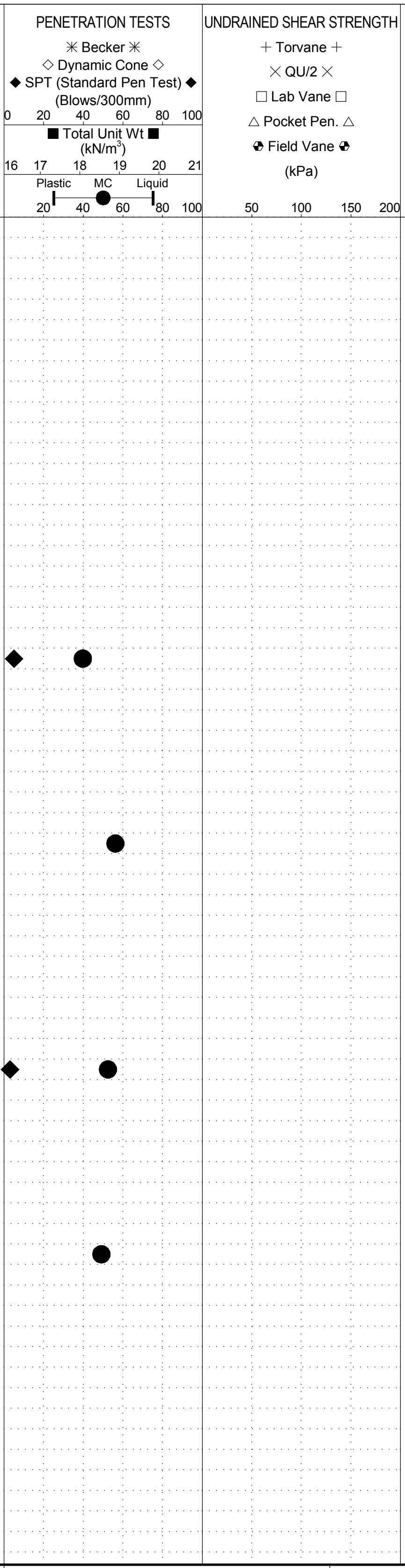
AECOM

LOGGED BY: Saba Ibrahim
COMPLETION DEPTH: 22.48 m
REVIEWED BY: Zeyad Shukri
COMPLETION DATE: 4/14/16
PROJECT ENGINEER: Zeyad Shukri

PROJECT: Waverley Underpass - Detailed Design
CLIENT: Dillon Consulting Ltd.
TESTHOLE NO: **TH16-03**
LOCATION: UTM: 14U,5523582 m N,630892 m E, 5.3 m north of CN north track, 7.9 north west of Waverley Street
PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.
METHOD: 125 mm SSA/ HQ Coring
ELEVATION (m): 233.88

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◸ NO RECOVERY ▌ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
0	SAND (FILL)- some silt, some gravel to gravelly - light brown, loose, moist					
						233
1	CLAY (FILL) - silty to some silt, trace to some sand - dark brown, firm, moist - high to intermediate plasticity					
	CLAY - trace silt - grey, firm to stiff, moist - high plasticity					
						232
2						
						231
3						
	- brown mottled grey, trace silt inclusions (< 12 mm in dia.) below 3.1 m	☒	S25	5	SPT Blows: (1,2,3) 100 % Recovery	
						230
4						
		■	G26			
						229
5						
						228
6						
	- trace oxidation below 6.1 m					
		☒	S27	3	SPT Blows: (1,1,2) 100 % Recovery	
						227
7						
	- grey below 7.4 m					
	- soft below 7.7 m	■	G28			
						226
8						
						225
9						
		▥	T29			
10	- some silt below 7.8 m					224

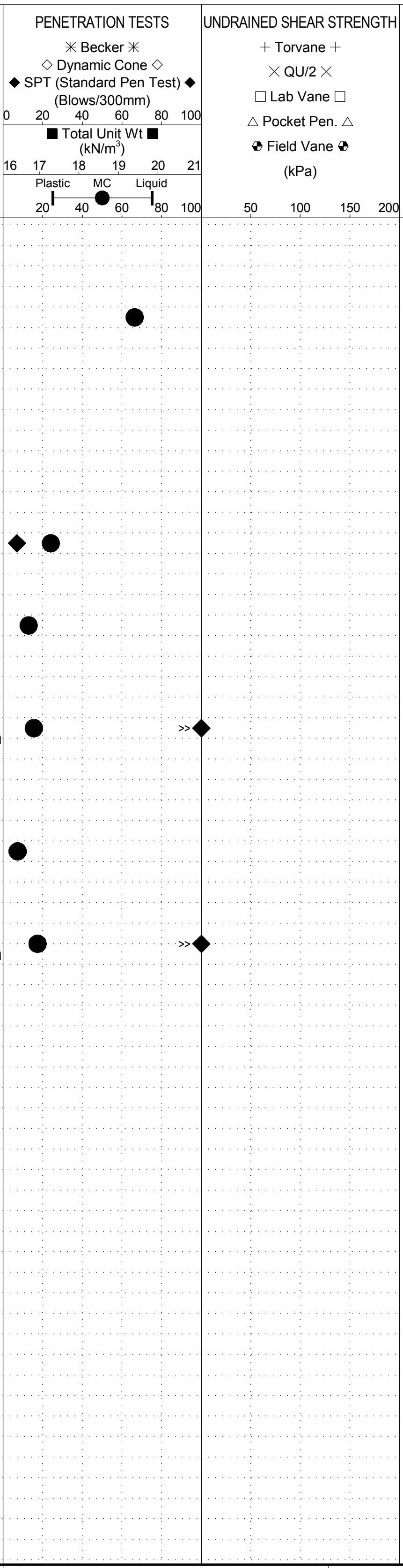


AECOM

LOGGED BY: Saba Ibrahim
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Zeyad Shukri
COMPLETION DEPTH: 27.48 m
COMPLETION DATE: 4/19/16

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-03**
LOCATION: UTM: 14U,5523582 m N,630892 m E, 5.3 m north of CN north track, 7.9 north west of Waverley Street		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA/ HQ Coring	ELEVATION (m): 233.88

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE



DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10						
		■	G30			223
11	- trace till inclusions below 11 m					
	- very soft to soft below 11.6 m					222
12	- some silt inclusions below 12 m					
	SILT- clayey - light brown, very soft, moist - low plasticity	☒	S31	7	SPT Blows: (2,2,5) 100 % Recovery	
13	CLAY - trace silt, trace to some gravel - grey, soft, moist - high plasticity	■	G32			221
	Glacial Till (SILT)- some gravel, some sand, trace clay - light grey, very dense, moist - low plasticity					
		☒	S33	100/ 152mm	SPT Blows: (100/152) 33 % Recovery	220
14						
		■	G34			219
15						
		☒	S35	50/ 102mm	SPT Blows: (45,50/102) 61 % Recovery	218
16	- boulders, cobbles with till matrix below 16 m					
		▮	C1		C1 RQD: NA C1 Recovery: 44 %	217
17						
		▮	C2		C2 RQD: NA C2 Recovery: 94 %	216
18	- reddish brown, gravelly below 18.1 m					
		▮	C3		C3 RQD: NA C3 Recovery: 79%	215
19						
		▮	C4		C4 RQD: NA C4 Recovery: 44%	
20						214

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 27.48 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/19/16
PROJECT ENGINEER: Zeyad Shukri	

PROJECT: Waverley Underpass - Detailed Design | CLIENT: Dillon Consulting Ltd. | TESTHOLE NO: **TH16-03**

LOCATION: UTM: 14U,5523582 m N,630892 m E, 5.3 m north of CN north track, 7.9 north west of Waverley Street | PROJECT NO.: 60321148

CONTRACTOR: Maple Leaf Drilling Ltd. | METHOD: 125 mm SSA/ HQ Coring | ELEVATION (m): 233.88

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

PENETRATION TESTS: ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane + × QU/2 × □ Lab Vane □ △ Pocket Pen. △ ⊕ Field Vane ⊕ (kPa) 50 100 150 200

DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
20		C5		C5 RQD: NA C5 Recovery: 20%	
	- C6 RQD: NA C6 Recovery: 100%	C6			213
21	LIMESTONE - very fine to fine grained - greyish white to creamish grey and white - R2 to R3 - weak to medium strong - laminated with fine grained SHALE - sub-horizontal bedding fractures,extermelly closed to closed spaced, close to open, clean, evidence of water flow (class 3)	C7A C7B C8		C7A RQD: NA C7A Recovery: 100 % C7B RQD: 52% C7B Recovery: 100 % C8 RQD: 67% C8 Recovery: 89 %	212
22	- CLAY SHALE infilling between 22.25 to 22.5 m	C9		C9 RQD: 30% C9 Recovery: 100 %	211
23	- light grey and white, R5 - very strong - non intact to moderately closed spaced, close to open, clean - no evidence of water flow (class 3) - undulating to planar, smooth to rough fractures	C10		C10 RQD: 44% C10 Recovery: 93 %, UCS=145.1 MPa	210
24	- non intact to widely closed spaced below 24.5 m				209
25	- prominent joint set between 24.9 to 25.2 m (10 to 25 degrees at core axis), open and clean (class 3)	C11		C11 RQD: 66% C11 Recovery: 91 %	208
26	- closed to moderately spaced, close to open, clean below 26 m	C12		C12 RQD: 73% C12 Recovery: 100 %	207
27	END OF TEST HOLE AT 27.5 m IN BEDROCK NOTES: 1. Power Auger Refusal at 15.8 m in Glacial TILL. 2. HQ coring below 15.8 m. 3. Seepage observed below 9.0 m upon drilling completion. 4. No sloughing was observed upon drilling completion. 5.Test hole backfilled with bentonite up to 1 m below ground surface and plugged with auger cutting to ground surface.				206
28					205
29					
30					204

AECOM

LOGGED BY: Saba Ibrahim | COMPLETION DEPTH: 27.48 m

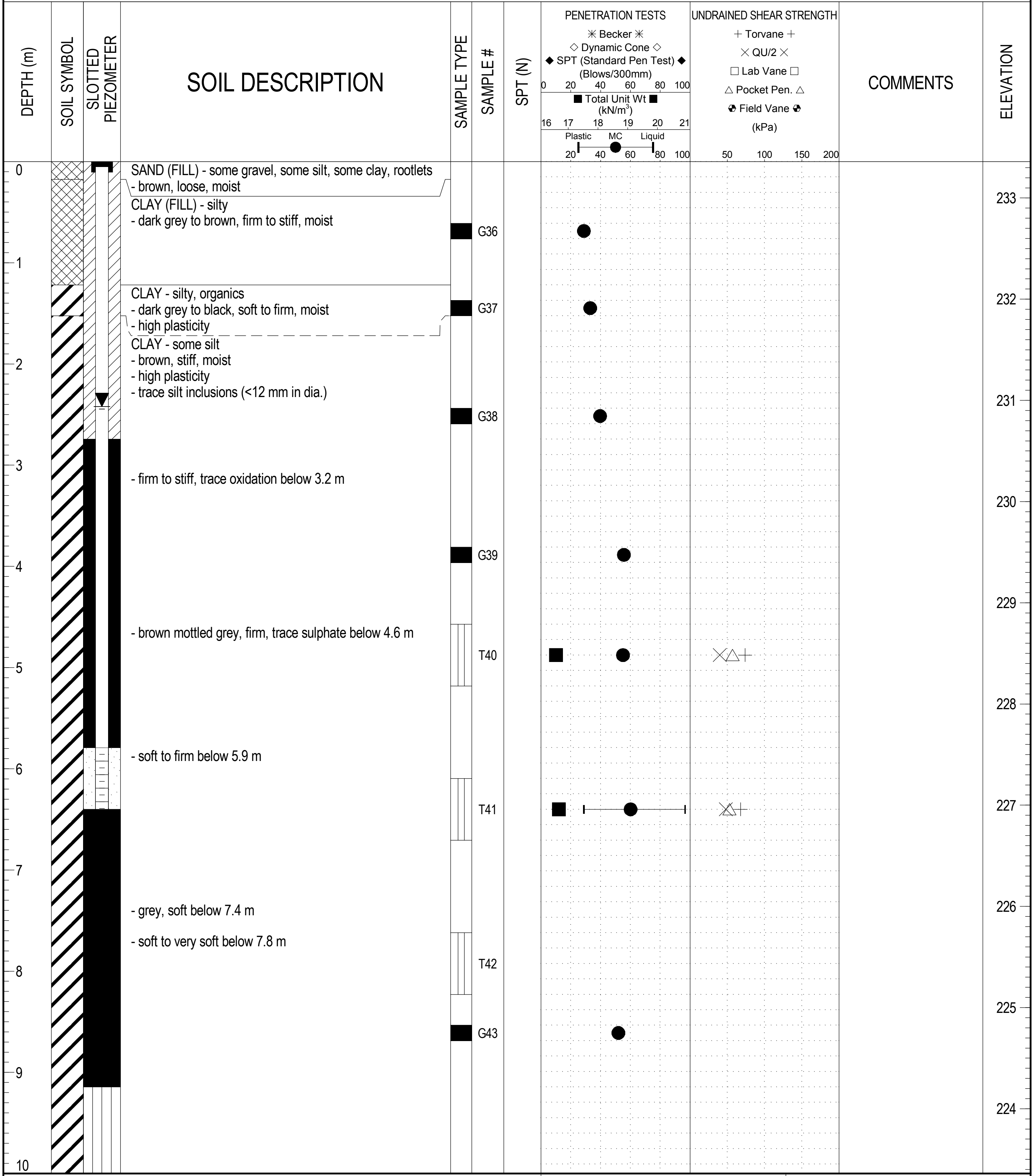
REVIEWED BY: Zeyad Shukri | COMPLETION DATE: 4/19/16

PROJECT ENGINEER: Zeyad Shukri

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-04**
LOCATION: UTM: 14U,5523519 m N,630502 m E, vicinity of LDS/CN crossing, north of CN north track		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 233.36

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



SAND (FILL) - some gravel, some silt, some clay, rootlets
- brown, loose, moist

CLAY (FILL) - silty
- dark grey to brown, firm to stiff, moist

CLAY - silty, organics
- dark grey to black, soft to firm, moist
- high plasticity

CLAY - some silt
- brown, stiff, moist
- high plasticity
- trace silt inclusions (<12 mm in dia.)

- firm to stiff, trace oxidation below 3.2 m

- brown mottled grey, firm, trace sulphate below 4.6 m

- soft to firm below 5.9 m

- grey, soft below 7.4 m

- soft to very soft below 7.8 m

LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 10.67 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/19/16
PROJECT ENGINEER: Zeyad Shukri	

PROJECT: Waverley Underpass - Detailed Design	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH16-04**
LOCATION: UTM: 14U,5523519 m N,630502 m E, vicinity of LDS/CN crossing, north of CN north track		PROJECT NO.: 60321148
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: 125 mm SSA	ELEVATION (m): 233.36

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▌ CORE

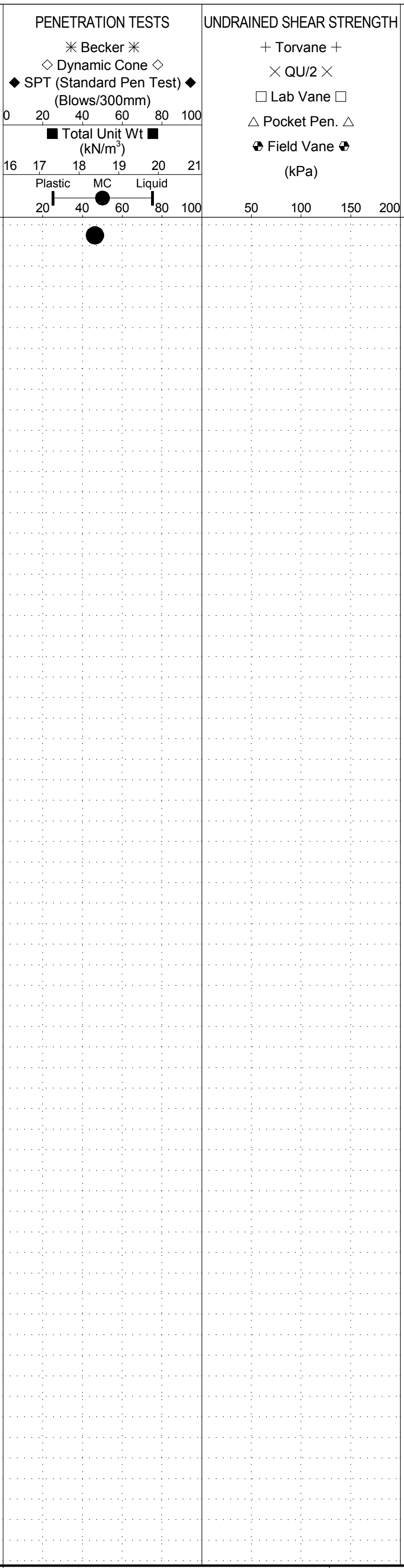
BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND

DEPTH (m)	SOIL SYMBOL	SLOTTED PIEZOMETER	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10	▨	▥	- trace till inclusions below 10.2 m	■	G44			223
			END OF TEST HOLE AT 10.67 m in CLAY					
11								222
12								221
13								220
14								219
15								218
16								217
17								216
18								215
19								214
20								

END OF TEST HOLE AT 10.67 m in CLAY

NOTES:

1. Groundwater was observed at 1.5 m upon drilling completion.
2. Sloughing was observed at 3.0 m below ground upon drilling completion.
3. Installed 25 mm diameter standpipe piezometer (SP16-04) to 6.5 m below ground surface with 0.3 m casagrande tip and flush mount at ground surafce.
4. Test hole backfilled with slough up to 9.1 m, bentonite up to 6.5, silica sand up to 5.8 m below ground surface, plugged with bentonite to 2.75 m below ground surface and finished with auger cutting to ground surface.
5. Groundwater monitoring:
- April 29, 2016 at Elv. 230.20 m
- May 13, 2016 at Elv. 230.60 m
- June 18, 2016 at Elv. 231.02 m
- June 24, 2016 at Elv. 231.08 m
- July 18, 2016 at Elv. 231.08 m
- August 30, 2016 at Elv. 230.98 m



LOGGED BY: Saba Ibrahim	COMPLETION DEPTH: 10.67 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 4/19/16
PROJECT ENGINEER: Zeyad Shukri	Page 2 of 2

AECOM

Appendix C

Laboratory Test Results

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Memorandum

To	Saba Ibrahim	Page	1
CC			
Subject	Waverly Underpass		
From	Jared Baldwin		
Date	September 22, 2014	Project Number	60321148

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Twenty-six (26) Moisture Content tests.
- Two (2) Atterberg Limits (3 points) tests.
- Three (3) Torvane, Pocket Penetrometer, Moisture Content, Bulk Density and Visual Description with Unconfined Compressive Strength, on Shelby tube samples.
- Four (4) Waxed Shelby tube Samples.

If you have any questions, please contact the undersigned.

Sincerely,

Jared Baldwin, M.Sc., P.Eng.
Geotechnical Engineer

Att.

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	Varies	Sample Date:	Varies
Sample Depth:	Varies	Lab Technician:	CMahe
Sample Number:	Varies	Date Tested:	August 19, 2014

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	**Moisture Content (%)**
TH14-01	G1	0.61 - 0.76 m	**38.8**
-	G3	2.13 - 2.29 m	**40.4**
-	G5	4.27 - 4.42 m	**54.7**
-	G10	8.23 - 8.38 m	**49.8**
-	G13	11.28 - 11.43 m	**44.2**
-	S15	12.19 - 12.34 m	**6.5**
TH14-02	G16	2.59 - 2.74 m	**39.8**
-	G19	5.64 - 5.79 m	**49.5**
-	T21	7.62 - 8.23 m	**48.5**
-	G23	10.06 - 10.21 m	**57.3**
-	G25	11.58 - 11.73 m	**39.7**
-	S28	13.41 - 13.87 m	**9.1**
-	S29	14.33 - 14.78 m	**13.1**
-	S30	15.24 - 15.70 m	**12.6**
TH14-03	G31	2.44 - 2.59 m	**35.6**
-	G33	5.33 - 5.49 m	**43.5**
-	G35	7.32 - 7.47 m	**46.8**
-	G38	10.97 - 11.13 m	**44.9**
-	G41	13.41 - 13.56 m	**9.7**
-	S42	13.72 - 14.17 m	**9.9**
TH14-04	G44	3.66 - 3.81 m	**49.0**
-	G46	6.40 - 6.55 m	**49.3**
-	G50	10.06 - 10.21 m	**37.0**
-	G52	12.80 - 12.95 m	**12.3**
-	S53	13.72 - 14.17 m	**7.6**
-	G47	7.32 - 7.47 m	**45.4**

Location	Sample	Depth (m)	**Moisture Content (%)**

AECOM

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

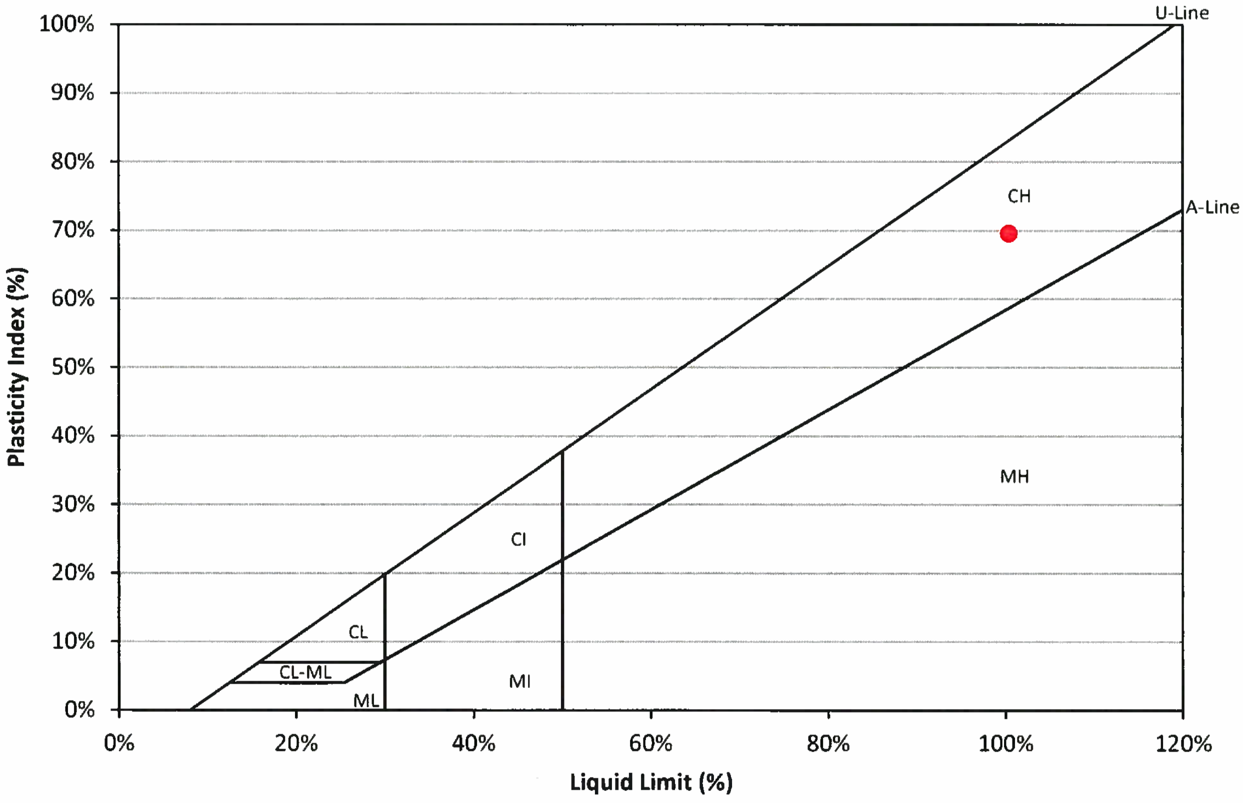
Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	14-01	Sample Date:	July 1, 2014
Sample Depth:	4.27	Lab Technician:	RDagg
Sample Number:	G5	Date Tested:	August 22, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	34	25	22
Wet Sample (g)	17.1	19.4	17.8
Dry Sample (g)	8.7	9.7	8.8
Water Content (%)	95.9%	100.0%	102.0%

Plastic Limit		
Trial	1	2
Wet Sample (g)	4.6	4.0
Dry Sample (g)	3.5	3.0
Water Content (%)	30.7%	30.7%



Liquid Limit (%): 100.3%	Plastic Limit (%): 30.7%	Plasticity Index (%): 69.6%

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

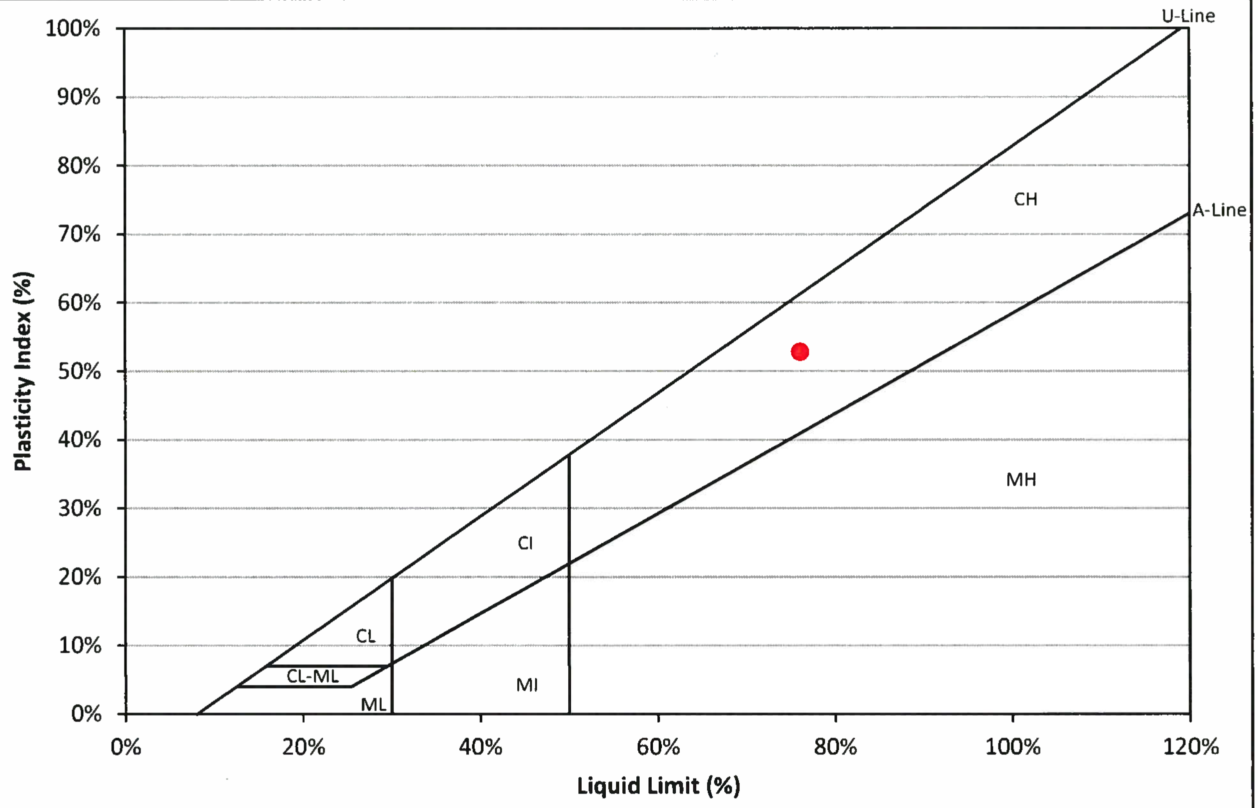
Project Name:	Waverly Underpass	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	14-02	Sample Date:	July 1, 2014
Sample Depth:	7.62	Lab Technician:	ML
Sample Number:	T21	Date Tested:	September 2, 2014

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	19	24	31
Wet Sample (g)	14.4	12.4	13.2
Dry Sample (g)	8.1	7.0	7.6
Water Content (%)	77.8%	76.1%	74.5%

Plastic Limit		
Trial	1	2
Wet Sample (g)	9.0	7.9
Dry Sample (g)	7.3	6.4
Water Content (%)	23.2%	23.1%



Liquid Limit (%): 76.0%	Plastic Limit (%): 23.2%	Plasticity Index (%): 52.9%

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-01**
SAMPLE NO.:	**T4**
SAMPLE DEPTH:	3.05 - 3.66 m
DATE TESTED:	2-Sep-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.55
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**53.9**
Undrained Shear Strength (ksf)	1.13
POCKET PENETROMETER	
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**69.5**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**34.8**
Undrained Shear Strength (ksf)	**0.726**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	442.0
Wt. Sample dry + tare (g)	303.6
Wt. Tare (g)	8.3
Moisture Content %	**46.9**
BULK DENSITY	
Sample Wt. (g)	1065.8
Diameter 1 (cm)	7.23
Diameter 2 (cm)	7.24
Diameter 3 (cm)	7.24
Avg. Diameter (cm)	**7.24**
Length 1 (cm)	15.34
Length 2 (cm)	15.35
Length 3 (cm)	15.36
Avg. Length (cm)	**15.35**
Volume (cm^3)	631.4
Moisture content (%)	46.9
Bulk Density (g/cm^3)	1.688
Bulk Density (kN/m^3)	**16.6**
Bulk Density (pcf)	**105.4**
Dry Density (kN/m^3)	**11.27**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

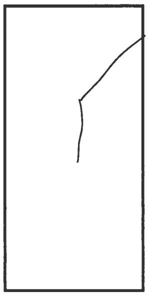
AECOM

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-01
SAMPLE NO.:	T4
SAMPLE DEPTH:	3.05 - 3.66 m
SAMPLE DATE:	February, 2014
TEST DATE:	2-Sep-14

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, brown, moist, firm, high plasticity,	
MOISTURE CONTENT:	46.9

SAMPLE DIAM.(Do):	72.37	(mm)	**INITIAL AREA, Ao:**	4113.1	(mm^2)
SAMPLE LENGTH, (Lo):	153.50	(mm)	**PISTON RATE:**	0.051	(inches / minute)
L / D RATIO:	2.12	(2 < L/D < 2.5)	**AXIAL STRAIN RATE, R:**	0.84	(0.5<R<2 % / minute)



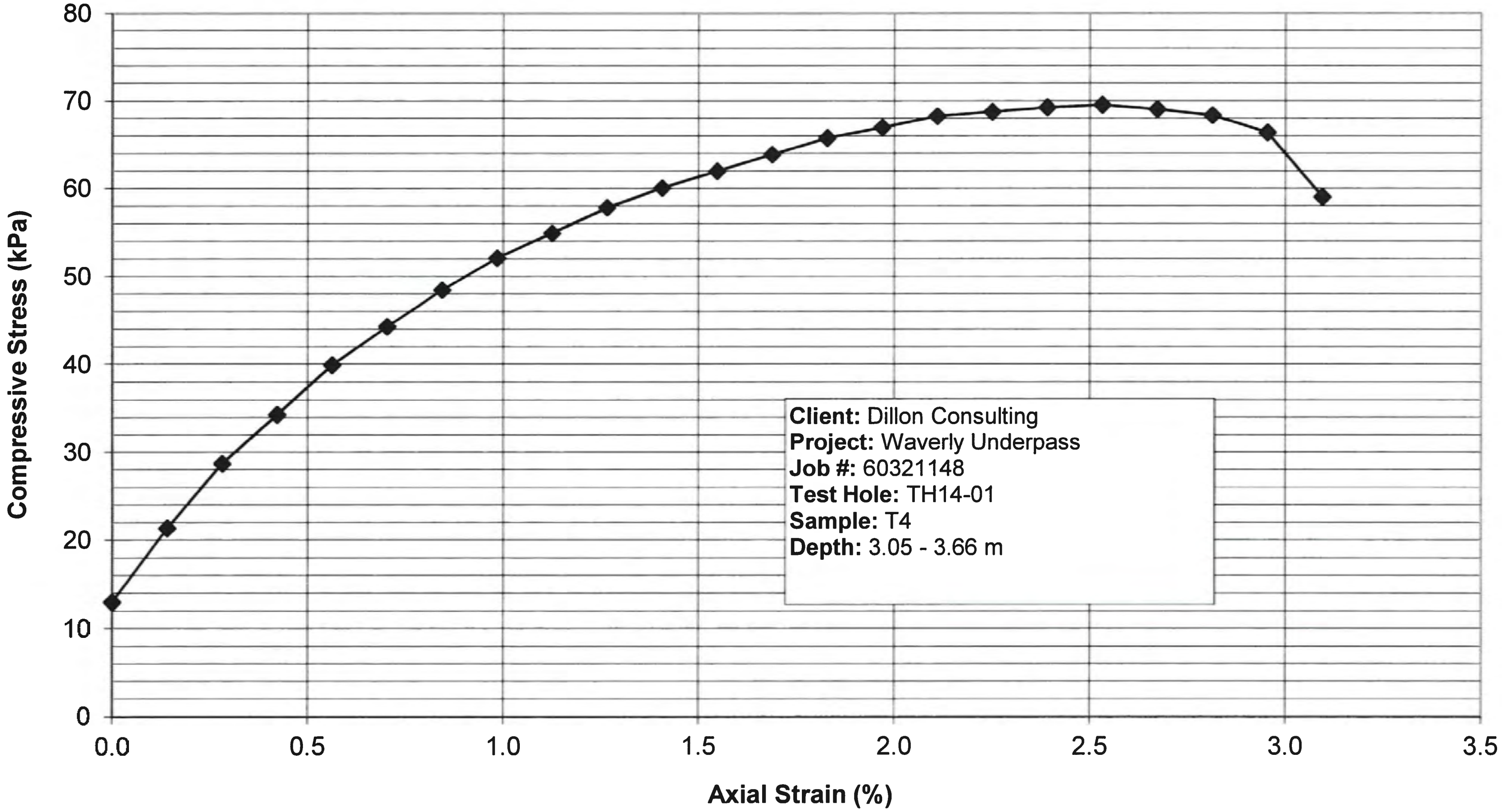
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_c		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0013	0.00	6.38	11.99	1.88	0.271	13.0
0.02	0.0021	0.14	6.38	19.77	3.10	0.446	21.4
0.03	0.0028	0.28	6.39	26.61	4.16	0.599	28.7
0.03	0.0034	0.42	6.40	31.86	4.98	0.717	34.3
0.04	0.0040	0.56	6.41	37.11	5.79	0.833	39.9
0.05	0.0044	0.70	6.42	41.23	6.42	0.925	44.3
0.06	0.0048	0.84	6.43	45.16	7.02	1.012	48.4
0.07	0.0052	0.98	6.44	48.63	7.55	1.088	52.1
0.08	0.0055	1.13	6.45	51.35	7.96	1.147	54.9
0.09	0.0058	1.27	6.46	54.16	8.39	1.208	57.8
0.09	0.0060	1.41	6.47	56.31	8.71	1.254	60.0
0.10	0.0062	1.55	6.48	58.19	8.99	1.294	62.0
0.11	0.0064	1.69	6.48	60.06	9.26	1.334	63.9
0.12	0.0066	1.83	6.49	61.94	9.54	1.373	65.8
0.13	0.0067	1.97	6.50	63.15	9.71	1.398	67.0
0.14	0.0069	2.11	6.51	64.47	9.90	1.425	68.2
0.14	0.0069	2.25	6.52	65.03	9.97	1.436	68.7
0.15	0.0070	2.39	6.53	65.59	10.04	1.446	69.2
0.16	0.0070	2.53	6.54	65.96	10.09	1.452	69.5
0.17	0.0070	2.67	6.55	65.59	10.01	1.442	69.0
0.18	0.0069	2.81	6.56	65.03	9.91	1.427	68.3
0.19	0.0068	2.95	6.57	63.25	9.63	1.386	66.4
0.20	0.0060	3.09	6.58	56.31	8.56	1.233	59.0

UNCONFINED COMPRESSIVE STRENGTH, q_u:	69.53	kPa
(based on maximum q_u value)	1.452	ksf
UNDRAINED SHEAR STRENGTH, S_u:	34.77	kPa
(based on maximum q_u value)	0.726	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-01
Sample: T4
Depth: 3.05 - 3.66 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

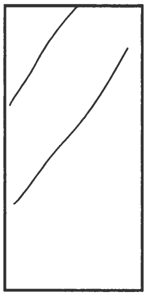
TEST HOLE NO.:	**TH14-01**
SAMPLE NO.:	**T11**
SAMPLE DEPTH:	9.14 - 9.75 m
DATE TESTED:	2-Sep-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.55
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**53.9**
Undrained Shear Strength (ksf)	1.13
POCKET PENETROMETER	
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**69.9**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**35.0**
Undrained Shear Strength (ksf)	**0.730**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	372.8
Wt. Sample dry + tare (g)	270.7
Wt. Tare (g)	8.3
Moisture Content %	**38.9**
BULK DENSITY	
Sample Wt. (g)	1072.3
Diameter 1 (cm)	7.22
Diameter 2 (cm)	7.23
Diameter 3 (cm)	7.23
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	15.33
Length 2 (cm)	15.34
Length 3 (cm)	15.32
Avg. Length (cm)	**15.33**
Volume (cm^3)	628.8
Moisture content (%)	38.9
Bulk Density (g/cm^3)	1.705
Bulk Density (kN/m^3)	**16.7**
Bulk Density (pcf)	**106.5**
Dry Density (kN/m^3)	**12.04**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-01
SAMPLE NO.:	T11
SAMPLE DEPTH:	9.14 - 9.75 m
SAMPLE DATE:	February, 2014
TEST DATE:	2-Sep-14

SOIL DESCRIPTION:	
CLAY; trace sand, trace silt inclusions, trace gravel (5mm), brown, moist, firm	
high plasticity,	
MOISTURE CONTENT:	38.9



FAILURE SKETCH

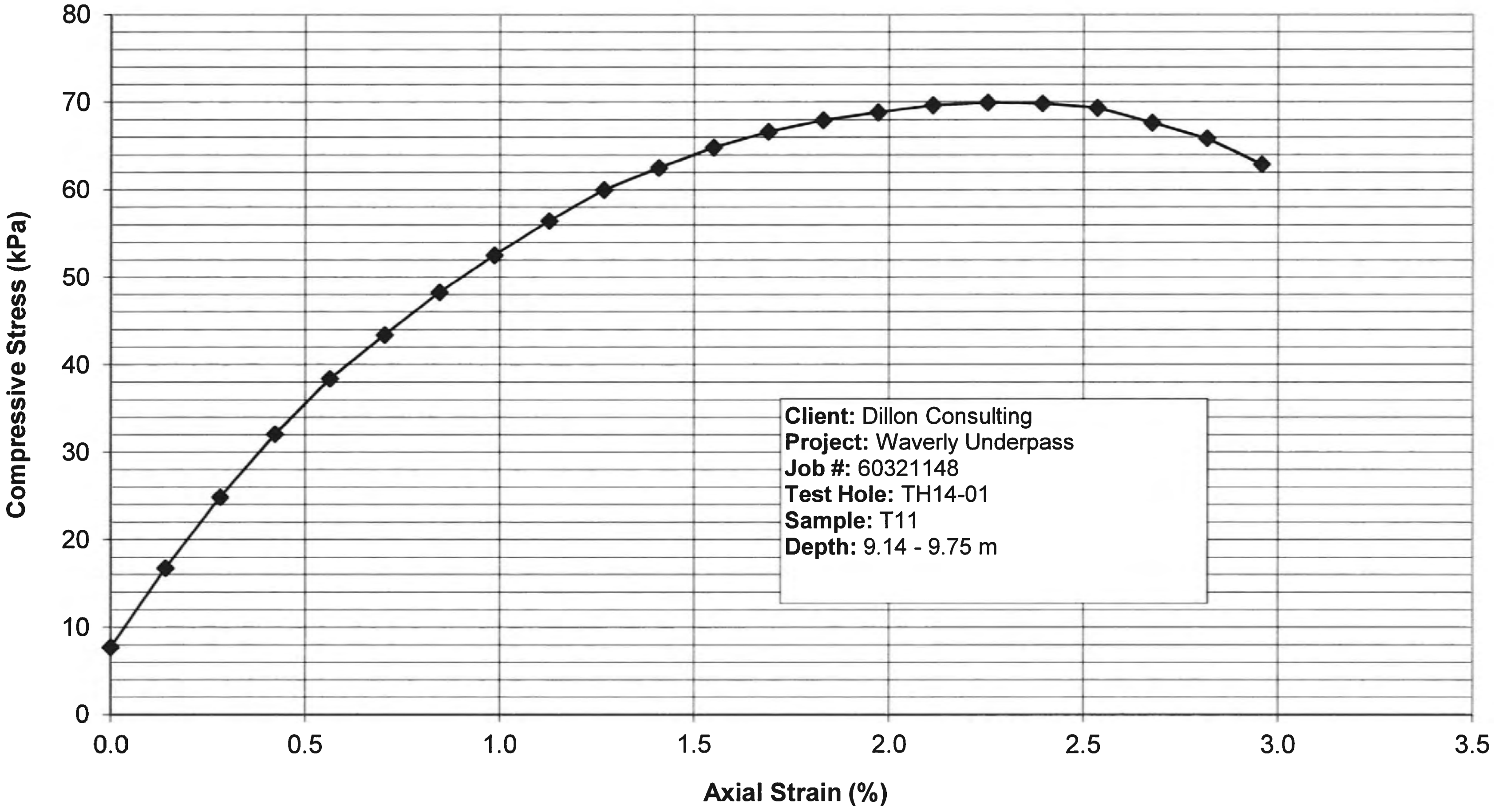
SAMPLE DIAM.(Do):	72.27	**(mm)**	**INITIAL AREA, Ao:**	4101.7	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.30	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.85	**(0.5<R<2 % / minute)**

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_c**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0008	0.00	6.36	7.12	1.12	0.161	7.7
0.02	0.0017	0.14	6.37	15.46	2.43	0.350	16.7
0.03	0.0025	0.28	6.38	22.96	3.60	0.518	24.8
0.03	0.0032	0.42	6.38	29.70	4.65	0.670	32.1
0.04	0.0038	0.56	6.39	35.61	5.57	0.802	38.4
0.05	0.0043	0.70	6.40	40.29	6.29	0.906	43.4
0.06	0.0048	0.85	6.41	44.88	7.00	1.008	48.3
0.07	0.0052	0.99	6.42	48.91	7.62	1.097	52.5
0.08	0.0056	1.13	6.43	52.66	8.19	1.179	56.5
0.09	0.0060	1.27	6.44	56.03	8.70	1.253	60.0
0.09	0.0062	1.41	6.45	58.47	9.07	1.306	62.5
0.10	0.0065	1.55	6.46	60.72	9.40	1.354	64.8
0.11	0.0067	1.69	6.47	62.50	9.66	1.392	66.6
0.12	0.0068	1.83	6.48	63.81	9.85	1.419	67.9
0.13	0.0069	1.97	6.49	64.75	9.98	1.438	68.8
0.14	0.0070	2.11	6.49	65.59	10.10	1.454	69.6
0.14	0.0070	2.25	6.50	65.96	10.14	1.460	69.9
0.15	0.0070	2.39	6.51	65.96	10.13	1.458	69.8
0.16	0.0070	2.54	6.52	65.59	10.06	1.448	69.3
0.17	0.0068	2.68	6.53	64.09	9.81	1.413	67.6
0.18	0.0067	2.82	6.54	62.50	9.55	1.376	65.9
0.19	0.0064	2.96	6.55	59.78	9.12	1.314	62.9

UNCONFINED COMPRESSIVE STRENGTH, q_u:	69.93	kPa
(based on maximum q_u value)	1.460	ksf
UNDRAINED SHEAR STRENGTH, S_u:	34.96	kPa
(based on maximum q_u value)	0.730	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-01
Sample: T11
Depth: 9.14 - 9.75 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH14-02**
SAMPLE NO.:	**T18**
SAMPLE DEPTH:	4.57 - 5.18 m
DATE TESTED:	2-Sep-14
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.80
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**78.5**
Undrained Shear Strength (ksf)	1.64
POCKET PENETROMETER	
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**82.3**
Unconfined compressive strength (ksf)	**1.7**
Undrained Shear Strength (kPa)	**41.2**
Undrained Shear Strength (ksf)	**0.860**
MOISTURE CONTENT	
Tare Number	SG36
Wt. Sample wet + tare (g)	416.1
Wt. Sample dry + tare (g)	285.3
Wt. Tare (g)	9.3
Moisture Content %	**47.4**
BULK DENSITY	
Sample Wt. (g)	1080.9
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.24
Diameter 3 (cm)	7.21
Avg. Diameter (cm)	**7.22**
Length 1 (cm)	15.34
Length 2 (cm)	15.33
Length 3 (cm)	15.35
Avg. Length (cm)	**15.34**
Volume (cm^3)	627.5
Moisture content (%)	47.4
Bulk Density (g/cm^3)	1.723
Bulk Density (kN/m^3)	**16.9**
Bulk Density (pcf)	**107.5**
Dry Density (kN/m^3)	**11.46**

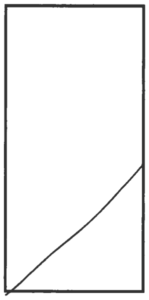
AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH14-02
SAMPLE NO.:	T18
SAMPLE DEPTH:	4.57 - 5.18 m
SAMPLE DATE:	February, 2014
TEST DATE:	2-Sep-14

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, brown, moist, firm, high plasticity,	
MOISTURE CONTENT:	47.4

SAMPLE DIAM.(Do):	72.17	**(mm)**	**INITIAL AREA, Ao:**	4090.4	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.40	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.13	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.84	**(0.5<R<2 % / minute)**



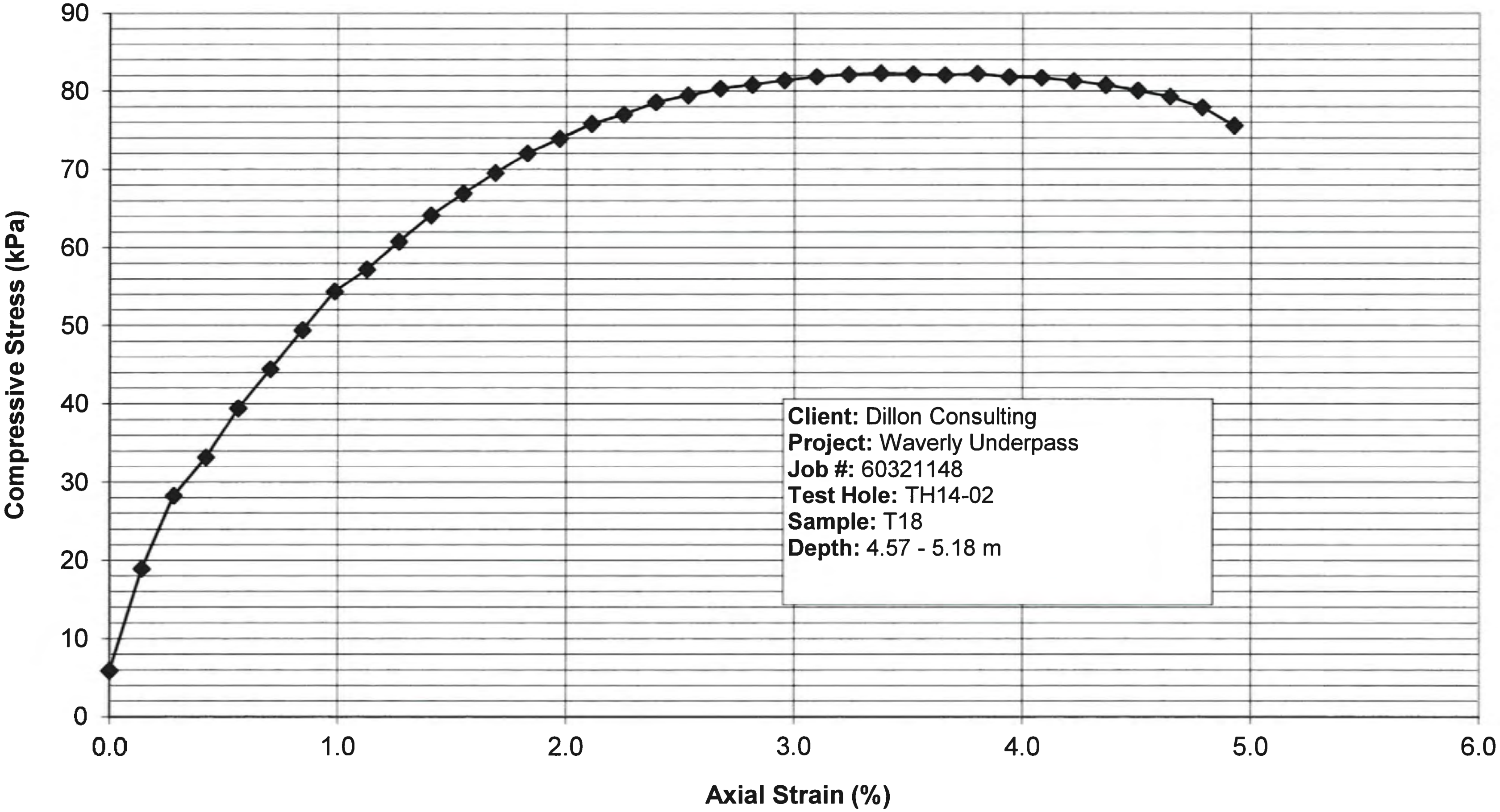
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_c		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0006	0.00	6.34	5.43	0.86	0.123	5.9
0.02	0.0019	0.14	6.35	17.43	2.75	0.395	18.9
0.03	0.0028	0.28	6.36	26.05	4.10	0.590	28.2
0.03	0.0033	0.42	6.37	30.64	4.81	0.693	33.2
0.04	0.0039	0.56	6.38	36.45	5.72	0.823	39.4
0.05	0.0044	0.70	6.39	41.13	6.44	0.928	44.4
0.06	0.0049	0.84	6.39	45.82	7.17	1.032	49.4
0.07	0.0054	0.99	6.40	50.50	7.89	1.136	54.4
0.08	0.0057	1.13	6.41	53.22	8.30	1.195	57.2
0.09	0.0060	1.27	6.42	56.59	8.81	1.269	60.8
0.09	0.0064	1.41	6.43	59.78	9.30	1.339	64.1
0.10	0.0067	1.55	6.44	62.50	9.70	1.398	66.9
0.11	0.0069	1.69	6.45	65.03	10.08	1.452	69.5
0.12	0.0072	1.83	6.46	67.46	10.45	1.504	72.0
0.13	0.0074	1.97	6.47	69.34	10.72	1.544	73.9
0.14	0.0076	2.11	6.48	71.21	10.99	1.583	75.8
0.14	0.0077	2.25	6.49	72.43	11.17	1.608	77.0
0.15	0.0079	2.39	6.50	74.02	11.40	1.641	78.6
0.16	0.0080	2.53	6.50	74.96	11.52	1.659	79.5
0.17	0.0081	2.67	6.51	75.90	11.65	1.678	80.3
0.18	0.0082	2.81	6.52	76.46	11.72	1.688	80.8
0.19	0.0082	2.96	6.53	77.12	11.80	1.700	81.4
0.20	0.0083	3.10	6.54	77.68	11.87	1.710	81.9
0.20	0.0083	3.24	6.55	78.05	11.91	1.715	82.1
0.21	0.0084	3.38	6.56	78.33	11.94	1.719	82.3
0.22	0.0084	3.52	6.57	78.33	11.92	1.717	82.2
0.23	0.0084	3.66	6.58	78.33	11.90	1.714	82.1
0.24	0.0084	3.80	6.59	78.61	11.93	1.718	82.2
0.25	0.0084	3.94	6.60	78.33	11.87	1.709	81.8
0.26	0.0084	4.08	6.61	78.33	11.85	1.707	81.7
0.26	0.0083	4.22	6.62	78.05	11.79	1.698	81.3
0.27	0.0083	4.36	6.63	77.68	11.72	1.687	80.8
0.28	0.0082	4.50	6.64	77.12	11.62	1.673	80.1
0.29	0.0082	4.64	6.65	76.46	11.50	1.656	79.3
0.30	0.0080	4.79	6.66	75.24	11.30	1.627	77.9
0.31	0.0078	4.93	6.67	73.09	10.96	1.578	75.6

UNCONFINED COMPRESSIVE STRENGTH, q_u:	82.31	kPa
(based on maximum q_u value)	1.719	ksf
UNDRAINED SHEAR STRENGTH, S_u:	41.15	kPa
(based on maximum q_u value)	0.860	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH14-02
Sample: T18
Depth: 4.57 - 5.18 m

AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Memorandum

To	Saba Ibrahim	Page	1
CC			
Subject	Dillon Consulting Ltd. - Waverly Underpass DD – Materials Testing Results		
From	Zeyad Shukri		
Date	April 26, 2016	Project Number	60321148

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Fifty (50) Moisture Content tests.
- One (1) Atterberg Limits (3 points) tests.
- Four (4) Torvane, Pocket Penetrometer, Moisture Content, Bulk Density and Visual Description with Unconfined Compressive Strength, on Shelby tube samples.

If you have any questions, please contact the undersigned.

Sincerely,

Zeyad Shukri Al-Hayazai, M.Sc., P.Eng.
Senior Geotechnical Engineer

Att.

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name: Waverley Underpass
Project Number: 60321148
Client: City of Winnipeg
Sample Location: Varies
Sample Depth: Varies
Sample Number: Varies

Supplier: AECOM
Specification: N/A
Field Technician: SIbrahim
Sample Date: Varies
Lab Technician: EManimbao
Date Tested: April 19, 2016

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	Moisture Content (%)
TH16-01	S1	4.57 - 5.03 m	**50.8%**
	G2	5.33 - 5.49 m	**47.4%**
	G3	6.71 - 6.86 m	**51.7%**
	S4	7.62 - 8.08 m	**45.7%**
	S5	9.14 - 9.60 m	**46.2%**
	T6	10.67 - 11.28 m	**34.0%**
	S7	12.19 - 12.65 m	**40.2%**
	S8	13.72 - 14.17 m	**11.0%**
	S9	15.24 - 15.70 m	**21.4%**
	S10	16.76 - 17.22 m	**19.8%**
	G11	17.37 - 17.53 m	**12.5%**
	S12	18.29 - 18.75 m	**26.8%**
TH16-02	S13	3.05 - 3.51 m	**41.9%**
	T14	4.57 - 5.18 m	**-**
	S15	6.10 - 6.55 m	**50.3%**
	T16	7.62 - 8.23 m	**-**
	G17	9.14 - 9.30 m	**49.7%**
	S18	10.67 - 11.13 m	**57.6%**
	G19	12.50 - 12.65 m	**12.5%**
	S20	13.41 - 13.87 m	**10.3%**
	S21	13.72 - 14.17 m	**9.9%**
	G22	14.48 - 14.63 m	**6.6%**
	S23	15.24 - 15.70 m	**11.3%**
	G24	15.85 - 16.00 m	**11.2%**
TH16-03	S25	3.05 - 3.51 m	**39.7%**
	G26	4.57 - 4.72 m	**56.2%**
	G27	6.10 - 6.25 m	**52.4%**
	G28	7.62 - 7.77 m	**49.1%**
	T29	9.14 - 9.75 m	**-**
	G30	10.67 - 10.82 m	**66.4%**
	S31	12.19 - 12.65 m	**24.1%**
	G32	12.95 - 13.11 m	**12.9%**
	S33	13.72 - 14.17 m	**15.5%**
	G34	14.63 - 14.78 m	**7.3%**
	S35	15.24 - 15.70 m	**17.4%**
TH16-04	G36	0.61 - 0.76 m	**28.8%**
	G37	1.37 - 1.52 m	**33.0%**
	G38	2.44 - 2.59 m	**39.6%**

Location	Sample	Depth (m)	Moisture Content (%)
	G39	3.81 - 3.96 m	**55.6%**
	T40	4.57 - 5.18 m	**55.0%**
	T41	6.10 - 6.71 m	**60.1%**
	T42	7.62 - 8.23 m	**-**
	G43	8.53 - 8.69 m	**51.9%**
	G44	10.06 - 10.21 m	**46.2%**
TH16-05	G45	0.30 - 0.46 m	**43.3%**
	G46	1.07 - 1.22 m	**46.2%**
	G47	2.29 - 2.44 m	**39.0%**
	G48	3.81 - 3.96 m	**53.2%**
	T49	4.57 - 5.18 m	**-**
	T50	6.10 - 6.71 m	**52.0%**
	T51	7.62 - 8.23 m	**50.6%**
	G52	9.91 - 10.06 m	**56.6%**
	G53	11.43 - 11.58 m	**39.3%**
	G54	12.80 - 12.95 m	**27.4%**
	G55	13.41 - 13.56 m	**10.4%**
			-
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AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

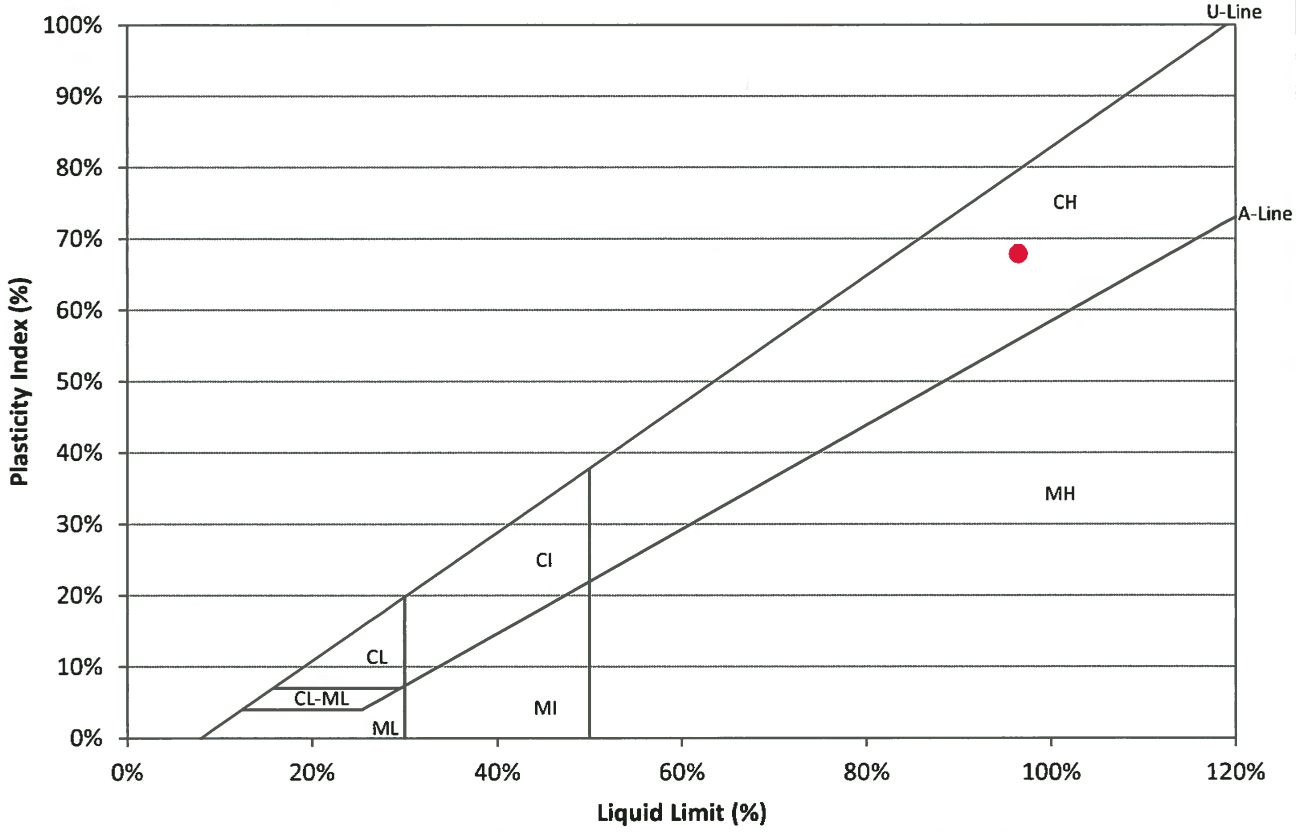
Project Name:	Waverly Underpass Phase II	Supplier:	AECOM
Project Number:	60321148	Specification:	N/A
Client:	Dillon Consulting	Field Technician:	SIbrahim
Sample Location:	TH16-04	Sample Date:	Varies
Sample Depth:	6.10 - 6.71 m	Lab Technician:	MLotecki
Sample Number:	T41	Date Tested:	April 22, 2016

Atterberg Limits

ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	17	24	35
Wet Sample (g)	10.4	9.6	9.1
Dry Sample (g)	5.2	4.9	4.7
Water Content (%)	99.9%	97.0%	93.6%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.1	6.1
Dry Sample (g)	4.7	4.8
Water Content (%)	28.6%	28.7%



Liquid Limit (%): 96.5%	Plastic Limit (%): 28.7%	Plasticity Index (%): 67.8%

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH16-04**
SAMPLE NO.:	**T40**
SAMPLE DEPTH:	4.57 - 5.18 m
DATE TESTED:	22-Apr-16
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.75
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**73.6**
Undrained Shear Strength (ksf)	1.54
POCKET PENETROMETER	
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**79.8**
Unconfined compressive strength (ksf)	**1.7**
Undrained Shear Strength (kPa)	**39.9**
Undrained Shear Strength (ksf)	**0.834**
MOISTURE CONTENT	
Tare Number	X2
Wt. Sample wet + tare (g)	284.7
Wt. Sample dry + tare (g)	186.7
Wt. Tare (g)	8.4
Moisture Content %	**55.0**
BULK DENSITY	
Sample Wt. (g)	1054.6
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.22
Diameter 3 (cm)	7.21
Avg. Diameter (cm)	**7.21**
Length 1 (cm)	15.35
Length 2 (cm)	15.30
Length 3 (cm)	15.31
Avg. Length (cm)	**15.32**
Volume (cm^3)	625.5
Moisture content (%)	55.0
Bulk Density (g/cm^3)	1.686
Bulk Density (kN/m^3)	**16.5**
Bulk Density (pcf)	**105.3**
Dry Density (kN/m^3)	**10.67**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

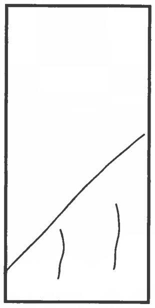
CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH16-04
SAMPLE NO.:	T40
SAMPLE DEPTH:	4.57 - 5.18 m
SAMPLE DATE:	February, 2014
TEST DATE:	22-Apr-16

SOIL DESCRIPTION:

CLAY; silty, trace till inclusions, trace sulphate inclusions, trace oxidation, brown, moist, brown, moist, firm, homogeneous, high plasticity

MOISTURE CONTENT: 55.0



FAILURE SKETCH

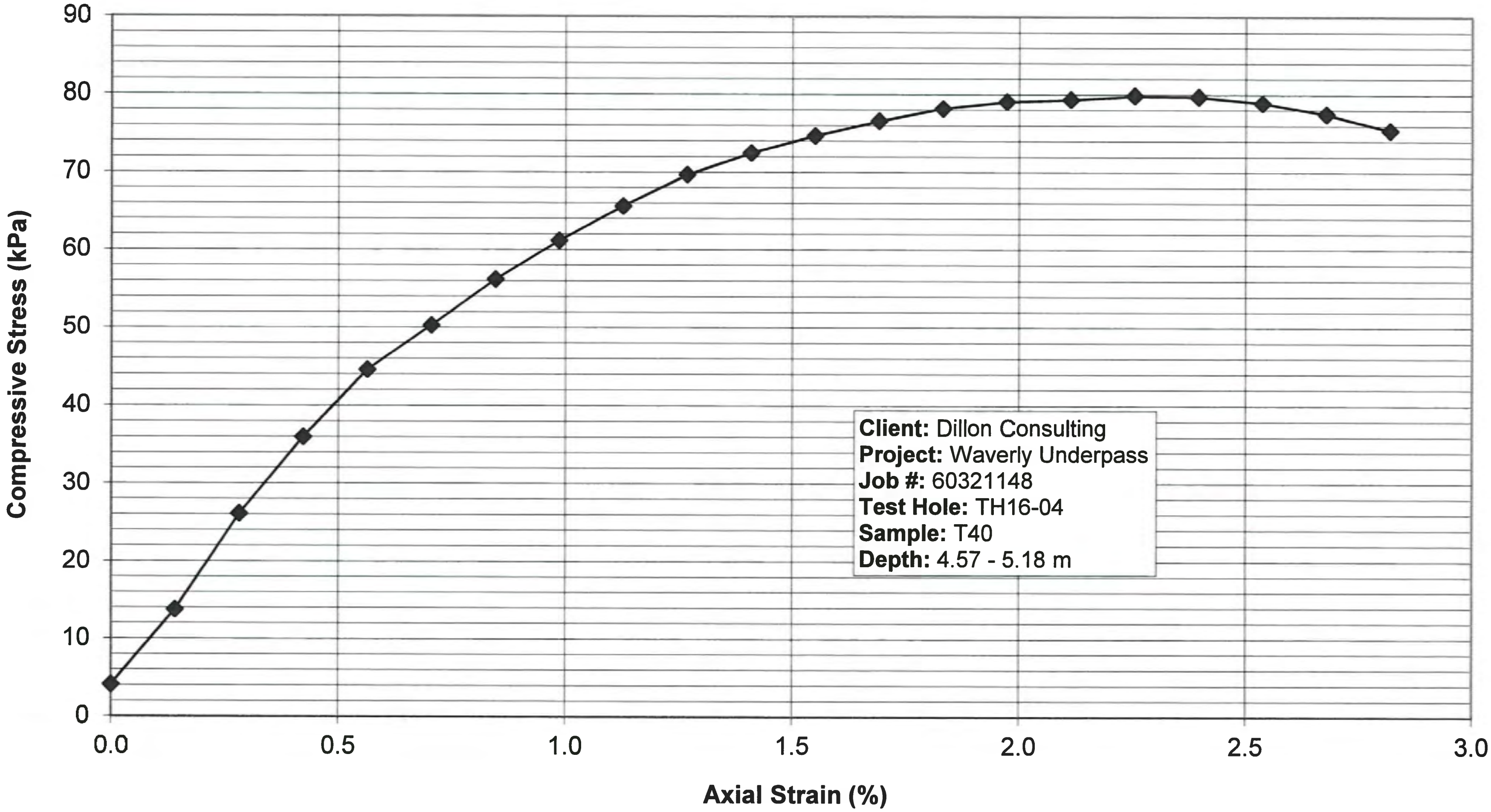
SAMPLE DIAM.(Do):	72.10	**(mm)**	**INITIAL AREA, Ao:**	4082.8	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.20	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.85	**(0.5<R<2 % / minute)**

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_C**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0004	0.00	6.33	3.75	0.59	0.085	4.1
0.02	0.0014	0.14	6.34	12.65	2.00	0.287	13.8
0.03	0.0026	0.28	6.35	23.99	3.78	0.544	26.1
0.03	0.0035	0.42	6.36	33.17	5.22	0.752	36.0
0.04	0.0044	0.56	6.36	41.13	6.46	0.931	44.6
0.05	0.0050	0.70	6.37	46.48	7.29	1.050	50.3
0.06	0.0056	0.85	6.38	52.00	8.15	1.173	56.2
0.07	0.0061	0.99	6.39	56.69	8.87	1.277	61.2
0.08	0.0065	1.13	6.40	60.91	9.52	1.370	65.6
0.09	0.0069	1.27	6.41	64.75	10.10	1.455	69.6
0.09	0.0072	1.41	6.42	67.46	10.51	1.513	72.5
0.10	0.0074	1.55	6.43	69.62	10.83	1.560	74.7
0.11	0.0076	1.69	6.44	71.49	11.11	1.599	76.6
0.12	0.0078	1.83	6.45	73.09	11.34	1.633	78.2
0.13	0.0079	1.97	6.46	74.02	11.47	1.651	79.1
0.14	0.0079	2.11	6.47	74.40	11.51	1.657	79.3
0.14	0.0080	2.25	6.47	74.96	11.58	1.667	79.8
0.15	0.0080	2.40	6.48	74.96	11.56	1.665	79.7
0.16	0.0079	2.54	6.49	74.30	11.44	1.648	78.9
0.17	0.0078	2.68	6.50	73.09	11.24	1.619	77.5
0.18	0.0076	2.82	6.51	71.21	10.94	1.575	75.4

UNCONFINED COMPRESSIVE STRENGTH, q_u:	79.83	kPa
(based on maximum q_u value)	1.667	ksf
UNDRAINED SHEAR STRENGTH, S_u:	39.91	kPa
(based on maximum q_u value)	0.834	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH16-04
Sample: T40
Depth: 4.57 - 5.18 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH16-04**
SAMPLE NO.:	**T41**
SAMPLE DEPTH:	6.10 - 6.71 m
DATE TESTED:	22-Apr-16
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.70
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**68.7**
Undrained Shear Strength (ksf)	1.43
POCKET PENETROMETER	
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
Reading - Qu (tsf)	1.25
Undrained Shear Strength (kPa)	**59.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**96.3**
Unconfined compressive strength (ksf)	**2.0**
Undrained Shear Strength (kPa)	**48.1**
Undrained Shear Strength (ksf)	**1.005**
MOISTURE CONTENT	
Tare Number	105
Wt. Sample wet + tare (g)	908.7
Wt. Sample dry + tare (g)	710.3
Wt. Tare (g)	380.1
Moisture Content %	**60.1**
BULK DENSITY	
Sample Wt. (g)	1058.3
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.22
Diameter 3 (cm)	7.23
Avg. Diameter (cm)	**7.22**
Length 1 (cm)	15.35
Length 2 (cm)	15.30
Length 3 (cm)	15.33
Avg. Length (cm)	**15.33**
Volume (cm^3)	626.9
Moisture content (%)	60.1
Bulk Density (g/cm^3)	1.688
Bulk Density (kN/m^3)	**16.6**
Bulk Density (pcf)	**105.4**
Dry Density (kN/m^3)	**10.34**

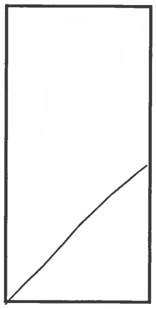
AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

AECOM

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH16-04
SAMPLE NO.:	T41
SAMPLE DEPTH:	6.10 - 6.71 m
SAMPLE DATE:	February, 2014
TEST DATE:	22-Apr-16

SOIL DESCRIPTION:	
CLAY; silty, trace till inclusions, trace sand, brown, moist, firm	
homogeneous, high plasticity	
MOISTURE CONTENT:	60.1



FAILURE SKETCH

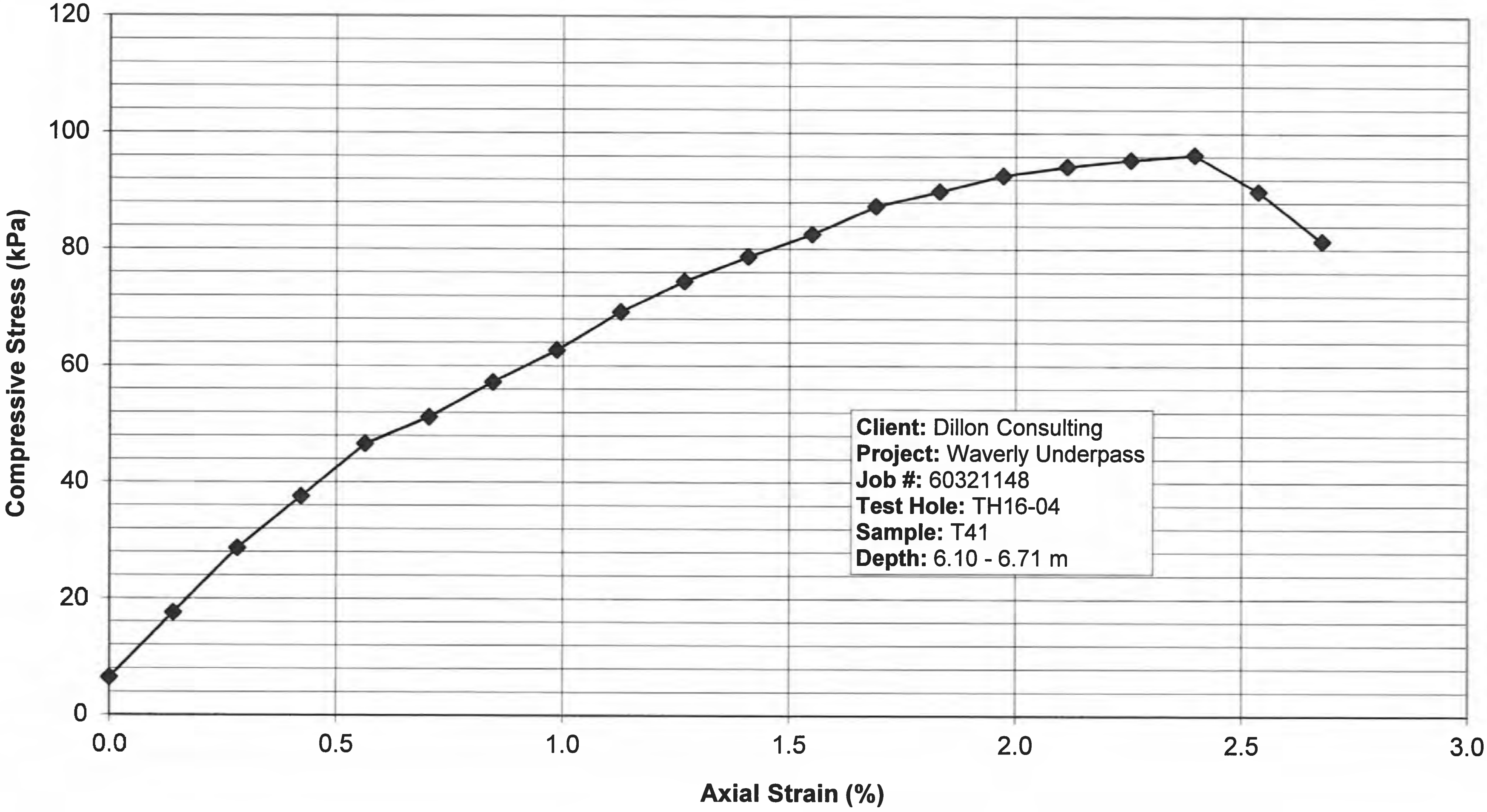
SAMPLE DIAM.(Do):	72.17	**(mm)**	**INITIAL AREA, Ao:**	4090.4	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.27	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.85	**(0.5<R<2 % / minute)**

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_c**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0006	0.00	6.34	5.90	0.93	0.134	6.4
0.02	0.0017	0.14	6.35	16.21	2.55	0.368	17.6
0.03	0.0028	0.28	6.36	26.42	4.16	0.598	28.7
0.03	0.0037	0.42	6.37	34.67	5.45	0.784	37.5
0.04	0.0046	0.56	6.38	43.10	6.76	0.973	46.6
0.05	0.0051	0.70	6.39	47.41	7.43	1.069	51.2
0.06	0.0057	0.85	6.39	53.03	8.29	1.194	57.2
0.07	0.0062	0.99	6.40	58.19	9.09	1.309	62.7
0.08	0.0069	1.13	6.41	64.37	10.04	1.446	69.2
0.09	0.0074	1.27	6.42	69.34	10.80	1.555	74.4
0.09	0.0078	1.41	6.43	73.37	11.41	1.643	78.7
0.10	0.0082	1.55	6.44	77.12	11.97	1.724	82.6
0.11	0.0087	1.69	6.45	81.71	12.67	1.824	87.4
0.12	0.0090	1.83	6.46	84.24	13.04	1.878	89.9
0.13	0.0093	1.97	6.47	86.95	13.44	1.936	92.7
0.14	0.0095	2.11	6.48	88.55	13.67	1.969	94.3
0.14	0.0096	2.25	6.49	89.76	13.84	1.993	95.4
0.15	0.0097	2.39	6.50	90.70	13.96	2.011	96.3
0.16	0.0091	2.54	6.51	84.89	13.05	1.879	90.0
0.17	0.0082	2.68	6.51	76.93	11.81	1.700	81.4

UNCONFINED COMPRESSIVE STRENGTH, q_u:	96.27	kPa
(based on maximum q_u value)	2.011	ksf
UNDRAINED SHEAR STRENGTH, S_u:	48.14	kPa
(based on maximum q_u value)	1.005	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH16-04
Sample: T41
Depth: 6.10 - 6.71 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH16-05**
SAMPLE NO.:	**T50**
SAMPLE DEPTH:	6.10 - 6.71 m
DATE TESTED:	22-Apr-16
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.60
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**58.8**
Undrained Shear Strength (ksf)	1.23
POCKET PENETROMETER	
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**88.2**
Unconfined compressive strength (ksf)	**1.8**
Undrained Shear Strength (kPa)	**44.1**
Undrained Shear Strength (ksf)	**0.921**
MOISTURE CONTENT	
Tare Number	SG6
Wt. Sample wet + tare (g)	243.7
Wt. Sample dry + tare (g)	163.2
Wt. Tare (g)	8.5
Moisture Content %	**52.0**
BULK DENSITY	
Sample Wt. (g)	1062.1
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.23
Diameter 3 (cm)	7.20
Avg. Diameter (cm)	**7.21**
Length 1 (cm)	15.30
Length 2 (cm)	15.33
Length 3 (cm)	15.29
Avg. Length (cm)	**15.31**
Volume (cm^3)	624.9
Moisture content (%)	52.0
Bulk Density (g/cm^3)	1.700
Bulk Density (kN/m^3)	**16.7**
Bulk Density (pcf)	**106.1**
Dry Density (kN/m^3)	**10.96**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

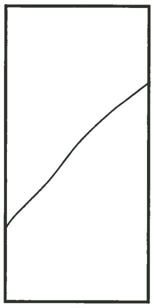
AECOM

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

TEST HOLE NO.:	TH16-05
SAMPLE NO.:	T50
SAMPLE DEPTH:	6.10 - 6.71 m
SAMPLE DATE:	February, 2014
TEST DATE:	22-Apr-16

SOIL DESCRIPTION:	
CLAY; silty, trace silt inclusions, brown, moist, firm, homogeneous,	
high plasticity	
MOISTURE CONTENT:	52.0

SAMPLE DIAM.(Do):	72.10	**(mm)**	**INITIAL AREA, Ao:**	4082.8	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.07	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.85	**(0.5<R<2 % / minute)**



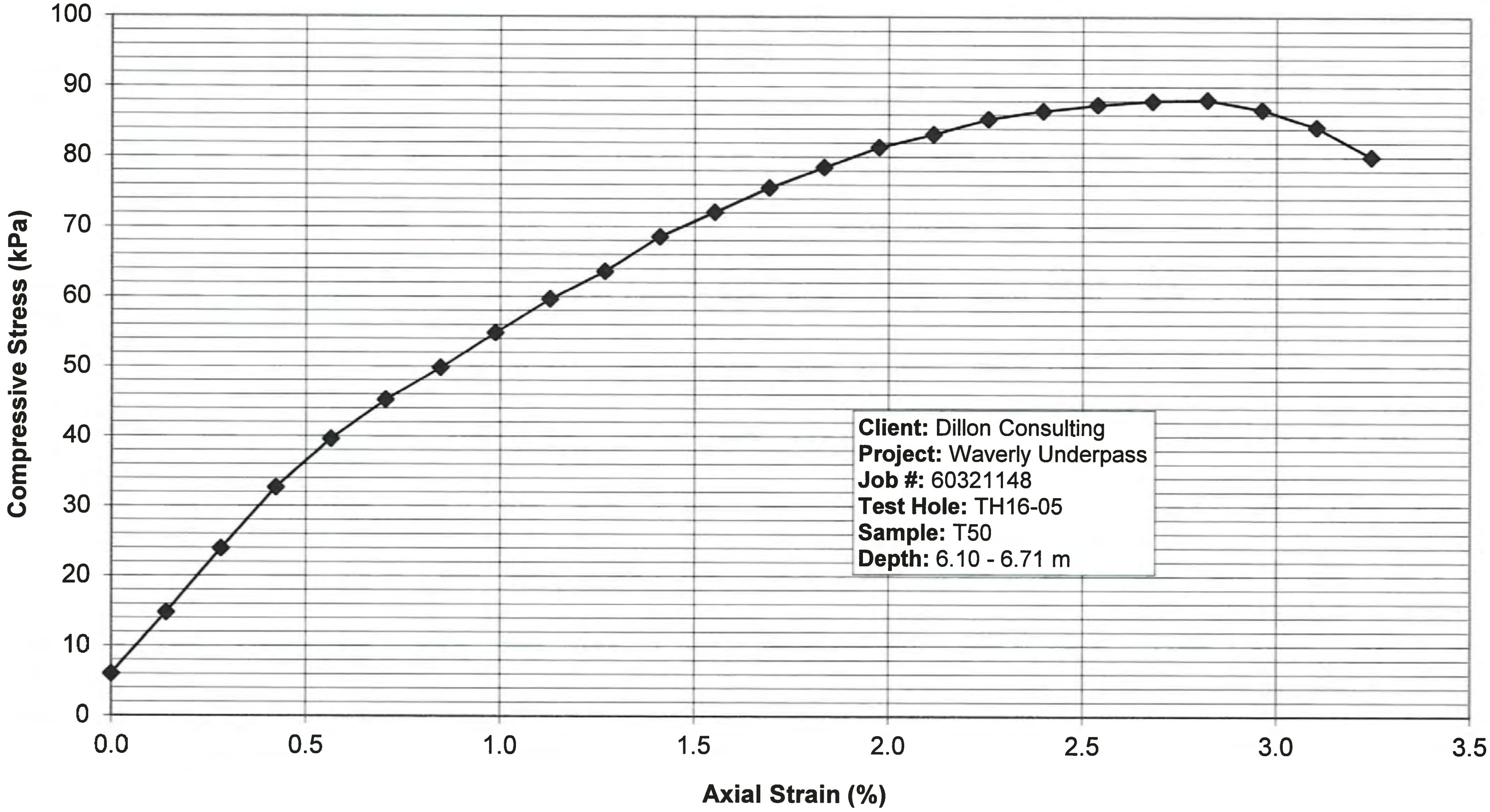
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_c		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0006	0.00	6.33	5.53	0.87	0.126	6.0
0.02	0.0015	0.14	6.34	13.59	2.14	0.309	14.8
0.03	0.0024	0.28	6.35	22.02	3.47	0.500	23.9
0.03	0.0032	0.42	6.36	30.08	4.73	0.682	32.6
0.04	0.0039	0.56	6.36	36.54	5.74	0.827	39.6
0.05	0.0045	0.71	6.37	41.79	6.56	0.944	45.2
0.06	0.0049	0.85	6.38	46.10	7.22	1.040	49.8
0.07	0.0054	0.99	6.39	50.79	7.95	1.144	54.8
0.08	0.0059	1.13	6.40	55.38	8.65	1.246	59.7
0.09	0.0063	1.27	6.41	59.12	9.22	1.328	63.6
0.09	0.0068	1.41	6.42	63.81	9.94	1.431	68.5
0.10	0.0072	1.55	6.43	67.18	10.45	1.505	72.1
0.11	0.0075	1.69	6.44	70.56	10.96	1.578	75.6
0.12	0.0078	1.83	6.45	73.37	11.38	1.639	78.5
0.13	0.0081	1.97	6.46	76.18	11.80	1.699	81.4
0.14	0.0083	2.12	6.47	78.05	12.07	1.738	83.2
0.14	0.0086	2.26	6.47	80.21	12.39	1.784	85.4
0.15	0.0087	2.40	6.48	81.43	12.56	1.808	86.6
0.16	0.0088	2.54	6.49	82.36	12.68	1.827	87.5
0.17	0.0089	2.68	6.50	83.02	12.77	1.838	88.0
0.18	0.0089	2.82	6.51	83.30	12.79	1.842	88.2
0.19	0.0088	2.96	6.52	82.08	12.59	1.812	86.8
0.20	0.0085	3.10	6.53	79.83	12.22	1.760	84.3
0.20	0.0081	3.24	6.54	75.90	11.60	1.671	80.0

UNCONFINED COMPRESSIVE STRENGTH, q_u:	88.19	kPa
(based on maximum q_u value)	1.842	ksf
UNDRAINED SHEAR STRENGTH, S_u:	44.10	kPa
(based on maximum q_u value)	0.921	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH16-05
Sample: T50
Depth: 6.10 - 6.71 m

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

AECOM

CLIENT: Dillon Consulting
PROJECT: Waverly Underpass
JOB NO.: 60321148

TEST HOLE NO.:	**TH16-05**
SAMPLE NO.:	**T51**
SAMPLE DEPTH:	7.62 - 8.23 m
DATE TESTED:	22-Apr-16
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.40
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**39.2**
Undrained Shear Strength (ksf)	0.82
POCKET PENETROMETER	
Reading - Qu (tsf)	0.25
Undrained Shear Strength (kPa)	**12.0**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
Reading - Qu (tsf)	0.50
Undrained Shear Strength (kPa)	**23.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**97.5**
Unconfined compressive strength (ksf)	**2.0**
Undrained Shear Strength (kPa)	**48.8**
Undrained Shear Strength (ksf)	**1.018**
MOISTURE CONTENT	
Tare Number	J32
Wt. Sample wet + tare (g)	301.5
Wt. Sample dry + tare (g)	203.1
Wt. Tare (g)	8.5
Moisture Content %	**50.6**
BULK DENSITY	
Sample Wt. (g)	1103.8
Diameter 1 (cm)	7.22
Diameter 2 (cm)	7.20
Diameter 3 (cm)	7.22
Avg. Diameter (cm)	**7.21**
Length 1 (cm)	15.30
Length 2 (cm)	15.33
Length 3 (cm)	15.33
Avg. Length (cm)	**15.32**
Volume (cm^3)	626.1
Moisture content (%)	50.6
Bulk Density (g/cm^3)	1.763
Bulk Density (kN/m^3)	**17.3**
Bulk Density (pcf)	**110.1**
Dry Density (kN/m^3)	**11.48**

AECOM

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

CLIENT:	Dillon Consulting
PROJECT:	Waverly Underpass
JOB NO.:	60247924

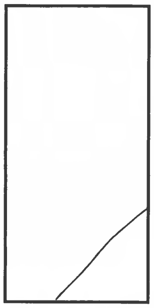
TEST HOLE NO.:	TH16-05
SAMPLE NO.:	T51
SAMPLE DEPTH:	7.62 - 8.23 m
SAMPLE DATE:	February, 2014
TEST DATE:	22-Apr-16

SOIL DESCRIPTION:

CLAY; silty, trace till inclusions, trace sand, trace gravel, brown/grey, moist, firm, homogeneous, high plasticity

MOISTURE CONTENT: 50.6

SAMPLE DIAM.(Do):	72.13	**(mm)**	**INITIAL AREA, Ao:**	4086.6	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.20	**(mm)**	**PISTON RATE:**	0.051	**(inches / minute)**
L / D RATIO:	2.12	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	0.85	**(0.5<R<2 % / minute)**



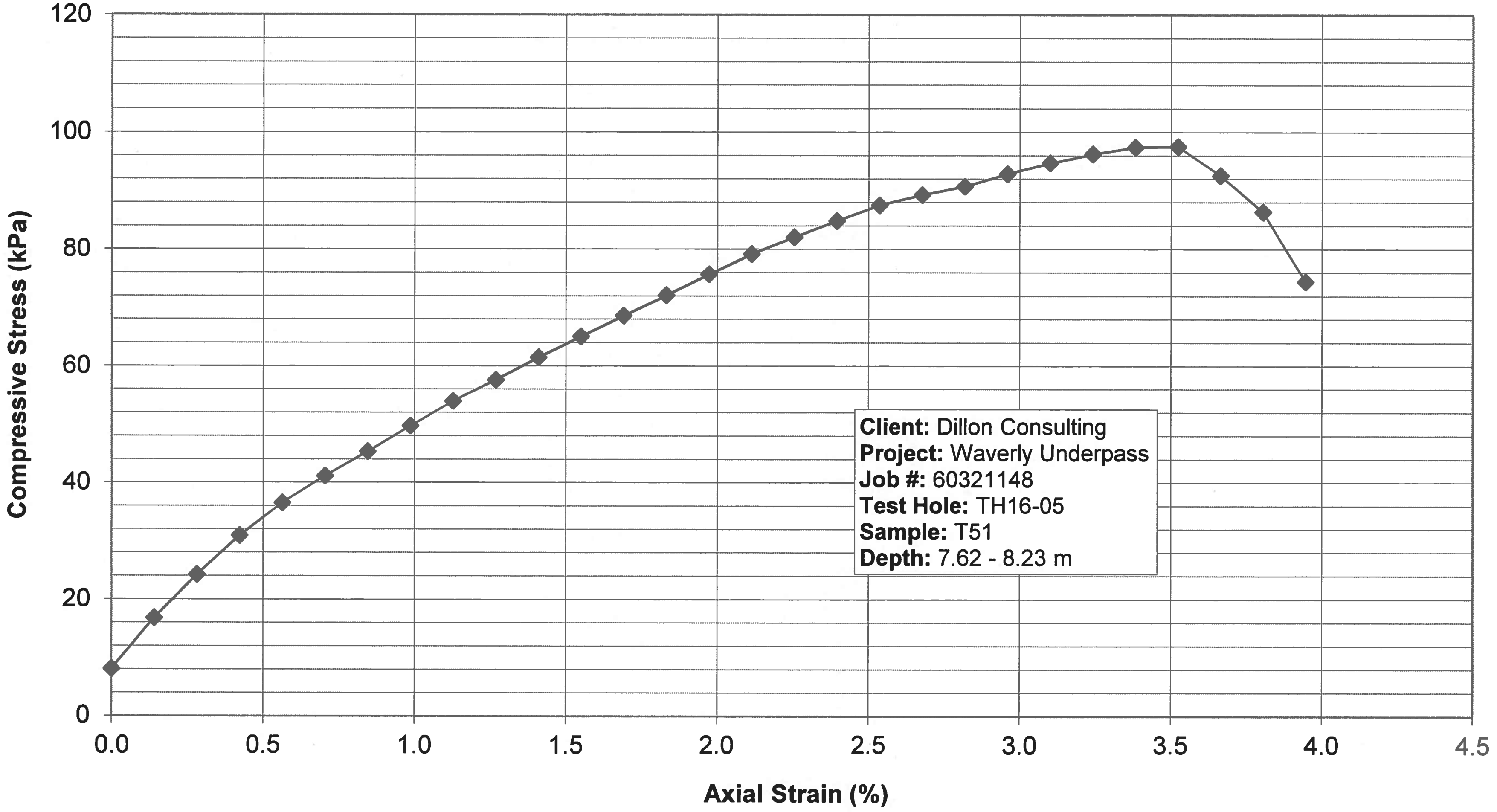
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_c**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0008	0.00	6.33	7.40	1.17	0.168	8.1
0.02	0.0017	0.14	6.34	15.46	2.44	0.351	16.8
0.03	0.0024	0.28	6.35	22.30	3.51	0.506	24.2
0.03	0.0030	0.42	6.36	28.48	4.48	0.645	30.9
0.04	0.0036	0.56	6.37	33.73	5.30	0.763	36.5
0.05	0.0041	0.70	6.38	38.04	5.96	0.859	41.1
0.06	0.0045	0.85	6.39	41.98	6.57	0.946	45.3
0.07	0.0049	0.99	6.40	46.10	7.21	1.038	49.7
0.08	0.0054	1.13	6.41	50.13	7.82	1.127	54.0
0.09	0.0057	1.27	6.42	53.60	8.35	1.203	57.6
0.09	0.0061	1.41	6.42	57.25	8.91	1.283	61.4
0.10	0.0065	1.55	6.43	60.72	9.44	1.359	65.1
0.11	0.0068	1.69	6.44	64.09	9.95	1.432	68.6
0.12	0.0072	1.83	6.45	67.46	10.46	1.506	72.1
0.13	0.0076	1.97	6.46	70.93	10.98	1.581	75.7
0.14	0.0079	2.11	6.47	74.30	11.48	1.653	79.2
0.14	0.0082	2.25	6.48	77.12	11.90	1.714	82.0
0.15	0.0085	2.40	6.49	79.83	12.30	1.771	84.8
0.16	0.0088	2.54	6.50	82.46	12.69	1.827	87.5
0.17	0.0090	2.68	6.51	84.24	12.94	1.864	89.2
0.18	0.0092	2.82	6.52	85.74	13.15	1.894	90.7
0.19	0.0094	2.96	6.53	87.89	13.46	1.939	92.8
0.20	0.0096	3.10	6.54	89.76	13.73	1.977	94.7
0.20	0.0098	3.24	6.55	91.36	13.96	2.010	96.2
0.21	0.0099	3.38	6.56	92.58	14.12	2.033	97.4
0.22	0.0099	3.52	6.57	92.86	14.14	2.037	97.5
0.23	0.0094	3.66	6.58	88.27	13.42	1.933	92.6
0.24	0.0088	3.81	6.58	82.46	12.52	1.803	86.3
0.25	0.0076	3.95	6.59	71.21	10.80	1.555	74.5

UNCONFINED COMPRESSIVE STRENGTH, q_u:	97.51	kPa
(based on maximum q_u value)	2.037	ksf
UNDRAINED SHEAR STRENGTH, S_u:	48.76	kPa
(based on maximum q_u value)	1.018	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Waverly Underpass
Job #: 60321148
Test Hole: TH16-05
Sample: T51
Depth: 7.62 - 8.23 m

ENG-TECH
CONSULTING LIMITED

Unit 6 - 854 Marion Street
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ROCK CORE

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7

File No.: 14-027-01
Ref. No.: 14-27-1-10

Attention: Saba Ibrahim

Project: WAVERLY UNDERPASS; PROJECT # 60321148

Contractor: -
Date Cored: July 10, 14 and 15
Cored By: Client

Page: 1 of 1
Date Received: Nov 10/14
Received By: ENG-TECH

Core No.	Location	Length		Average Diameter (mm)	Compressive Strength (MPa)	Date Tested (m/d/y)
		Cored (mm)	Tested (mm)			
1	TH 14-02; sample No. R7, 24.0 – 24.3m.	254	113	63.0	194.4	Nov 13/14
2	TH 14-03; sample No. C6, 22.48 – 22.80m.	331	116	60.8	120.9	Nov 13/14
3	TH 14-04; sample No. R9, 25.4 – 25.6m.	244	118	60.9	114.9	Nov 13/14

__ METHOD ASTM D 2938
__ METHOD OTHER

__ MOISTURE CONDITIONED
__DRY CONDITIONED

_x_OTHER (As received)

Comments: The unconfined strength was determined in accordance with ASTM D2938-95 procedure with the cores in the as received moisture content. Core # 3 contained a vertical crack from the top to the bottom of specimen (as received).

Email: saba.ibrahim@aecom.com

ENG-TECH Consulting Limited

per______________________
Danny Holfeld, Principal
Ph: (204) 233-1694 Fx: (204) 235-1579

ENG-TECH
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eng_tech@mts.net
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ROCK CORE

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7

File No.: 16-027-01

Ref. No.: 16-27-1-5

Attention: Saba Ibrahim

Project: WAVERLEY UNDERPASS; PROJECT NO. 60321148

Contractor: Maple Leaf
Date Cored: Apr 13 and 19/16
Cored By: Client

Page: 1 of 1
Date Received: Jun 7/16
Received By: ENG-TECH (Paul L'Angais)

Core No.	Location	Length		Average Diameter (mm)	Compressive Strength (MPa)	Date Tested (m/d/y)
		Cored (mm)	Tested (mm)			
1	C5, TH16-01, Depth: 80' - 85'	2040	122.5	60.5	107.7	Jun 15/16
2	C10, TH16-03, Depth: 75'5" - 80'2"	5200	123.0	60.5	145.1	Jun 15/16

x_ METHOD ASTM D 2938
__ METHOD OTHER
__ MOISTURE CONDITIONED
__ DRY CONDITIONED
x_OTHER (As received)

Comments: The unconfined strength was determined in accordance with ASTM D2938-95 procedure with the cores in the as received moisture content.

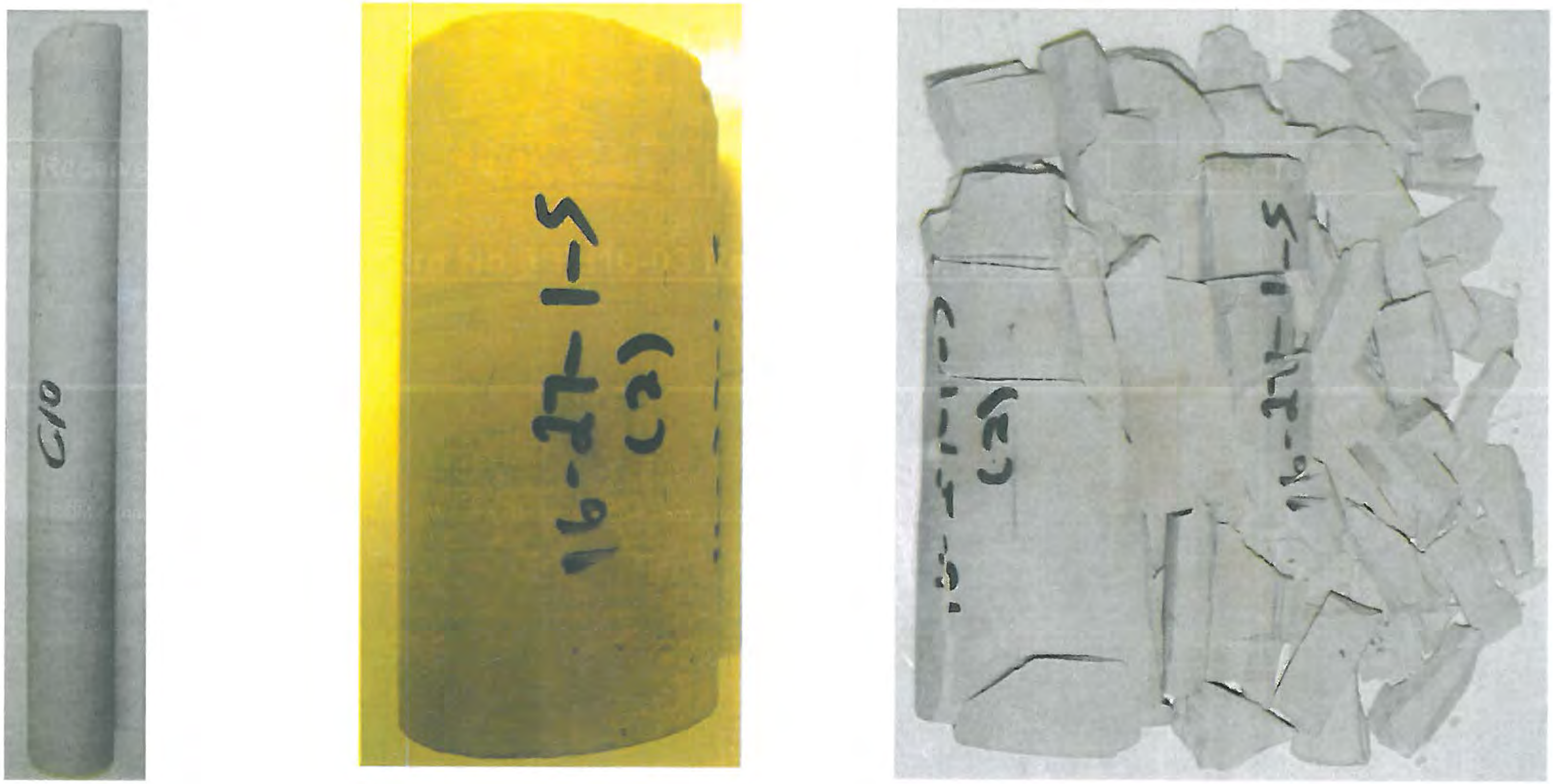
Enclosure; Photographs (2 pages)

Cc: Email: saba.ibrahim@aecom.com

ENG-TECH Consulting Limited

Per

Danny Holfeld, Principal
Ph: (204) 233-1694 Fx: (204) 235-1579



As Received

Before Test

After Test

Core No.5 TH16-03 Reference No. 16-027-1-5

As Received

Before Test

After Test

Core No.5 TH16-01 Reference No. 16-027-1-5