APPENDIX ‘A’

GEOTECHNICAL INVESTIGATION

Dillon Consulting Limited

Route 90 – Geotechnical Investigation

Prepared by:

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Project Number:

60282083 (402.19.2)

Date:

August 2013

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AECOM: 2012-01-06

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August 2, 2013

Mr. Mike Lau, P.Eng., Ph.D.
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Dear Mr. Lau:

Project No: **60282083 (402.19.2)**
Regarding: **Route 90 – Geotechnical Investigation**

AECOM Canada Ltd. is pleased to submit our final report regarding the above referenced project. If you have any questions regarding our submission, please do not hesitate to contact Zeyad Shukri of our office directly at 204-477-5381.

Sincerely,
AECOM Canada Ltd.

for Ron Typliski, P.Eng.
Vice-President, Environment
Manitoba/Saskatchewan District
Canada West Region

ZS:dh

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1	Z. Shukri	February 15, 2013	Draft
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1. Introduction

The City of Winnipeg has retained Dillon Consulting (Dillon) and AECOM Canada Ltd. (AECOM) to provide detailed design, including geotechnical engineering services, for the proposed Route 90 extension flyover structure. Construction of the east and west approach embankments was completed in July 2012 and September 2011, respectively.

This report documents the 2012 geotechnical investigation, identifies the geotechnical conditions that affect the design and construction of foundations for the proposed structure, and provides recommendations for detailed design of the geotechnical components including the foundation, and stability of the head slopes and side slopes of the approach embankments.

2. Available Information

Dillon and National Testing Laboratory (NTL) made available two existing geotechnical reports related to the subject site and existing embankments. A brief overview of the available information is summarized as follows:

- Construction of the west approach started in August and ended in September, 2011, at a final embankment height of 6.0 m. On average the side and head slopes are, 5 Horizontal (H):1 Vertical (V) and 4.6H:1V, respectively.

- Construction of the east approach started in June and ended in July, 2012, at a final embankment height of 6.8 m. Due to the Manitoba Hydro right of way, side slopes range from 4.3H:1V to 5.5H:1V. The head slope is maintained at 4H:1V.

Reported slope stability results show that a long term stability safety factor of 1.5 was satisfied. However, continuous monitoring and additional stability analysis was recommended to identify the bridge construction timeline and the final embankment configuration. The measured ground water level (GWL) was approximately 2 m below existing ground surface (elevation 232 m).

3. Geotechnical Investigation

3.1 Field Work

In the period from November 26 to December 01, 2012, AECOM completed a geotechnical investigation program, assisted by Paddock Drilling Ltd. The program consisted of a total of five (5) test holes. Three (3) deep test holes (TH12-02, TH12-03 and TH12-04) were drilled within the proximity of the proposed abutments: TH12-02 was drilled on the crest of the west embankment; TH12-03 was drilled at the proposed location of the center pier, and; TH12-04 at the toe of the east embankment head slope. The remaining two intermediate depth test holes, TH12-01 and TH12-05, were completed at the approximate locations of the proposed retaining walls at the east and west embankments, respectively. The test holes were drilled using 125 mm diameter solid-stem augers, and HQ coring was completed below auger refusal depths. The approximate locations of the test holes are shown on the Test Hole Location Plan in Appendix A.

Deep test holes (TH12-02 to TH12-04) were advanced at least 6 m into bedrock due to the poor quality of the bedrock. Standard penetration tests (SPT) were completed at selected depths. Disturbed and relatively undisturbed soil samples and rock cores were collected for further visual inspection and testing.

The intermediate depth test holes (TH12-01 and TH12-05) were advanced to approximately 11 m below existing grade at the toe of the side slopes of the east and west approach embankments. At each test hole location, disturbed samples from auger cuttings and SPT's and relatively undisturbed samples (Shelby tubes) were collected for further visual inspection and testing.

3.2 Laboratory Testing Program

Laboratory testing completed on selected samples included moisture content, unit weight, Atterberg limits, undrained shear strength, gradation, consolidation and uniaxial compressive strength test for rock cores. Test hole logs were prepared for each test hole to record the description and the relative position of the soil strata, location of samples obtained, field and laboratory test results, and other pertinent information. Uniaxial compressive strength tests on two rock cores show an average strength of 57 MPa. The test hole logs are attached in Appendix B.

3.3 Subsurface Conditions

In descending order the soil profile consists of:

- Clay Fill
- Glacio-lacustrine Clay
- Silt
- Glacial Till
- Limestone Bedrock

Each of these units is described further below. Profiles of selected soil properties and measured SPT N-values are presented on Figures 01 and 02.

Clay Fill

Clay fill was encountered at the surface of all test hole locations. Thicknesses of the clay fill ranged from 0.60 to 1.40 m in test holes TH12-01 and TH12-03 to TH12-05. Test hole TH12-02, which was drilled at the crest of the west embankment, encountered clay fill to a depth of approximately 5.5 m below grade. The top 0.45 m of clay fill was frozen at the time of investigation. Below the frozen zone, the clay fill was silty and contained trace amounts of organics and sand. Generally, the clay fill was brown to dark brown, stiff to very stiff, moist and of intermediate plasticity.

Glacio-Lacustrine Clay

The clay fill was underlain by galcio-lacustrine clay that was approximately 9.7 to 13.4 m in thickness. Generally, the clay contained some silt, was brown changing to grey and firm to stiff becoming soft with increasing depth, moist and of high plasticity.

Moisture content ranged from 24 to 64 percent. The average bulk unit weight of the clay was 16.3 kN/m^3. Undrained shear strength measured from unconfined compression tests ranged from 26 to approximately 37 kPa. The clay encountered within 2 m of ground surface in test holes TH12-02 and TH11-04 was relatively stiffer, denser and of lower moisture content than the clay encountered in other test holes. The slight difference in properties provides evidence of some consolidation under and within the vicinity of the embankment foot print.

Silt

A moist silt layer was encountered in each test hole below the clay fill or within the upper portion of the lacustrine clay. The thickness of the silt layer ranged from 0.15 to 1.10 m. Generally, the silt was light brown, firm, moist and of intermediate plasticity. Moisture content ranged from 21 to 39 percent.

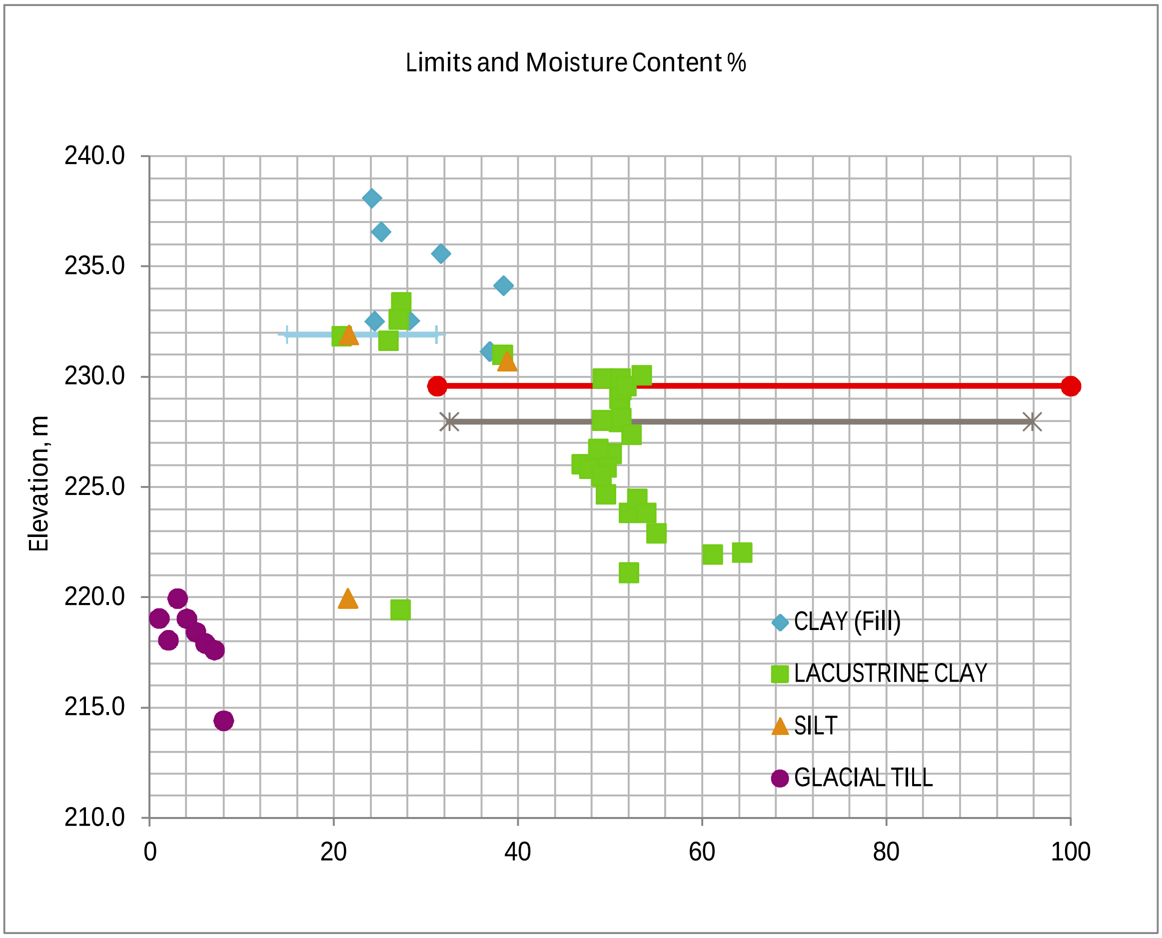
Glacial Till (Silt)

The clay was underlain by glacial till that typically contained variable amounts of sand and gravel. Boulders and cobbles are known to be present within the till unit and were encountered during the drilling. Where drilling advanced below the till unit, the thickness of the till layer varied from 5.0 to 6.25 m. The till was brown to light grey, soft/loose in the upper zone and became dense to very dense with increasing depth. Silt was observed on the surface of the till layer in test hole TH12-02 during drilling. Coring was necessary to advance the drilling through very dense and boulder/cobble dominated lower zone of the till. The till was moist to wet and of low plasticity. Measured moisture contents ranged from 4 to 21 percent.

Limestone Bedrock

The till was underlain by 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 the bedrock surface ranged from 18.6 and 19.8 m below existing grade (top of bedrock at an approximate elevation of 213.7 m). Based on the estimated Rock Quality Designation (RQD) values for the recovered rock cores, the rock quality encountered in test holes TH12-02, TH12-03 and TH12-04 was very poor to good quality. Uniaxial compressive strength tests completed on two competent rock cores recovered from test holes TH12-02 and TH12-04 indicated compressive strength in the range of 56 to 59 MPa. Photos of tested rock cores samples are shown along with the laboratory test results in Appendix C.

Figure 01: Figure 01: Profile for Selected Soil Test Results



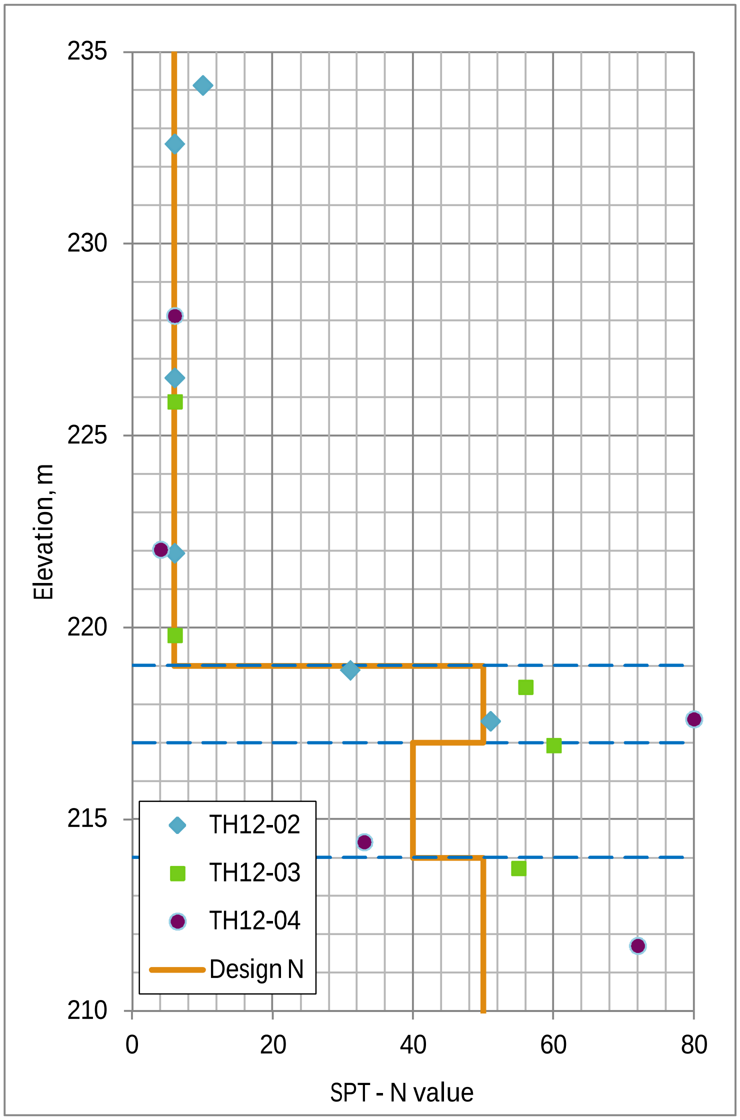


Figure 02: Profile for Measured SPT N Values

3.4 Groundwater Conditions

Seepage was observed during drilling in the till layer (approximately at El. 219 m) encountered in test holes TH12-02 through TH12-04. Due to the low permeability of the clay, seepage was not observed in the clay during drilling. Standpipe piezometers were not installed during the current investigation for groundwater monitoring. However, recent monitoring from the existing vibrating wire piezometers installed at the east and west embankment indicated that the groundwater water level ranged from El. 232.11 m to 233.34 m at the west embankment, and varied from El. 235.50 m to 241.33 m at the east embankment. Results from recent monitoring are presented in Table 01. The piezometer tip elevation level corresponds to the middle and lower portion of the clay strata. Fluctuations in the water table level are normal and will occur throughout the year depending upon variations in precipitation, evaporation, surface run-off, seasonal changes and other developments in this area.

Table 01: Summary of GWL Measurements

Piezometer Designation (by NTL)	Groundwater Level (GWL) Depth (Elevation), m		Piezometer Location
	January 18, 2013	February 20, 2013	
PZ-A1 (in Clay)	-1.71 (232.10)	-1.70 (232.11)	West Embankment
PZ-B1 (in Clay)	-0.58 (233.23)	-0.47 (233.34)	West Embankment
PZ-C1 (in Clay)	+3.86 (236.76)	+3.83 (236.73)	East Embankment
PZ-C2 (in Clay)	+2.20 (235.13)	+2.57 (235.50)	East Embankment
PZ-D1 (in Clay)	+8.40 (241.33)	+8.40 (241.33)	East Embankment
PZ-D2 (in Clay)	+3.00 (235.93)	+3.01 (235.94)	East Embankment

4. Foundations

4.1 Bridge 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 system alternatives include:

- Driven Pre-Cast Pre-Stressed Concrete Pile
- Driven Steel Piles
- Cast-In-Place Rock-Socketed Caissons

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

Driven PPC piles can be designed to support the heavy foundation loads of the proposed flyover. If used, pre-cast concrete piles should be driven to practical refusal into the very dense glacial till or onto the underlying bedrock. Provided that a hammer with a rated energy of at least 40 kJ per blow is used, the piles may be assigned the conventional capacities shown in Table 02. These traditional pile capacities are based on a series of studies and load tests that have been successfully used in the Winnipeg area for several decades.

Table 02: Allowable Pile Capacity Driven Pre-Cast Concrete 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 Table 02. In this regard, an embedment length ranging from 14 to 21 m below existing ground surface is estimated. 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 number of piles in the group multiplied by the allowable capacity per pile.

Pre-construction Wave Equation analysis and dynamic monitoring using a Pile Driving Analyzer (PDA) during construction should be used 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 pre-cast concrete 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 2.5 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. However, as a result of the identified groundwater conditions, the pre-bore should not be advanced below an elevation of 231 m. The diameter of the auger used to pre-bore should be a maximum of 50 mm larger than the pile diameter.

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. Piles that are damaged, excessively out of alignment or refuse prematurely may need to be replaced, pending a review to assess their load carrying capacity and any consequences of expected settlement on performance by the structural designer.

8. Where a steel follower is required to install piles below the surrounding ground surface, the refusal criteria should be increased by up to 50% in order to account for additional energy losses through the use of the follower or as determined from PDA monitoring.

9. The driving of all piles should be documented by experienced geotechnical personnel to confirm and record acceptable piling installation.

4.1.2 Driven Steel Piles

Driven steel H piles are considered to support bridge 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. The structural capacity of the pile can be determined from the steel sectional area and the maximum allowable stresses of $0.3f_y$. Practical refusal can be defined as 15 blows/25 mm penetration using a well maintained hammer with rated energy of not less than 50 kJ. For preliminary design purposes, it is anticipated that piles driven to elevation +214m would provide a sufficient capacity and fulfill driving criteria.

The actual refusal criteria and load capacity for the specific steel section and pile driving system should be established based on pre-construction Wave Equation analysis and PDA testing so that the geotechnical capacity can be confirmed and to protect against pile damage during installation.

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, if pile spacing is as indicated in the recommendations provided below. The design capacity of a pile group can be taken as the number of piles in the group multiplied by the allowable capacity per pile.

The following additional recommendations regarding steel piles are provided.

1. The minimum thickness of metal in the flange or web of the HP section should be 9.5 mm.

2. The weight of the embedded portion of the pile may be neglected in the design.

3. The pile cross-sections must be designed to withstand the design loads, handling stresses and driving forces during installation.

4. Piles should be fitted with an appropriate toe or shoe to protect the pile tip during installation.

5. Pile spacing should be a minimum of 3 pile diameters measured centre to centre.

6. 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.

7. The driving of all piles should be documented by experienced geotechnical personnel to confirm and record acceptable pile installation. It is recommended that the Geotechnical Engineer of Record be retained to perform foundation inspection services.

8. Any piles that are damaged, excessively out of alignment, or refuse prematurely may need to be replaced, pending a review of load carrying capacity by the Structural and Geotechnical Engineers of Record.

9. Subject to the Engineer approval, a pre-bore can be used to assist in pile installation. The pre-bore diameter should not exceed the pile size. Sloughing should be expected and the piles should be inserted into the bore immediately after the completion of drilling.

4.1.3 Pile Lateral Capacity

Battered piles can provide lateral resistance equal to the horizontal component of its axial load. Where practical, primary horizontal forces on pile foundations should be resisted by battered piles. Due to the lateral load imposed by the approach embankment at the head slope against the structural concrete box, a total horizontal force of 3,500 kN is anticipated to be resisted by the pile group.

4.1.4 Pile Downdrag

Negative skin friction in the magnitude of approximately 25 kPa over 15 m of the pile length should be expected, depending on the degree of consolidation at the time of installation.

4.1.5 Pile Settlements

In general, the settlement of a single pile will depend on a number of factors including load magnitude, strength-deformation properties of the foundation soils, load transfer mechanism, relative proportions of the loads carried by shaft friction and end bearing, and construction workmanship. In the case of end bearing piles, the full toe resistance is typically mobilized at pile displacements in the range of 1 to 2 percent of the pile toe diameter of driven piles. For the allowable end-bearing values given in Table 02, the estimated pile head settlement of a single end bearing pile may be assumed to be in the range of 1 to 2 percent of the pile toe diameter, not including elastic shortening due to the compressive load acting on the pile.

4.1.6 Cast-In-Place Rock-Socketed Caissons

Drilled caissons socketed into sound bedrock are considered to be a viable foundation system to support the proposed heavy structure. Local practice is to design the drilled shafts based on values of allowable end bearing and shaft adhesion of 3.0 and 1.0 MPa, respectively, provided that down hole inspection and assessment of the rock competency are undertaken. The assessment of the rock competency consists of probe drilling to 2 m below socket depth to detect the presence of voids or clay layers of any significance. In the event that the socket cannot be visually inspected, inspection of the recovered rock core and/or down hole 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 0.69 MPa with no contribution from end bearing. Safety concerns related to man entry into the boring (e.g., presence of soil gases) may preclude undertaking a visual inspection.

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

Based on the three test holes advanced into the bedrock (TH12-02 to TH12-04), the top 5 m of the bedrock is dominated by poor to fair quality rock. The thickness of the fractured bedrock is variable and could be in excess of 6 m.

Inspection of the recovered rock cores by qualified and experienced geotechnical personnel and down hole video inspection are required to aid in assessing 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 one socket diameter within sound, 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 locally available coring equipment. The rock sockets should not be spaced closer than 2.5 socket diameters, centre to centre. Tremie placement of concrete would likely be required.

The wet, granular till encountered below the glacio-lacustrine clay in test holes TH12-02 through TH12-04 may cave in during construction. As such, a temporary steel casing may be needed for proper caisson installation.

Should this type of foundation be 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.

4.2 Retaining Walls Foundation

Loads from retaining walls could range from light to heavy depending on the type and dimensions of the walls. Foundation requirements could be governed by lateral rather than axial resistance and/or construction aspects. Heavy loads from retaining walls can be supported using deep foundation elements including driven PPC and steel piles. The ease of installing battered driven piles makes these piles preferable for wall foundations. Related recommendations provided in Sections 4.1.1 and 4.1.2 can be used for wall application. Light and moderately loaded walls can be supported on shallow foundation or cast-in place friction piles.

4.2.1 Shallow Foundations

Shallow foundations can be used to support light to moderate loads and transfer and distribute the loads to the underlying soil at a pressure consistent with the requirements of the structure and the bearing capacity of the soil. The main issues with shallow foundation design at this site are the proximity to a Hydro right of way (particularly along the east approach embankment) and the requirements for protection against frost. Sufficient soil cover or insulation should be provided to protect against frost action. In this regard, shallow foundations should be located at a depth not shallower than the frost penetration depth of 2.5 m. This depth can be reduced if thermal insulation is used to protect against frost penetration provided the footing is bearing on competent soil.

The top of the native clay beneath the existing clay fill can be considered adequate bearing stratum to support shallow foundations provided the supported structures are designed to accommodate the expected settlement. An allowable bearing capacity of 85 kPa can be used for preliminary design purposes in this regard. The bearing capacity value will be influenced by the depth and width of the footing and the load inclination. Further details can be provided during the detailed design phase.

We understand that the road alignment has been changed to fulfill other requirements; therefore, part of the approach embankment will be shifted away from the existing embankment. Engineering fill should be placed at the new locations with proper compaction to avoid any differential settlement between the existing and the extended part of the embankment. New fill should be placed in maximum 300-mm loose lifts and compacted to a minimum of 98% of the Standard Proctor maximum dry density (SPMDD).

Different configurations of spread footings may result in a potential for load superposition and overstressing of the subsoil. Under these circumstances, reviewing the soil bearing capacity or modification to the footing configuration may be required so that settlement is within acceptable limits.

Ultimate unit resistance to sliding at the interface of the footing and the soil can be taken as the smaller of one half the normal stress at the interface or the clay cohesion value of 30 to 45 kPa. A minimum factor of safety of 1.5 should be applied against sliding.

The footing excavation can be backfilled using the excavated material. 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 bond breaker/thermal insulation between the footing and adjacent soil can be used to protect against adfreeze bond development.

Total and differential settlement magnitude and rate under spread footings can be estimated using one-dimensional consolidation theory. Footing 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.

5. Retaining Walls

The proposed project includes construction of walls to separate the Hydro tower right of way from the approach embankment on the east, to retain part of the east and west embankment side slopes at the toe and retain 35 m section of the east and west embankment side slopes to accommodate future road upgrades. Design considerations for walls supporting cuts and fills, and wall-specific design considerations 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. 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-composite products can be used behind other wall types to facilitate drainage. Retaining walls in excess of 1.5 m may also be equipped with weep holes to protect against buildup of hydrostatic pressure.

5.1 Wall Alternatives

The availability of construction space and the proximity to and potential impact on existing buildings and infrastructures are the governing factors that define the wall types in this project. Traditional gravity type walls (i.e., reinforced concrete and Mechanically Stabilized Earth (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 environments. 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, secant pile walls and slurry walls with/without tie backs depending on the wall design height. 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 well for staged construction and can be designed efficiently to reduce temporary shoring requirements.

Two options were considered in this project:

1. Two rows of sheet piles along the approach embankment
2. MSE wall with light weight material (Cematrix)

5.2 Lateral Earth Pressure

Lateral earth pressures transferred to bridge abutments or to retaining walls will be a function of backfill/retained material, method of placement and compaction of backfill, and amount of horizontal deflection allowed by abutment or walls after 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, active (K_a) and at-rest (Ko) earth pressure coefficients of 0.33 and 0.50, respectively, and a passive earth pressure coefficient of 3.0 can be used in the design of walls. However, if cohesive soils are being retained, active (K_a) and at-rest (Ko) earth pressure coefficients of 0.57 and 0.72, respectively, can be used in the design. A minimum factor of safety of 1.5 should be applied to the available passive resistance. A passive earth pressure coefficient of 1.75 can be used in the design of wall.

Compaction of backfill near the retaining wall within a distance equal to the top of the retaining wall to the wall base at the passive side should be conducted with 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.

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

5.3 Internal Stability

The final configuration of walls should be designed to satisfy design objectives related to bearing capacity, sliding, overturning and overall stability.

6. Embankments

The existing east and west embankments were constructed prior to the current investigation. The west embankment and foundation was explored for disturbed and relatively undisturbed samples. Visual examination and laboratory testing were conducted on the collected samples. Analysis was carried out to assess:

1. Consolidation settlement of the foundation soils
2. Slope stability

6.1 Consolidation Settlement

Settlement analysis was carried out to estimate the magnitude and rate of consolidation settlement of the foundation soil below the proposed embankment.

For modelling purposes, the lacustrine clay was divided into two layers (Layer I and II) to accommodate the variable soil stiffness. Layer I is 5 m thick, brown overconsolidated stiff clay. Layer II is normally consolidated, grey, soft to firm and extends to the glacial till surface. Based on laboratory testing and theoretical correlations, the consolidation parameters in Table 03 below were used for settlement analysis. According to site-specific measurements and observations, a GWL at 2 meters below ground surface was assumed for the calculation of consolidation settlement.

Table 03: Consolidation Parameters

Parameter	Value		Comment
	Layer I	Layer II	
Compression Index C_c	0.28	0.69	-
Recompression Index C_r	0.09	0.10	-
Coefficient of Vertical Consolidation C_v	0.90 m^2/yr		-
Coefficient of Horizontal Consolidation C_h	0.90 m^2/yr		Assumed $C_h = 1C_v$

Based on one-dimensional consolidation settlement analysis, ultimate settlement expected under the maximum embankment load (embankment height of 6.8 m) is approximately 600 mm. The time to achieve termination of primary consolidation (normally considered at 90 percent consolidation) is estimated to be in the order of 60 years.

Based on calculated results presented in Table 04 below, estimated settlement after one year of consolidation is 60 mm and estimated post-consolidation settlement is 90% x 600 – 60 = 480 mm occurring over a period of 60 years. Existing embankment elevations at the west approach embankment to date show an estimated total consolidation settlement of 70 mm. A summary of settlement analysis is shown in Table 04 below.

Table 04: Summary of Estimated Consolidation Settlement Analysis

	Time (yrs)	1	2	3	4	5	100
Estimated Consolidation Settlement (mm)	6.8m height embankment	60	80	112	160	175	595
Estimated Degree of Consolidation (%)		10	13	18	27	29	99
Estimated Rate of Settlement (mm/yr)		60	20	32	48	<15	<5

The potential for minimal differential settlement, (i.e. east and west embankments) from settlement occurring beneath the embankment fill cannot be completely eliminated. However, with the use of surcharge, such impacts are expected to be minimized. The potential for such movements is greatest where the pile-embankment interaction is in close proximity. While total and differential settlement cannot be quantified with reasonable accuracy by one-dimensional consolidation analysis, it is realistic to expect the settlement to be less than the estimated settlement for the embankment. Post-construction monitoring will provide information regarding the magnitude and trend of settlement.

A detailed graph showing the time rate settlement for the embankment is shown in Appendix E.

6.2 Slope Stability

An adequate factor of safety (FS) against slope instability must be achieved for head slope, side slopes and retained soil slope of the approach embankments, on both sides of the proposed bridge. In this regard, a design objective FS of 1.5 for long-term conditions and 1.3 for short-term conditions have been selected. These objectives are consistent with acceptable design practice in the Winnipeg area.

Stability analysis was completed to investigate the stability under two conditions:

- **Proposed Condition** – Final configuration was adopted with a maximum embankment height of 6.8 m. Two options were considered for this condition:
    - Option One, assuming sheet piles wall, and;
    - Option Two, adopting MSE wall with light weight material (Cematrix).

- **Future Condition** – Installation of retaining walls along the proposed future roadway was taken into account. Only sheet piles were considered for this condition.

For each condition, stability analysis for side slope, retained soil against the walls and head slope were completed to determine if additional design measures are required to attain the design objective factor of safety. Analyses for current GWL from recent monitoring and stabilized GWL were completed.

The soil strength properties used in the analysis are summarized in Table 05. These parameters were selected based on laboratory test results from collected samples and experience from similar projects. The parameters are within the range of locally accepted values. Stabilized GWL used in the analysis was at an elevation of 232.0, (i.e., 1 m below ground surface).

Table 05: Strength Parameters for Stability Assessment

Material	Total Unit Weight, (γ)	Cohesion, (C`)	Friction Angle, (Φ`)
	kN/m^3	kPa	degree
Clay Fill	18	5	18
Native Clay (Lacustrine)	16	5	16
Glacial Till	21	10	30
MSE Wall	21	100	45
Cematrix Material	6	100	45

6.2.1 Proposed Condition

6.2.1.1 Option One – Sheet Pile Wall

Based on the developed design concepts, an embedded wall will likely be required along the approach embankment and at the south side of the east approach embankment to separate the Hydro tower right of way and along a 35 m section of the existing east and west approach embankment side slopes. Table 06 displays wall locations with minimum required embedment depths:

Table 06: Proposed Sheet Pile Installation Minimum Embedment Depths and Design Parameters

Wall Location	Embedment Depth (m)*	Retained Soil	Retained Soil Height (m)
Close to Hydro tower right of way	7	Clay fill	2.0
Side slopes with two rows of sheet piles	10	Clay fill	6.8
Along future roadway	8	Clay fill	4.0

* Embedment depth extracted from stability analysis.

The results of the stability analysis are presented graphically in Appendix D and summarized in Table 07. The results indicate the following:

- Proposed new toe configuration with side slopes of 5H:1V satisfies the design objective FS of 1.5 for both the east and west approach embankments.
- Proposed concrete box abutment head slope for the east and west approach embankments satisfy the long-term design objective FS of 1.5.

- Proposed Hydro tower right of way with 6 m clearance from 5H:1V side slope and 2 m retained soil on the south side and 4H:1V on the north side for the east approach embankment satisfy the long-term design objective FS of 1.5 with a minimum wall embedment of 7 m.

- Side slopes with sheet pile retaining walls for both the east and west approach embankment satisfy the long-term design objective FS of 1.5.

Table 07: Summary of Proposed Configuration Slope Stability Analysis

	Description	Case	B-bar/ GWL	Critical FS	Design FS	File #	Figure #
East Approach	Side Slope @ 5H:1V	Existing PWP	B=0.60	***1.32***	1.30	A002-2	001
	Side Slope @ 5H:1V	Long-Term	232	***1.73***	1.50	A002-2	002
	Concrete Box Abutment @ Head	Existing PWP	B=0.60	***1.36***	1.30	B001-2	003
	Concrete Box Abutment @ Head	Long-Term	232	***1.49***	1.50	B001-2	004
	Hydro Tower - North Side Slope	Existing PWP	B=0.60	***1.45***	1.30	005-2	005
	Hydro Tower - North Side Slope	Long-Term	232	***1.88***	1.50	005-2	006
	Hydro Tower with Sheet Pile-South	Existing PWP	B=0.60	***1.32***	1.30	005-2	007
	Hydro Tower with Sheet Pile-South	Long-Term	232	***1.66***	1.50	005-2	008
	Side Slope with Sheet Pile Wall	Existing PWP	B=0.60	***1.34***	1.30	008-2	009
	Side Slope with Sheet Pile Wall	Long-Term	232	***1.58***	1.50	008-2	010
West Approach	Side Slope @ 5H:1V	Existing PWP	B=0.6	***1.32***	1.30	A004-2	011
	Side Slope @ 5H:1V	Long-Term	232	***1.73***	1.50	A004-2	012
	Concrete Box Abutment @ Head	Existing PWP	B=0.6	***1.36***	1.30	B018	013
	Concrete Box Abutment @ Head	Long-Term	232	***1.51***	1.50	B016	014
	Side Slope with sheet pile Wall	Existing PWP	B=0.6	***1.37***	1.30	006	015
	Side Slope with sheet pile Wall	Long-Term	232	***1.58***	1.50	006	016

Note: PWP denotes Pore Water Pressure

6.2.1.2 Option Two – MSE Wall

- Further to the above, stability analysis for MSE wall was completed to investigate the feasibility of using MSE wall instead of sheet piles. Long-term and short-term conditions were analyzed for selected configurations of side slopes, head slopes, and along the Hydro tower right of way. It was assumed that any granular material used as part of the MSE wall shall not be considered in the global stability analysis of the wall. Internal stability of the wall is the contractor’s responsibility, thus no analysis was carried out to check the internal stability.

- Stability analyses for MSE wall indicates that additional stabilization measures should be incorporated in the head slope and side slope design to achieve design objective FS for both short- and long-term scenarios. This stabilization measurement includes the use of Cematrix material as a light weight fill or equivalent.

- Table 08 displays the Cematrix profile that should be constructed for MSE wall. Head slope of the embankment fill against the MSE wall and abutment are designed as a vertical face with geogrid reinforcement for stability analysis. The base of wall should be embedded into the ground up to 0.6 m below ground level. However, the top 0.6 m of soil below the MSE wall should be replaced a minimum distance of 15 m away from the edge of the abutment to minimize the differential settlement due to pile-embankment interaction in close proximity. For modelling purposes, the maximum width of the MSE wall along the hydro lines was assumed to be 0.70 x Maximum height of embankment.

Table 08: Proposed Profile for Light Weight Material (Cematrix)

Embankment Height		Cematrix Thickness (m)
From (m)	**To (m)**	
6.5	7.0	3.0
6.0	6.5	2.0
5.5	6.0	1.5
5.0	5.5	1.0

The results of the stability analysis are presented graphically in Appendix D and summarized in Table 09.

Table 09: Summary of Proposed Configuration Slope Stability Analysis

	Description	Case	B-bar/ GWL	Critical FS	Design FS	File #	Figure #
East Approach	Hydro Tower with MSE Wall-South	Existing PWP	B=0.50	***1.30***	1.30	D10	019
	Hydro Tower with MSE Wall-South	Long-Term	232	***1.61***	1.50	D09	020
	Side Slope with MSE Wall*	Existing PWP	B=0.60	***1.29***	1.30	D04	-
	Side Slope with MSE Wall*	Long-Term	232	***1.69***	1.50	D03	-
	Head Slope with MSE Wall*	Existing PWP	B=0.60	***1.89***	1.30	D06	021
	Head Slope with MSE Wall*	Long-Term	231	***2.27***	1.50	D05	022
West Approach	Side Slope with MSE Wall*	Existing PWP	B=0.60	***1.29***	1.30	D04	023
	Side Slope with MSE Wall*	Long-Term	232	***1.69***	1.50	D03	024
	Head Slope with MSE Wall*	Existing PWP	B=0.60	***1.87***	1.30	D02	025
	Head Slope with MSE Wall*	Long-Term	231	***2.23***	1.50	D01	026

6.2.2 Future Condition

Cut retaining walls on the north side of the east approach embankment and south side of the west approach embankment satisfy the long-term design objective FS of 1.5 with a minimum wall embedment of 8 m. The results of the stability analysis are presented graphically in Appendix D and summarized in Table 10.

Table 10: Summary of Future Slope Stability Analysis

	Description	Case	B-bar/ GWL	Critical FS	Design FS	File #	Figure #
East Approach	North Side Slope Wall @ 5H:1V	Long-Term	232	***1.51***	1.50	C002	017
West Approach	South Side Slope Wall @ 5H:1V	Long-Term	232	***1.50***	1.50	C002	018

Due to the current elevated GWL on the east approach embankment, it is recommended that GWL monitoring be continued. Construction activities on the east side should be subject to the results of the GWL monitoring results. Additional analysis should be completed during the detailed design phase to assess the stability of the approach embankment, considering the pile installation interaction and the future MSE retaining wall on the east and west approach embankment.

7. Closure

The findings and recommendations of this report were based on the results of field and laboratory investigations, combined with an interpolation of soil and ground water conditions between the test hole locations. If conditions are encountered that appears to be from those shown by the test hole drilled at this site and described in this report, or if assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendation can be reviewed and justified, 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 possibility of variation in soil conditions, which may result in modifications of the design and construction procedures.

Appendix A

Test Hole Location Plan

Last saved by: MAHEC(2012-12-12) Last Plotted: 2012-12-12
Filename: P:\60282083\000-CAD\02-SHEETS\B\01\60282083-FIG-00-0000-ROUTE90EXTENSION.DWG

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

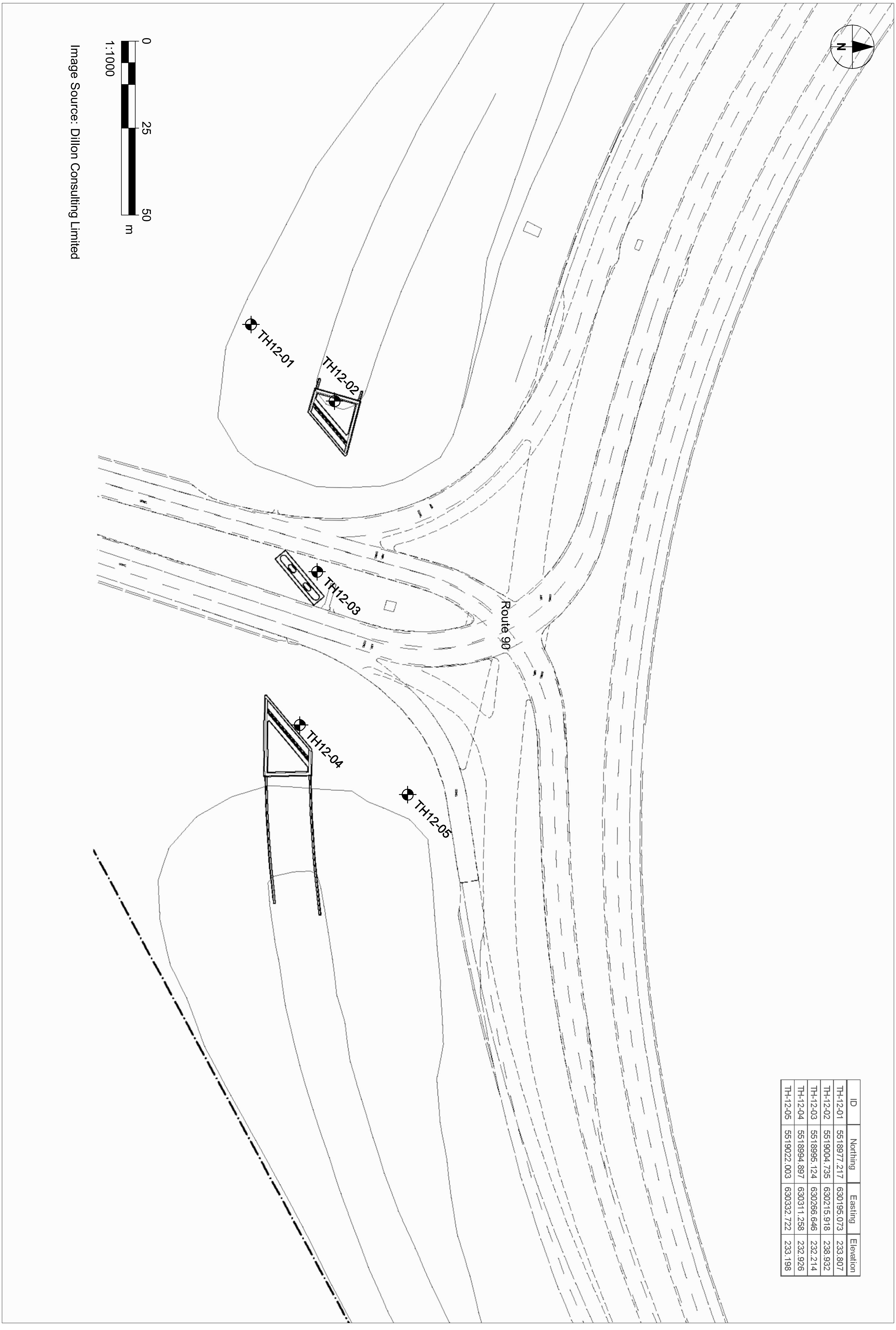


Image Source: Dillon Consulting Limited

ID	Northing	Easting	Elevation
TH-12-01	5518977.217	630195.073	233.807
TH-12-02	5519004.735	630215.918	238.932
TH-12-03	5518995.124	630266.646	232.214
TH-12-04	5518994.897	630311.258	232.926
TH-12-05	5519022.003	630332.722	233.198

Route 90 Extension

Dillon Consulting Limited, Route 90

Test Hole Location Plan

AECOM

Figure: 01

Appendix B

Test Hole Logs

AECOM Canada Ltd.

GENERAL STATEMENT

NORMAL VARIABILITY OF SUBSURFACE CONDITIONS

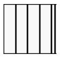
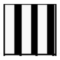
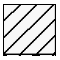
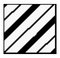
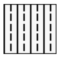
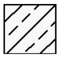
The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability for the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. Our description of the 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.

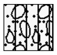
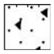
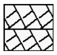
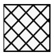
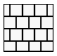
The analysis and recommendations presented in this report are based on the data obtained from the borings and test pit excavations made at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere 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.

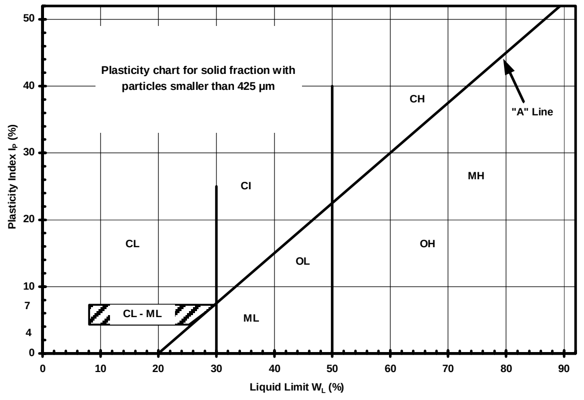
In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced 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				AECOM 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			AECOM
	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

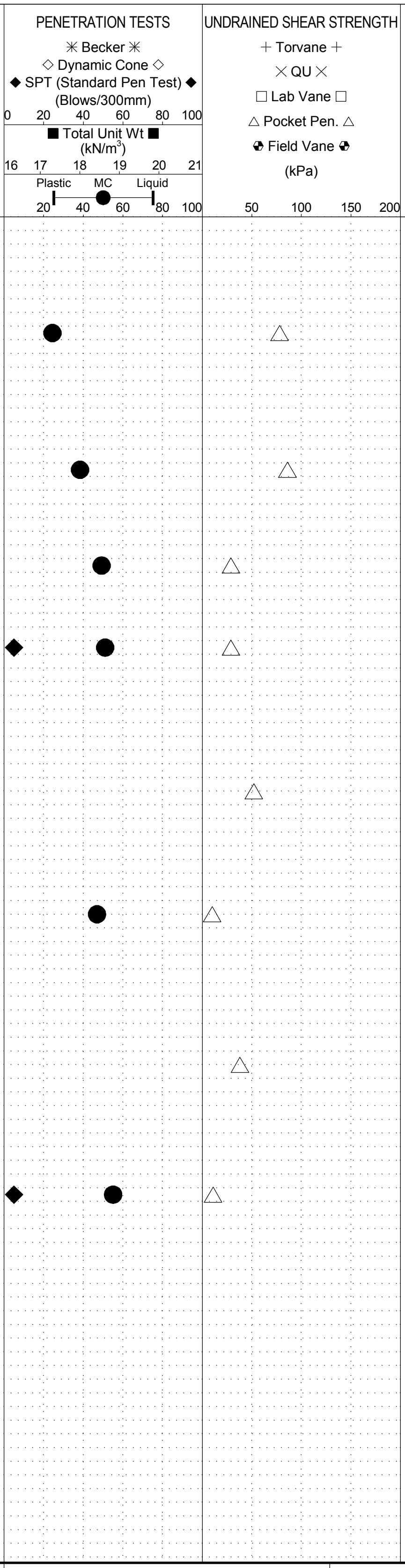
The resistance (N) of a non-cohesive soil can be related to compactness condition as follows

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

PROJECT: Route 90 Extension
CLIENT: Dillon Consulting Ltd.
TESTHOLE NO: **TH12-01**
LOCATION: West Embankment - Side Slope Toe (N: 5518977, E: 630195)
PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.
METHOD: Mobile B-59, 125 mm SSA
ELEVATION (m): 233.81

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	▩	CLAY (Fill) - silty, trace organics, trace sand - brown to dark brown, moist, stiff - intermediate plasticity					
1	▨	CLAY - some silt - brown, moist, stiff - high plasticity	■	G58			233
2	▥	SILT - some clay - light brown, moist, firm - low plasticity					232
3	▨	CLAY - some silt, trace gravel - brown, moist, stiff - high plasticity, silt inclusions	■	G59			231
4	▨		■	G60			230
5	▨	- greyish brown, trace oxidation below 4.57 m	☒	S61	5	- 2,2,3 blows/150 mm - SPT Recovery: 100%	229
6	▨	- grey, firm to soft	▥	T62		- Tube Recovery: 100%	228 227
7	▨	- soft, silt lens (up to 25 mm thick)	■	G63			226
8	▨						225
9	▨		▥	T64		- Tube Recovery: 100%	224
10	▨						
11	▨		☒	S65	5	- 1,2,3 blows/150 mm - SPT Recovery: 100%	223
12		END OF TEST HOLE AT 11.13 m BGS IN CLAY Notes: 1. No seepage or sloughing observed. 2. Test hole backfilled with auger cuttings and sealed with bentonite chips upon completion. 3. BGS - "below ground surface".					222
13							221
14							220
15							219



AECOM

LOGGED BY: Samuel O.
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Zeyad Shukri
COMPLETION DEPTH: 11.13 m
COMPLETION DATE: 11/29/12

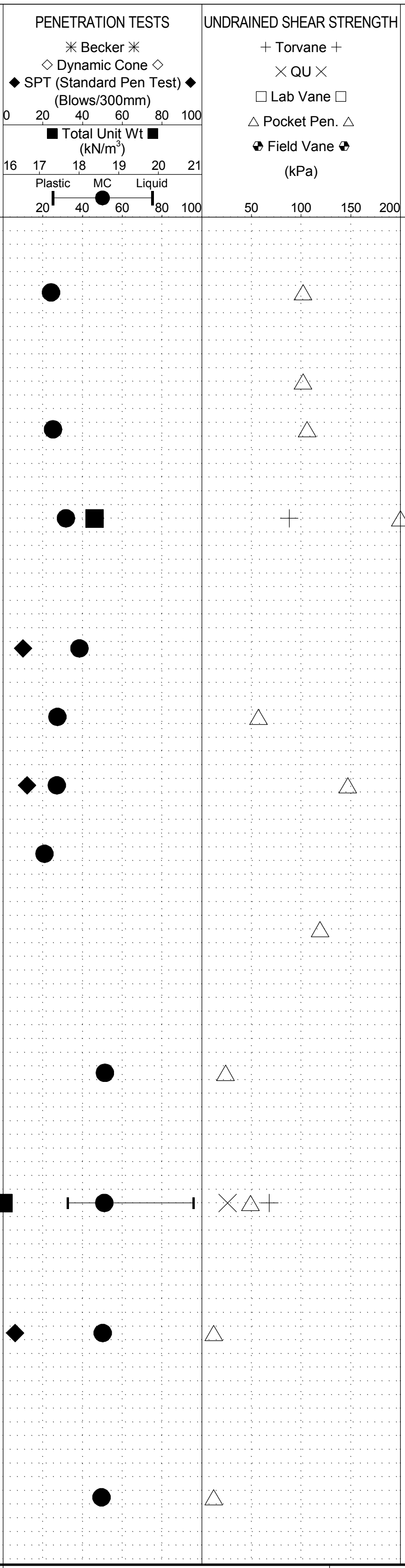
PROJECT: Route 90 Extension
CLIENT: Dillon Consulting Ltd.
TESTHOLE NO: **TH12-02**
LOCATION: West embankment crest (N: 5519004, E: 630215)
PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.
METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA
ELEVATION (m): 238.93

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

DEPTH (m)	SOIL DESCRIPTION
0	CLAY (Fill) - silty, trace organics, trace sand - brown to dark brown, moist, stiff - intermediate plasticity
5.5	CLAY - some silt - brown, moist, stiff to very stiff - high plasticity, laminated - silt inclusions
6.85	SILT - some sand, some clay - light brown, moist, stiff - intermediate plasticity
7.6	CLAY - some silt - brown, moist, stiff to very stiff - high plasticity, laminated
9.45	- greyish brown below 9.45 m - silt lens (up to 13 mm thick) - trace oxidation
10.36	- grey, firm to soft below 10.36 m
12.19	- silt and sand pockets up to 25 mm thick below 12.19 m

SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
■	G23		
▥	T24		- Tube Recovery: 100%
■	G25		
▥	T26		- Gravel: 0.0%, Sand: 2.4%, Silt: 29.1%, Clay: 68.5% - Bulk Density: 18.3 kN/m^3 - Tube Recovery: 100%
☒	S27	10	- 3,5,5 blows/ 150 mm - SPT Recovery: 100%
■	G28		
☒	S29	12	- 5,5,7 blows/ 150 mm - SPT Recovery: 83%
■	G30		- Gravel: 0.0%, Sand: 19.0%, Silt: 62.4%, Clay: 18.6%
▥	T31		- Tube Recovery: 96%
■	G32		
▥	T33A		- Gravel: 0.0%, Sand: 0.0%, Silt: 13.9%, Clay: 86.1% - LL: 96%, PL: 33%, PI: 63%, Bulk Density: 16 kN/m^3 - Tube Recovery: 100%
☒	S33B	6	- 2,3,3 blows/ 150 mm - SPT Recovery: 100%
■	G33C		

DEPTH (m)	ELEVATION
1	238
2	237
3	236
4	235
5	234
6	233
7	232
8	231
9	230
10	229
11	228
12	227
13	226
14	225
15	

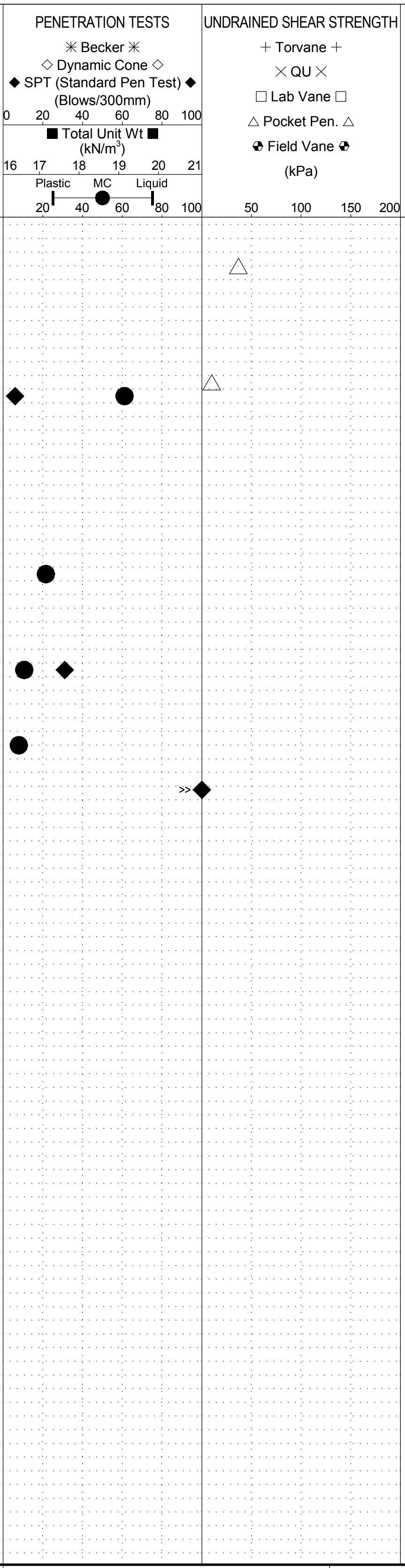


AECOM

LOGGED BY: Samuel O.
REVIEWED BY: Zeyad Shukri
PROJECT ENGINEER: Zeyad Shukri
COMPLETION DEPTH: 35.05 m
COMPLETION DATE: 12/1/12

PROJECT: Route 90 Extension	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH12-02**
LOCATION: West embankment crest (N: 5519004, E: 630215)		PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA	ELEVATION (m): 238.93

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



DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
15						
		▥	T34		- Tube Recovery: 100%	
16						223
17		☒	S35	6	- 1,2,4 blows/ 150 mm - SPT Recovery: 100%	222
18						221
19	SILT - sandy, some clay, trace gravel - grey, moist to wet, firm - low plasticity	■	G36			220
	TILL - silty, some sand, some clay, trace to some gravel - greyish brown, moist, dense to very dense					
20		☒	S37	31	- 9,14,17 blows/ 150 mm - SPT Recovery: 100%	219
	- sand seam (0.60 m thick) - brown, wet, loose	■	G38			
21						218
	- power auger refusal at 21.34 m bgs	☒	S39	51/ 76mm	- 51 blows/ 76 mm	
22		▌	C68		- Core Recovery: 44%	217
23						216
		▌	C69		- Core Recovery: 96%	
24						215
25	- grey, cobbles to boulder below 25.0 m	▌	C70		- C70 RQD: 33% - Core Recovery: 52%	214
	LIMESTONE (Bedrock) - light grey, core angle: 90 degrees - fine grained, no foliation - close spacing, unaltered faces, rough undulating joints - R3, medium strong - fossiliferous, evidence of water flow - vuggy - oxidized, R2, weak to 27.4 m					
26						213
27		▌	C71		- C71 RQD: 22% - Core Recovery: 79%	212
	- altered yellow, gapped fractures (180 degrees to core axis) below 27.4 m - close spacing, smooth undulating to smooth planar fractures					
28		▌	C72		- C72 RQD: 24% - Core Recovery: 98%	211
29	- grey, R3, medium strong - evidence of water flow - white, laminated below 29.3 m					210
30		▌	C73		- C73 RQD: 8%	

LOGGED BY: Samuel O.	COMPLETION DEPTH: 35.05 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 12/1/12
PROJECT ENGINEER: Zeyad Shukri	Page 2 of 3

PROJECT: Route 90 Extension	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH12-02**
LOCATION: West embankment crest (N: 5519004, E: 630215)		PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA	ELEVATION (m): 238.93

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

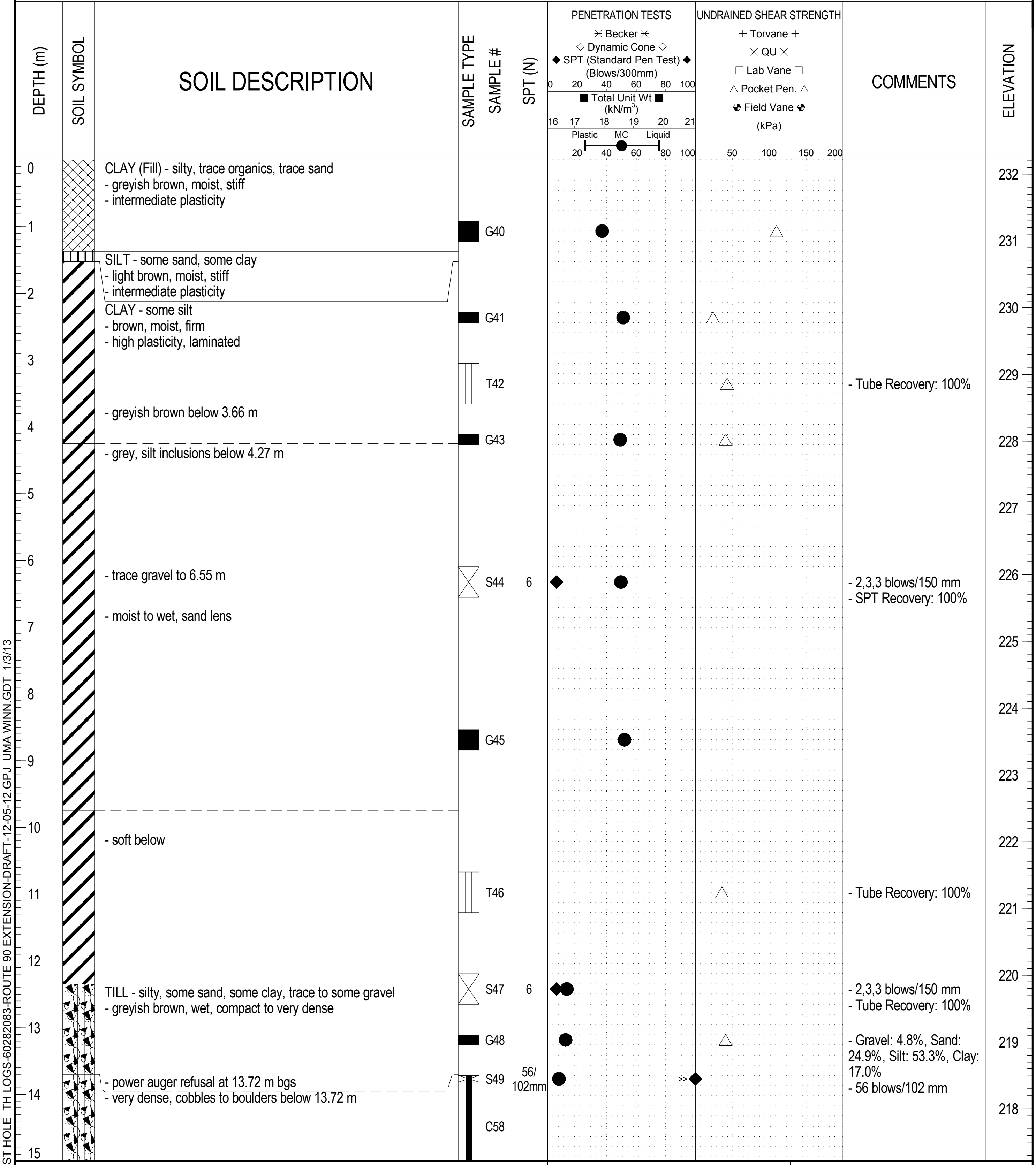
DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU × ☐ Lab Vane ☐ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
30	▨		▌					- Core Recovery: 59%	
31	▨		▌	C74				- C74 RQD: 52% - Core Recovery: 90%	208
32	▨		▌						207
33	▨		▌	C75				- C75 RQD: 65% - Core Recovery: 100%	206
34	▨		▌	C76				- C76 RQD: 79% - Core Recovery: 100%	205
35		END OF TEST HOLE AT 35.05 m BGS IN BEDROCK Notes: 1. Power auger refusal at 21.34 m below ground surface in TILL. 2. HQ coring below 21.34 m. 3. Seepage observed at 20.42 m below ground surface. 4. Test hole grouted up to 0.31 m and sealed with bentonite chips to ground surface. 5. BGS - "below ground surface".							204
36									203
37									202
38									201
39									200
40									199
41									198
42									197
43									196
44									195
45									

AECOM	LOGGED BY: Samuel O.	COMPLETION DEPTH: 35.05 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 12/1/12
	PROJECT ENGINEER: Zeyad Shukri	Page 3 of 3

PROJECT: Route 90 Extension	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH12-03**
LOCATION: Center Pier (N: 5518995, E: 630266)		PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA	ELEVATION (m): 232.21

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

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
CLAY (Fill) - silty, trace organics, trace sand - greyish brown, moist, stiff - intermediate plasticity	■	G40		
SILT - some sand, some clay - light brown, moist, stiff - intermediate plasticity				
CLAY - some silt - brown, moist, firm - high plasticity, laminated	■	G41		
	▥	T42		- Tube Recovery: 100%
- greyish brown below 3.66 m	■	G43		
- grey, silt inclusions below 4.27 m				
- trace gravel to 6.55 m	☒	S44	6	- 2,3,3 blows/150 mm - SPT Recovery: 100%
- moist to wet, sand lens	■	G45		
- soft below	▥	T46		- Tube Recovery: 100%
TILL - silty, some sand, some clay, trace to some gravel - greyish brown, wet, compact to very dense	☒	S47	6	- 2,3,3 blows/150 mm - Tube Recovery: 100%
	■	G48		- Gravel: 4.8%, Sand: 24.9%, Silt: 53.3%, Clay: 17.0%
- power auger refusal at 13.72 m bgs - very dense, cobbles to boulders below 13.72 m	☒	S49	56/ 102mm	- 56 blows/102 mm
	▌	C58		



AECOM	LOGGED BY: Samuel O.	COMPLETION DEPTH: 25.91 m
	REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 11/30/12
	PROJECT ENGINEER: Zeyad Shukri	Page 1 of 2

PROJECT: Route 90 Extension | CLIENT: Dillon Consulting Ltd. | TESTHOLE NO: **TH12-03**

LOCATION: Center Pier (N: 5518995, E: 630266) | PROJECT NO.: 60282083

CONTRACTOR: Paddock Drilling Ltd. | METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA | ELEVATION (m): 232.21

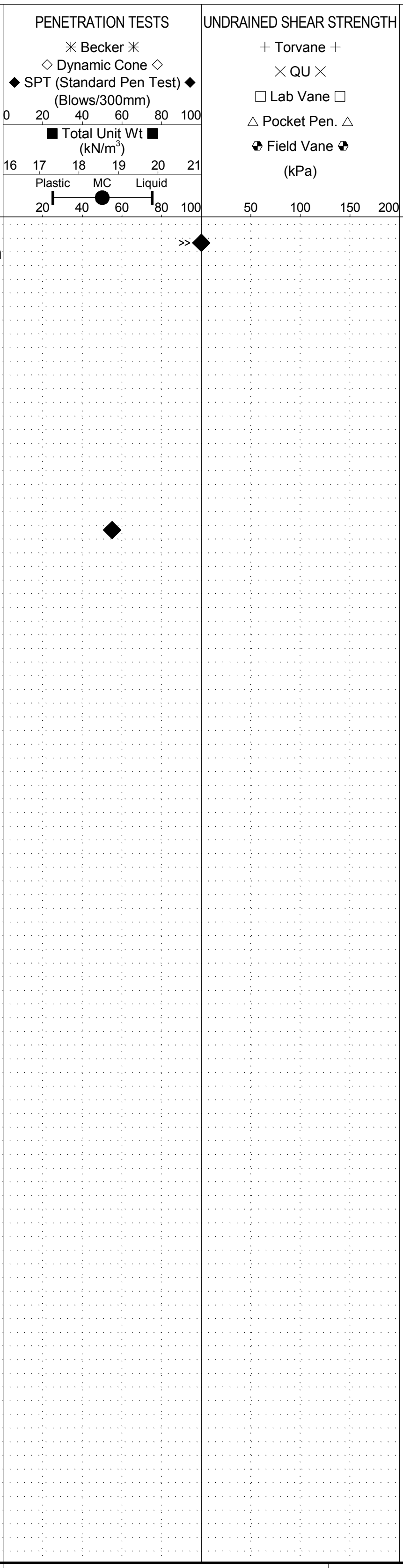
SAMPLE TYPE: ■ GRAB | ▥ SHELBY TUBE | ☒ SPLIT SPOON | ☰ BULK | ◿ NO RECOVERY | ▌ CORE

SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
- dense to very dense	S59	60/ 102mm	- 60 blows/102 mm
	C60		- Core Recovery: 88%
	C61		- Core Recovery: 21%
- sand seam (102 mm thick)	S62	55	- SPT Recovery: 72% - 3,4,51 blows/150 mm
LIMESTONE (Bedrock) - light grey, pockets of softer yellow, core angle: 90 degrees - fine grained, no foliation, vuggy - close spacing, slightly altered faces, rough undulating joints - R3, medium strong - yellowish grey below 19.20 m	C63		- C63 RQD: 28% - Core Recovery: 86%
	C64		- C64 RQD: 56% - Core Recovery: 87%
- oxidized, R2, weak to 21.64 m			
- white, laminated below 21.95 m - gapped fractures (180 degrees to core axis) below 22.10 m - close spacing, smooth undulating to smooth planar fractures - unaltered faces, R3, medium strong	C65		- C65 RQD: 7% - Core Recovery: 92%
	C66		- C66 RQD: 41% - Core Recovery: 87%
- evidence of water flow	C67		- C67 RQD: 20% - Core Recovery: 17%

DEPTH (m): 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

ELEVATION: 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206, 205, 204, 203

END OF TEST HOLE AT 25.91 m IN BEDROCK
Notes:
1. Power auger refusal at 13.72 m below ground surface in TILL.
2. HQ coring below 13.72 m.
3. Seepage observed at 12.34 m below ground surface.
4. Test hole grouted up to 0.31 m and sealed with bentonite chips to ground surface.
5. BGS - "below ground surface".



AECOM

LOGGED BY: Samuel O. | COMPLETION DEPTH: 25.91 m

REVIEWED BY: Zeyad Shukri | COMPLETION DATE: 11/30/12

PROJECT ENGINEER: Zeyad Shukri

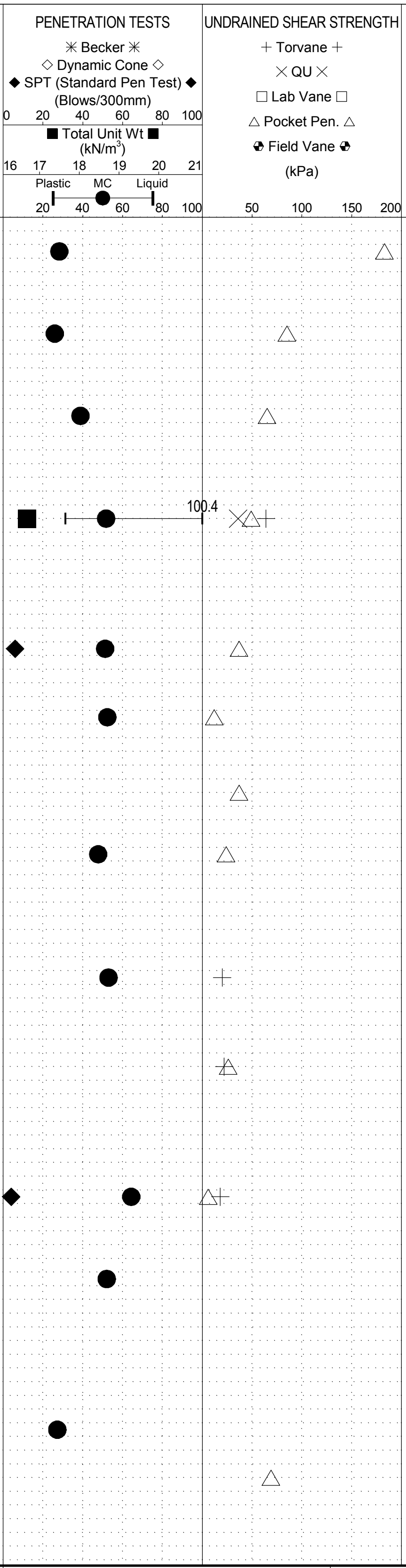
PROJECT: Route 90 Extension	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH12-04**
LOCATION: East Abutment (N: 5518994, E: 630311)		PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA	ELEVATION (m): 232.93

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

DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

ELEVATION: 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
CLAY (Fill) - silty, trace sand, trace organic - brown to dark brown, moist, stiff to very stiff - intermediate plasticity	■	G1		
CLAY - trace silt - brown to dark brown, moist, stiff - high plasticity, laminated	■	G2		
- silt inclusions, trace organics	■	G3		
SILT - some sand, some silt - light brown, moist, stiff - intermediate plasticity				- Gravel: 0.0%, Sand: 0.0%, Silt: 14.6%, Clay: 85.4%
CLAY - trace silt - brown to dark brown, moist, stiff - high plasticity, laminated - firm below 2.44 m	▥	T4		- LL: 100%, PL: 31%, PI: 69%, Bulk Density: 16.6 kN/m^3 - Tube Recovery: 100%
- greyish brown below 4.57 m	☒	S5	6	- 2,2,4 blows/150 mm - SPT Recovery: 100%
- grey, firm to soft, trace oxidation below 5.49 m	■	G6		
	▥	T7		- Tube Recovery: 100%
	■	G8		
- soft below	■	G9		
	▥	T10		- Tube Recovery: 100%
- silt lens (up to 19 mm thick)	☒	S11	4	- 1,2,2 blows/150 mm - SPT Recovery: 100%
	■	G12		
- trace sand, trace gravel	■	G13		
	▥	T14		- Tube Recovery: 100%
TILL - sandy, some gravel, trace silt - brown to grey, wet, compact				



LOGGED BY: Samuel O.	COMPLETION DEPTH: 27.43 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 11/29/12
PROJECT ENGINEER: Zeyad Shukri	Page 1 of 2

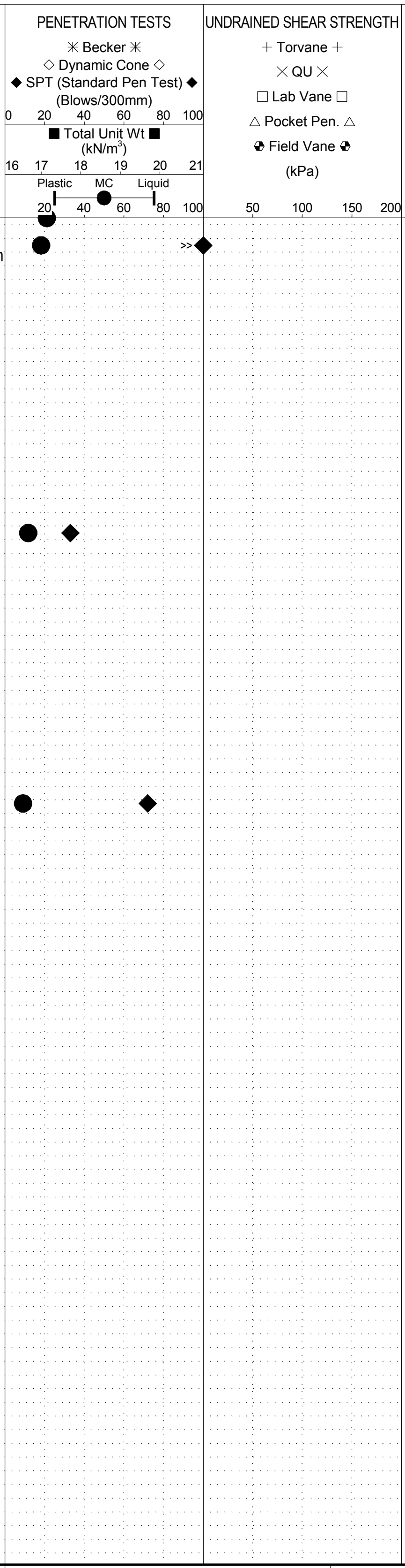
PROJECT: Route 90 Extension | CLIENT: Dillon Consulting Ltd. | TESTHOLE NO: **TH12-04**

LOCATION: East Abutment (N: 5518994, E: 630311) | PROJECT NO.: 60282083

CONTRACTOR: Paddock Drilling Ltd. | METHOD: Mobile B-59 / Acker SS-3, 125 mm SSA | ELEVATION (m): 232.93

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

DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
15		G15			
	- power auger refusal at 15.39 m bgs	S16	80/ 152mm	- 80 blows/150mm	
	- grey, cobbles to boulder below 15.39 m				
	- dense to very dense				217
16		C17		- Core Recovery: 41%	
17					216
		C18		- Core Recovery: 44%	
18					215
		S19A	33	- 23,20,13 blows/150 mm - SPT Recovery: 83%	
19		C19B		- C19B RQD: 13% - Core Recovery: 51%	214
	LIMESTONE (Bedrock) - yellowish grey, pockets of softer yellow, core angle: 90 degrees - fine grained, no foliation				
20	- close spacing, slightly altered faces, rough undulating joints - R3, medium strong				213
	- vuggy	C19C		- Core Recovery: 0%	
21					212
		S19D	72	- 15,21,51 blows/150 mm - SPT Recovery: 47%	
22	- oxidized, R2, weak to 22.25 m	C20		- C20 RQD: 12%	211
	- laminated, evidence of water flow below 22.25 m			- Core Recovery: 79%	
23	- white, R3, medium strong - gapped fractures (180 degrees to core axis) below 22.86 m - close spacing, unaltered, smooth planar faces				210
		C21		- C21 RQD: 41% - Core Recovery: 85%	
24					209
25					208
		C22		- C22 RQD: 26% - Core Recovery: 100%	
26					207
		C23		- C23 RQD: 82%	
27					206
	END OF TEST HOLE AT 27.43 m IN BEDROCK Notes: 1. Power auger refusal at 15.39 m below ground surface in TILL. 2. HQ coring below 15.39 m. 3. Zero percent core recovery from 19.81 to 21.34 m below ground surface. 4. Seepage observed at 14.33 m below ground surface. 5. Test hole grouted up to 0.31 m and sealed with bentonite chips to ground surface. 6. BGS - "below ground surface".				
28					205
29					204
30					



LOGGED BY: Samuel O. | COMPLETION DEPTH: 27.43 m

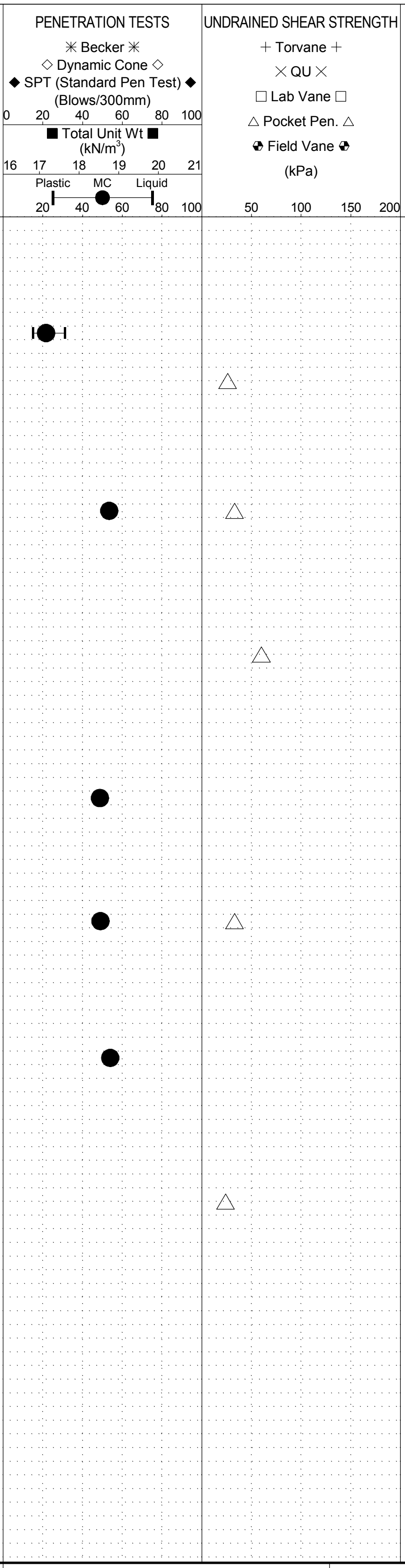
REVIEWED BY: Zeyad Shukri | COMPLETION DATE: 11/29/12

PROJECT ENGINEER: Zeyad Shukri

PROJECT: Route 90 Extension	CLIENT: Dillon Consulting Ltd.	TESTHOLE NO: **TH12-05**
LOCATION: East embankment side slope toe (N: 5519022, E: 630332)		PROJECT NO.: 60282083
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Mobile B-59, 125 mm SSA	ELEVATION (m): 233.20

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

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
0	CLAY (Fill) - silty, trace organic, trace sand - brown, moist, stiff - intermediate plasticity				
1	CLAY - trace silt - brown, moist, firm - high plasticity	GRAB	G50		- Gravel: 0.1%, Sand: 13.0%, Silt: 59.2%, Clay: 27.7%
	SILT - light brown, moist, firm to stiff - intermediate plasticity	SHELBY TUBE	T51		- Tube Recovery: 100%
	CLAY - brown, moist, stiff - high plasticity				
	- greyish brown below 3.35 m	SPLIT SPOON	S52		- 2,2,3 blows/150 mm - SPT Recovery: 100%
	- sand seam (102 mm thick) - silt inclusions to 9.14 m				
		SHELBY TUBE	T53		- Tube Recovery: 100%
	- grey, firm to soft				
		GRAB	G54		
		SPLIT SPOON	S55		- 2,2,3 blows/150 mm - SPT Recovery: 100%
	- silt lens up to 13 mm thick	GRAB	G56		
		SHELBY TUBE	T57		- Tube Recovery: 100%
11.28	END OF TEST HOLE AT 11.28 m IN CLAY Notes: 1. No seepage or sloughing observed. 2. Test hole backfilled with auger cuttings and sealed with bentonite chips upon completion.				



DEPTH (m)	ELEVATION
0	233
1	232
2	231
3	230
4	229
5	228
6	227
7	226
8	225
9	224
10	223
11	222
12	221
13	220
14	219
15	

AECOM

LOGGED BY: Samuel O.	COMPLETION DEPTH: 11.28 m
REVIEWED BY: Zeyad Shukri	COMPLETION DATE: 11/29/12
PROJECT ENGINEER: Zeyad Shukri	Page 1 of 1

Appendix C

Laboratory Test Results

AECOM

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

204 477 5381 tel
204 284 2040 fax

Memorandum

To	Sam Oshati	Page	1
CC			
Subject	Route 90 Extension		
From	Stephen Petsche		
Date	December 10, 2012	60282083	

Attached are testing results for the above noted project. The testing included forty-six (46) Moisture Content tests, three (3) Atterberg Limits tests and six (6) Grain Size Distribution (hydrometer method) tests on samples submitted to the lab. The testing also included Torvane, Pocket Penetrometer, Unconfined Compressive Strength, Moisture Content, Bulk Density and Visual Description on three (3) shelby tube samples. The additional Oedometer consolidation tests will be reported upon completion.

If you have any questions, please call.

Sincerely,

Stephen Petsche, C.E.T.
Coordinator, Lab and Technical Services

Attach.

MOISTURE CONTENT

JOB No.: 60282083 DATE: December 4, 2012
CLIENT: Dillon Consulting
PROJECT: Route 90 Extension

HOLE NO.	TH12-04	-	-	-	-	-
SAMPLE NO.	G1	G2	G3	S5	G6	G8
DEPTH (FT)	1.0 - 1.5	4.0 - 4.5	7.0 - 7.5	15.0 - 16.5	18.0 - 18.5	23.0 - 23.5
MOISTURE CONTENT %	28.2	25.9	38.8	51.2	52.3	47.7
HOLE NO.	TH12-04	-	-	-	-	-
SAMPLE NO.	G9	S11	G12	G13	G15	S16
DEPTH (FT)	27.5 - 28.0	35.0 - 36.5	38.5 - 39.0	44.0 - 44.5	49.0 - 49.5	50.0 - 50.5
MOISTURE CONTENT %	52.9	64.3	52.0	27.2	21.1	18.2
HOLE NO.	TH12-04	-	TH12-02	-	-	-
SAMPLE NO.	S20	S22	G23	G25	S27	G28
DEPTH (FT)	60.0 - 61.5	69.0 - 70.5	2.5 - 3.0	7.5 - 8.0	15.0 - 16.5	18.0 - 18.5
MOISTURE CONTENT %	11.7	9.1	24.1	25.1	38.4	27.3
HOLE NO.	TH12-02	-	-	-	-	-
SAMPLE NO.	S29	G30	G32	G33C	S33B	G35
DEPTH (FT)	20.0 - 21.5	23.0 - 23.5	31.0 - 31.5	46.5 - 47.0	40.0 - 41.5	55.0 - 56.5
MOISTURE CONTENT %	27.0	20.8	51.2	49.5	50.1	61.1

NOTES:

AECOM

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

MOISTURE CONTENT

JOB No.: 60282083

DATE: December 4, 2012

CLIENT: Dillon Consulting

PROJECT: Route 90 Extension

HOLE NO.	TH12-02	-	-	TH12-03	-	-
SAMPLE NO.	G36	S37	G38	G40	G41	G43
DEPTH (FT)	62.0 - 62.5	65.0 - 66.5	68.0 - 69.0	3.0 - 4.0	7.5 - 8.0	13.5 - 14.0
MOISTURE CONTENT %	21.5	10.6	7.9	36.9	51.1	49.1

HOLE NO.	TH12-03	-	-	-	-	TH12-05
SAMPLE NO.	S44	G45	S47	G48	S49	G50
DEPTH (FT)	20.0 - 21.5	27.0 - 28.0	40.0 - 41.5	43.0 - 43.5	45.0 - 45.5	4.0 - 4.5
MOISTURE CONTENT %	49.6	52.0	12.9	12.1	7.6	21.6

HOLE NO.	TH12-05	-	-	-	TH12-01	-
SAMPLE NO.	S52	G54	S55	G56	G58	G59
DEPTH (FT)	10.0 - 11.5	21.0 - 21.5	25.0 - 26.5	30.5 - 31.0	4.0 - 4.5	9.0 - 9.5
MOISTURE CONTENT %	53.4	48.7	49.0	53.9	24.4	38.3

HOLE NO.	TH12-01	-	-	-		
SAMPLE NO.	G60	S61	G63	S65		
DEPTH (FT)	12.5 - 13.0	15.0 - 16.5	25.0 - 26.0	35.0 - 36.5		
MOISTURE CONTENT %	49.2	51.0	46.9	55.0		

NOTES:

ATTERBERG
(ASTM D4318-98)

AECOM

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

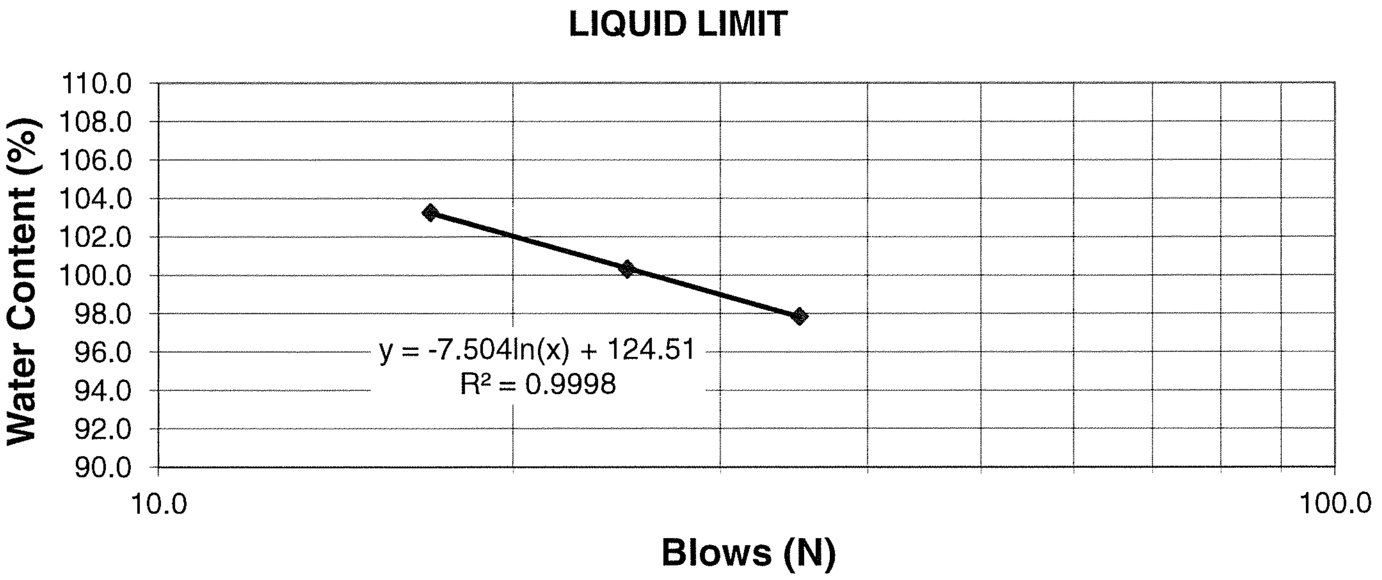
JOB No.:	60282083	DATE:	7-Dec-12
CLIENT:	Dillon Consulting	TEST HOLE:	TH12-04
PROJECT:	Route 90 Extension	SAMPLE:	T4
LOCATION:		DEPTH:	10.0 - 12.0'
		TECH.:	AL

Liquid Limit
WATER CONTENT

Blows	35	25	17		
WT. SAMPLE WET + TARE (gr)	89.709	92.284	94.550		
WT. SAMPLE DRY + TARE (gr)	85.604	88.099	90.350		
WT. TARE (gr)	81.409	83.927	86.283		
WT. WATER (gr)	4.105	4.185	4.200		
WT. DRY SOIL (gr)	4.195	4.172	4.067		
MOISTURE CONTENT (%)	97.855	100.312	103.270		

Plastic Limit
WATER CONTENT

WT. SAMPLE WET + TARE (gr)	91.793	89.088			
WT. SAMPLE DRY + TARE (gr)	90.318	87.976			
WT. TARE (gr)	85.644	84.372			
WT. WATER (gr)	1.475	1.112			
WT. DRY SOIL (gr)	4.674	3.604			
MOISTURE CONTENT (%)	31.558	30.855			



Liquid Limit = 100.4	Plastic Limit = 31.2	Plasticity Index = 69.1

ATTERBERG
(ASTM D4318-98)

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

JOB No.: 60282083
CLIENT: Dillon Consulting
PROJECT: Route 90 Extension
LOCATION:

DATE: 7-Dec-12
TEST HOLE: TH12-02
SAMPLE: T33A
DEPTH: 35.0 - 37.0'
TECH.: AL

Liquid Limit

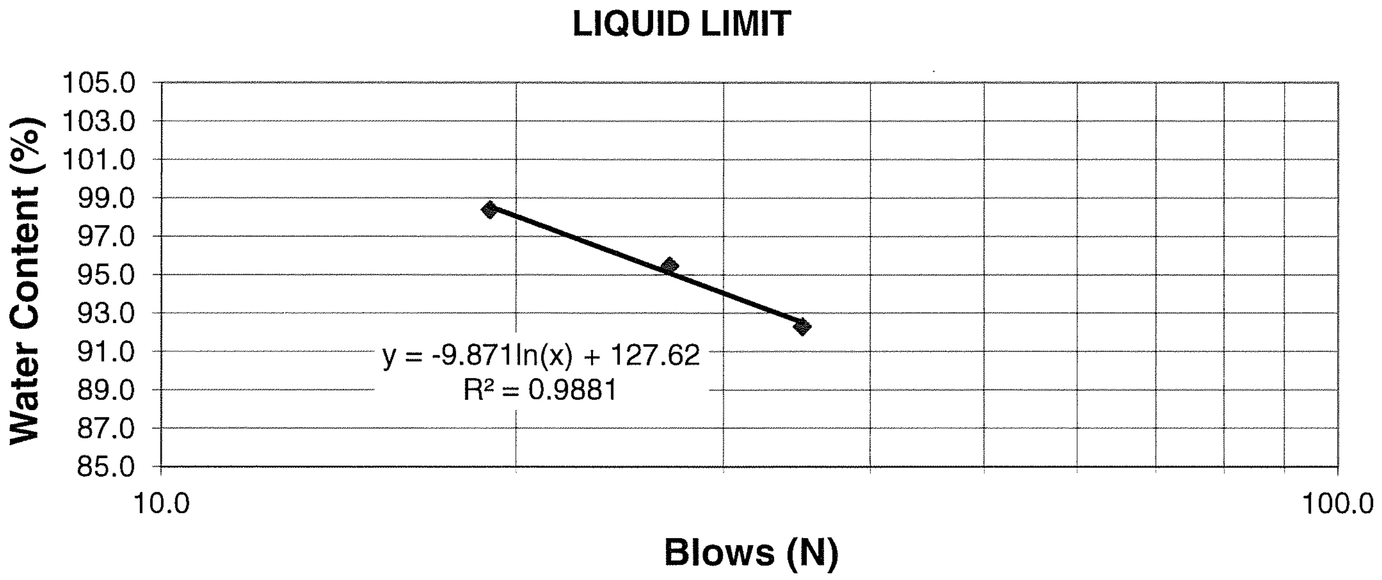
WATER CONTENT

Blows	35	27	19		
WT. SAMPLE WET + TARE (gr)	87.412	94.726	89.228		
WT. SAMPLE DRY + TARE (gr)	83.574	90.577	85.190		
WT. TARE (gr)	79.416	86.231	81.086		
WT. WATER (gr)	3.838	4.149	4.038		
WT. DRY SOIL (gr)	4.158	4.346	4.104		
MOISTURE CONTENT (%)	92.304	95.467	98.392		

Plastic Limit

WATER CONTENT

WT. SAMPLE WET + TARE (gr)	85.514	91.260			
WT. SAMPLE DRY + TARE (gr)	84.226	89.757			
WT. TARE (gr)	80.237	85.153			
WT. WATER (gr)	1.288	1.503			
WT. DRY SOIL (gr)	3.989	4.604			
MOISTURE CONTENT (%)	32.289	32.646			



Liquid Limit = 95.8	Plastic Limit = 32.5	Plasticity Index = 63.4

ATTERBERG
(ASTM D4318-98)

AECOM
MATERIALS LABORATORY
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99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
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JOB No.:	60282083	DATE:	7-Dec-12
CLIENT:	Dillon Consulting	TEST HOLE:	TH12-05
PROJECT:	Route 90 Extension	SAMPLE:	G50
LOCATION:		DEPTH:	4.0 - 4.5'
		TECH.:	AL

Liquid Limit

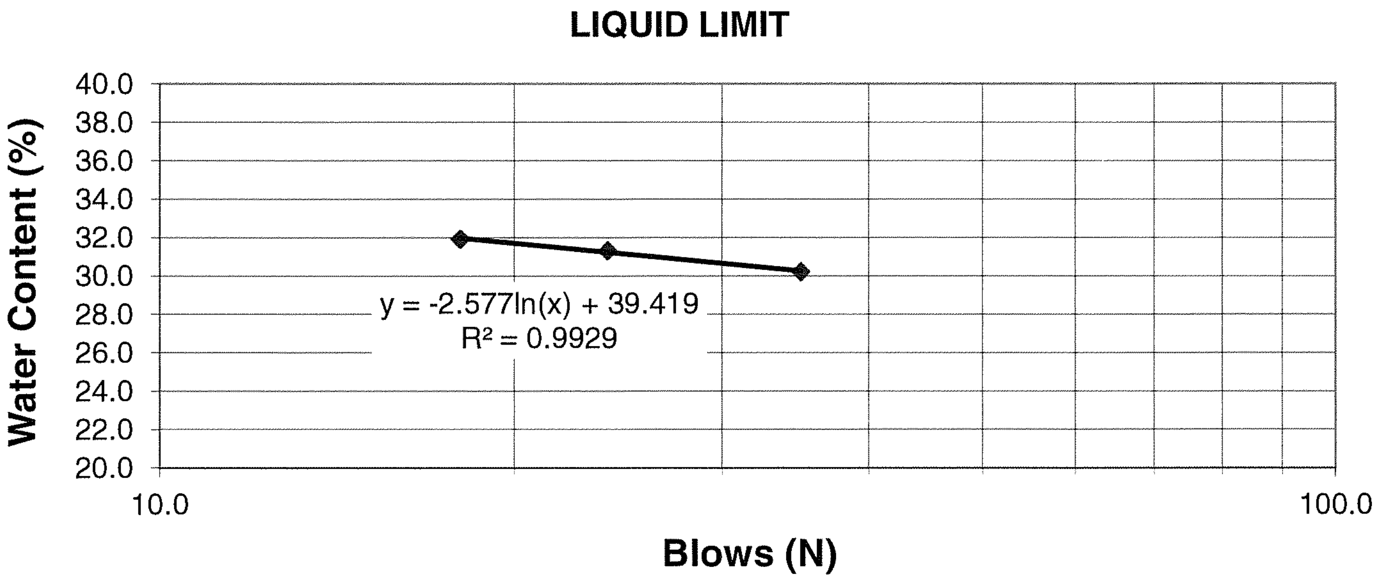
WATER CONTENT

Blows	35	24	18		
WT. SAMPLE WET + TARE (gr)	90.590	90.499	91.299		
WT. SAMPLE DRY + TARE (gr)	88.178	88.018	88.581		
WT. TARE (gr)	80.197	80.095	80.067		
WT. WATER (gr)	2.412	2.481	2.718		
WT. DRY SOIL (gr)	7.981	7.923	8.514		
MOISTURE CONTENT (%)	30.222	31.314	31.924		

Plastic Limit

WATER CONTENT

WT. SAMPLE WET + TARE (gr)	86.006	84.965			
WT. SAMPLE DRY + TARE (gr)	85.231	84.343			
WT. TARE (gr)	80.062	80.164			
WT. WATER (gr)	0.775	0.622			
WT. DRY SOIL (gr)	5.169	4.179			
MOISTURE CONTENT (%)	14.993	14.884			



Liquid Limit =	31.1	Plastic Limit =	14.9	Plasticity Index =	16.2

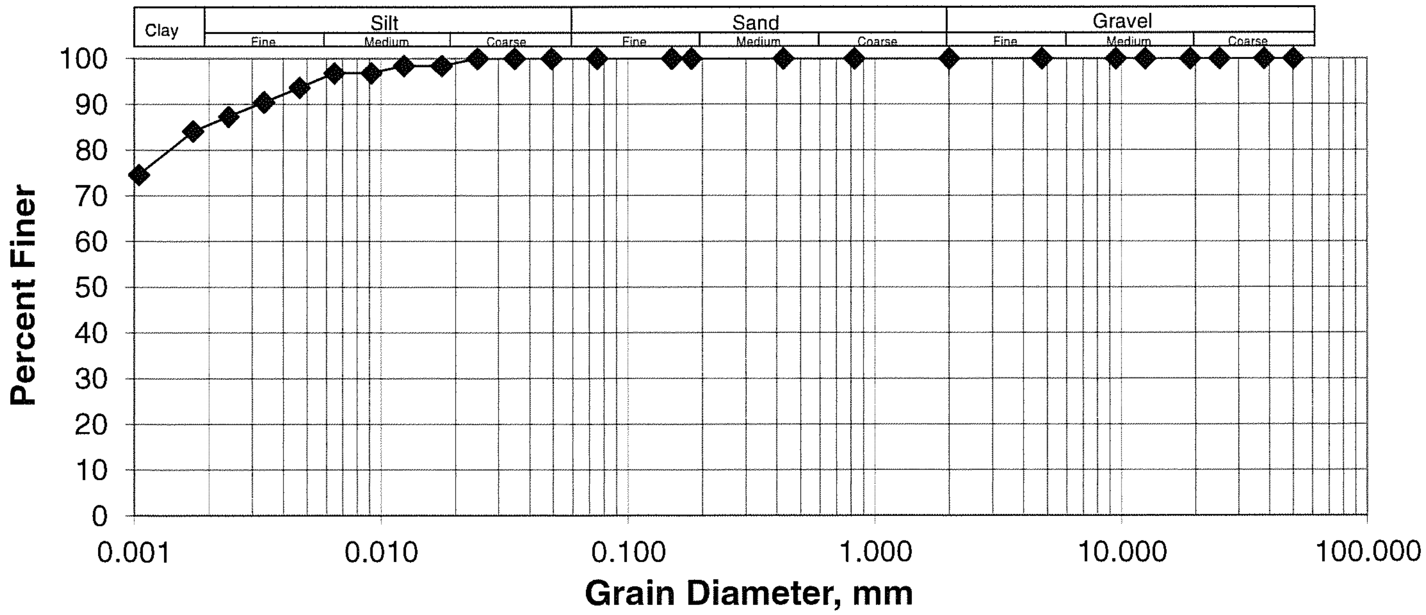
GRAIN SIZE DISTRIBUTION

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

Job No.:	60282083	Hole No.:	TH12-04
Client:	Dillon Consulting	Sample No.:	T4
Project :	Route 90 Extension	Depth:	10.0 - 12.0'
Date Tested:	7-Dec-12	Date Sampled:	
Tested By:		Sampled By:	

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	0.0175	98.4
9.5	100.0	0.075	100.0	0.0124	98.4
4.75	100.0			0.0091	96.8
2.00	100.0			0.0065	96.8
				0.0047	93.6
				0.0033	90.5
				0.0024	87.3
				0.0017	84.1
				0.0010	74.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**14.6%**
Sand	**0.0%**	**Clay**	**85.4%**

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

GRAIN SIZE DISTRIBUTION

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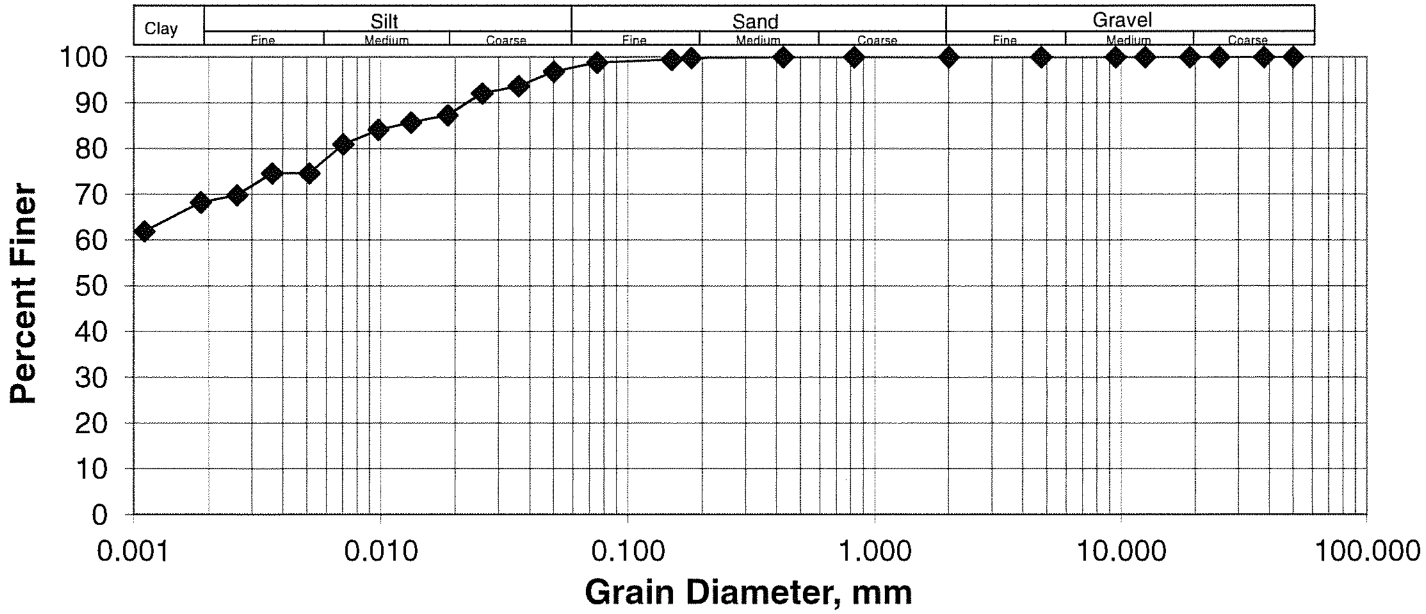
Job No.: 60282083
Client: Dillon Consulting
Project : Route 90 Extension
Date Tested: 7-Dec-12
Tested By:

Hole No.: TH12-02
Sample No.: T26
Depth: 10.0 - 12.0'
Date Sampled:
Sampled By:

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	98.8
38.0	100.0	0.83	100.0	0.0500	96.8
25.0	100.0	0.43	100.0	0.0360	93.6
19.0	100.0	0.18	99.8	0.0257	92.1
12.5	100.0	0.15	99.4	0.0186	87.3
9.5	100.0	0.075	98.8	0.0133	85.7
4.75	100.0			0.0098	84.1
2.00	100.0			0.0070	80.9
				0.0051	74.6
				0.0036	74.6
				0.0026	69.8
				0.0019	68.2
				0.0011	61.9

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**29.1%**
Sand	**2.4%**	**Clay**	**68.5%**

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

GRAIN SIZE DISTRIBUTION

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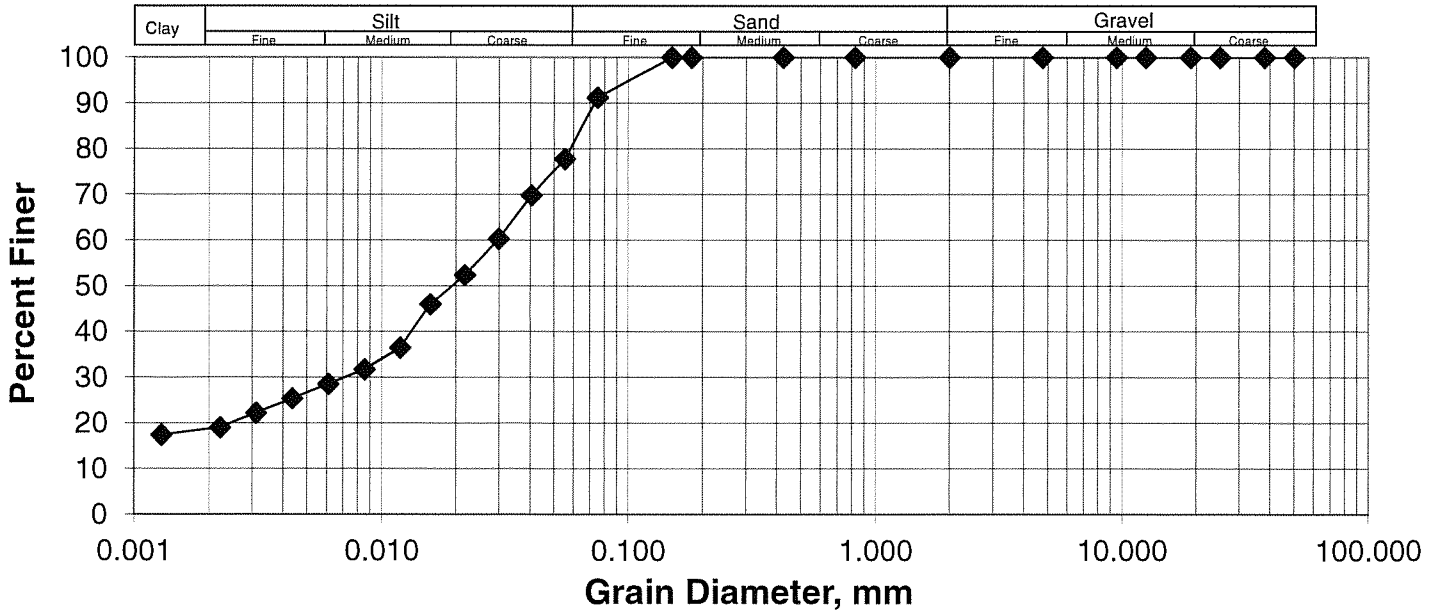
Job No.: 60282083
Client: Dillon Consulting
Project : Route 90 Extension
Date Tested: 6-Dec-12
Tested By:

Hole No.: TH12-02
Sample No.: G30
Depth: 23.0 - 23.5'
Date Sampled:
Sampled By:

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	91.2
38.0	100.0	0.83	100.0	0.0552	77.8
25.0	100.0	0.43	100.0	0.0405	69.8
19.0	100.0	0.18	100.0	0.0298	60.3
12.5	100.0	0.15	100.0	0.0217	52.3
9.5	100.0	0.075	91.2	0.0157	46.0
4.75	100.0			0.0119	36.5
2.00	100.0			0.0085	31.7
				0.0061	28.5
				0.0044	25.3
				0.0031	22.2
				0.0022	19.0
				0.0013	17.4

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**62.4%**
Sand	**19.0%**	**Clay**	**18.6%**

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

GRAIN SIZE DISTRIBUTION

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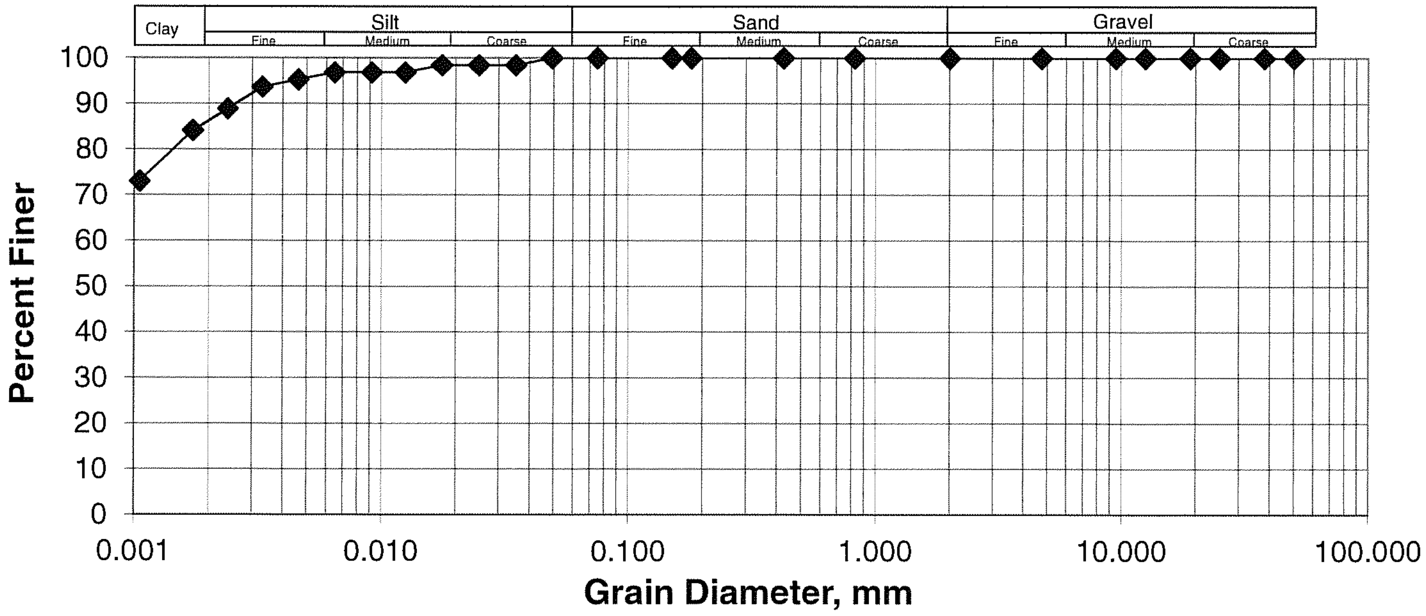
Job No.: 60282083
Client: Dillon Consulting
Project : Route 90 Extension
Date Tested: 7-Dec-12
Tested By:

Hole No.: TH12-02
Sample No.: T33A
Depth: 35.0 - 37.0'
Date Sampled:
Sampled By:

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	0.0351	98.4
19.0	100.0	0.18	100.0	0.0248	98.4
12.5	100.0	0.15	100.0	0.0175	98.4
9.5	100.0	0.075	100.0	0.0125	96.8
4.75	100.0			0.0091	96.8
2.00	100.0			0.0065	96.8
				0.0046	95.2
				0.0033	93.6
				0.0024	88.9
				0.0017	84.1
				0.0011	73.0

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**13.9%**
Sand	**0.0%**	**Clay**	**86.1%**

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

GRAIN SIZE DISTRIBUTION

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AECOM
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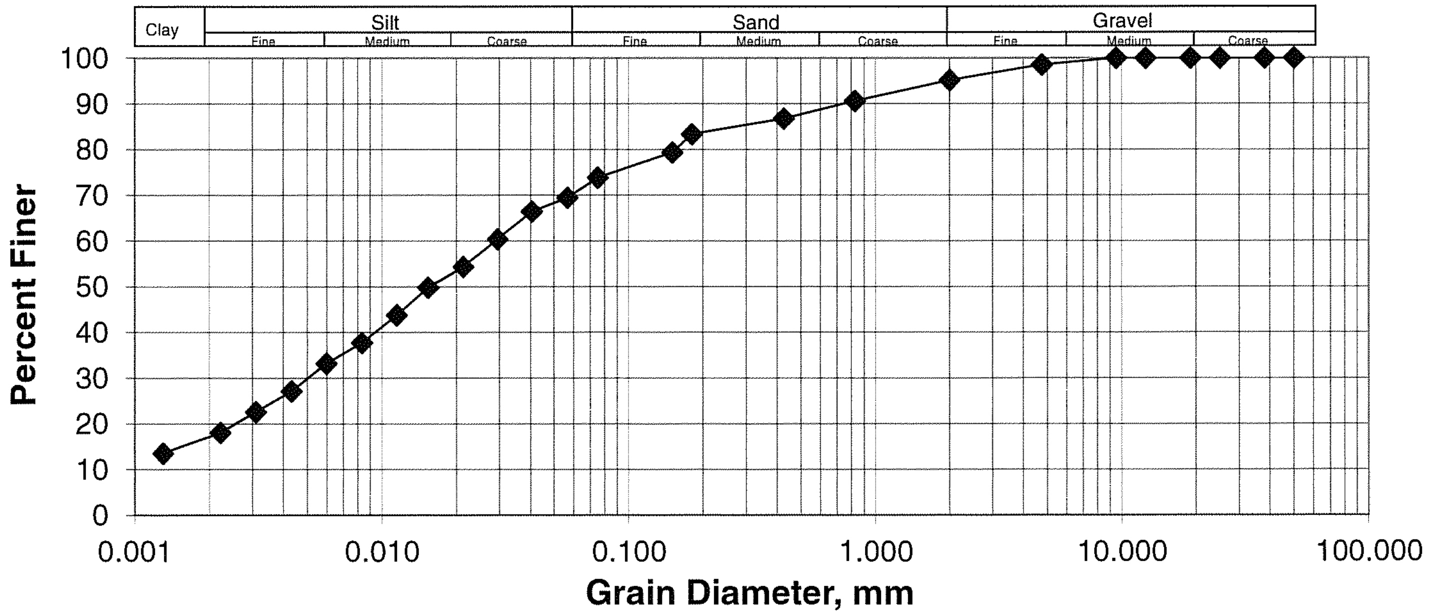
Job No.: 60282083
Client: Dillon Consulting
Project : Route 90 Extension
Date Tested: 6-Dec-12
Tested By:

Hole No.: TH12-03
Sample No.: G48
Depth: 43.0 - 43.5'
Date Sampled:
Sampled By:

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	95.2	0.0750	73.8
38.0	100.0	0.83	90.6	0.0565	69.5
25.0	100.0	0.43	86.8	0.0405	66.4
19.0	100.0	0.18	83.4	0.0294	60.4
12.5	100.0	0.15	79.4	0.0213	54.3
9.5	100.0	0.075	73.8	0.0154	49.8
4.75	98.5			0.0115	43.8
2.00	95.2			0.0083	37.7
				0.0060	33.2
				0.0043	27.1
				0.0031	22.6
				0.0022	18.1
				0.0013	13.5

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**4.8%**	**Silt**	**53.3%**
Sand	**24.9%**	**Clay**	**17.0%**

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

GRAIN SIZE DISTRIBUTION

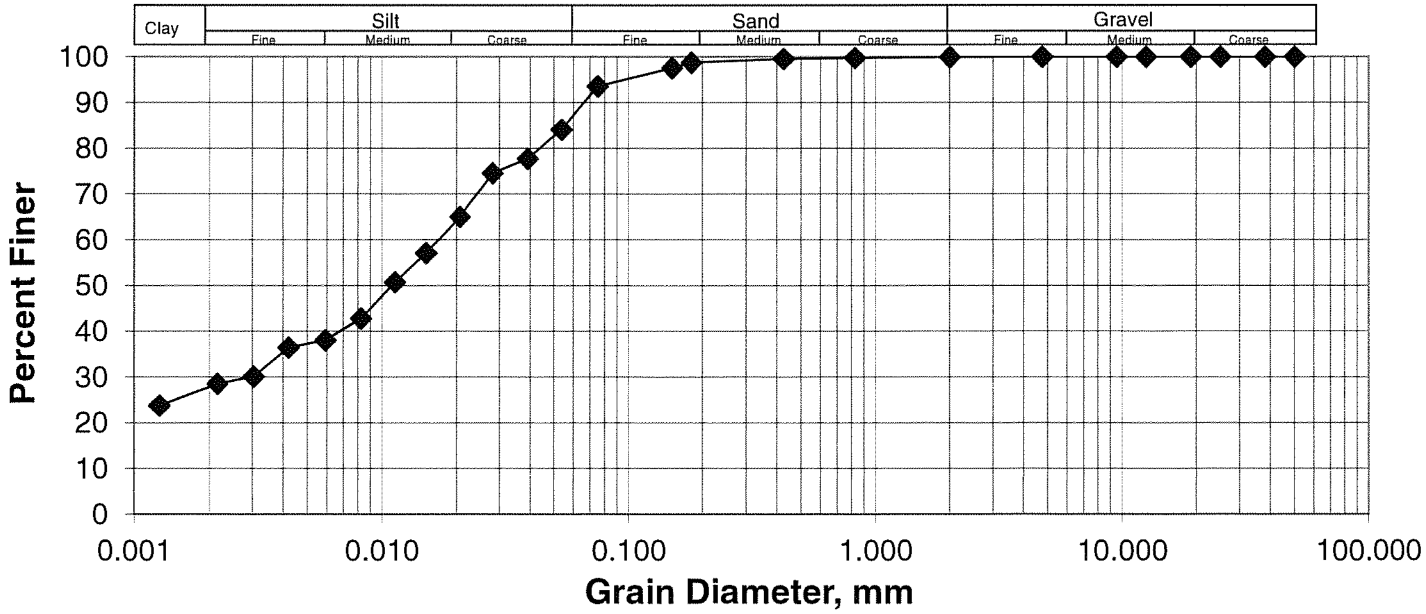
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381 **fax** (204) 284-2040

Job No.: 60282083
Client: Dillon Consulting
Project : Route 90 Extension
Date Tested: 7-Dec-12
Tested By:

Hole No.: TH12-05
Sample No.: G50
Depth: 4.0 - 4.5'
Date Sampled:
Sampled By:

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	99.9	0.0750	93.5
38.0	100.0	0.83	99.7	0.0536	84.1
25.0	100.0	0.43	99.5	0.0391	77.7
19.0	100.0	0.18	98.7	0.0280	74.5
12.5	100.0	0.15	97.5	0.0207	65.0
9.5	100.0	0.075	93.5	0.0151	57.1
4.75	100.0			0.0113	50.7
2.00	99.9			0.0082	42.8
				0.0059	38.0
				0.0042	36.4
				0.0030	30.1
				0.0022	28.5
				0.0013	23.7

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.1%**	**Silt**	**59.2%**
Sand	**13.0%**	**Clay**	**27.7%**

** 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: Route 90 Extension
JOB NO.: 60282083

TEST HOLE NO.:	**TH12-04**
SAMPLE NO.:	**T4**
SAMPLE DEPTH:	10.0 - 12.0'
DATE TESTED:	5-Dec-12
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.65
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**63.8**
Undrained Shear Strength (ksf)	1.33
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)	1.25
Undrained Shear Strength (kPa)	**59.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**71.1**
Unconfined compressive strength (ksf)	**1.5**
Undrained Shear Strength (kPa)	**35.6**
Undrained Shear Strength (ksf)	**0.743**
MOISTURE CONTENT	
Tare Number	B60
Wt. Sample wet + tare (g)	467.3
Wt. Sample dry + tare (g)	310.9
Wt. Tare (g)	8.3
Moisture Content %	**51.7**
BULK DENSITY	
Sample Wt. (g)	1071.6
Diameter 1 (cm)	7.24
Diameter 2 (cm)	7.20
Diameter 3 (cm)	7.25
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	15.34
Length 2 (cm)	15.40
Length 3 (cm)	15.41
Avg. Length (cm)	**15.38**
Volume (cm^3)	631.6
Moisture content (%)	51.7
Bulk Density (g/cm^3)	1.697
Bulk Density (kN/m^3)	**16.6**
Bulk Density (pcf)	**105.9**
Dry Density (kN/m^3)	**10.97**

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

AECOM

CLIENT:	Dillon Consulting
PROJECT:	Route 90 Extension
JOB NO.:	60282083

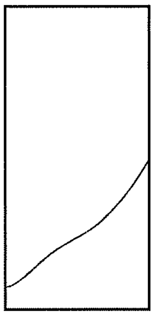
TEST HOLE NO.:	TH12-04
SAMPLE NO.:	T4
SAMPLE DEPTH:	10.0 - 12.0'
SAMPLE DATE:	26-Nov-12
TEST DATE:	5-Dec-12

SOIL DESCRIPTION:

CLAY; silt pockets (4 mm), brown, moist, firm, high plasticity,
slickensides, stratified (6 - 20 mm)

MOISTURE CONTENT: 51.7

SAMPLE DIAM.(Do):	72.30	**(mm)**	**INITIAL AREA, Ao:**	4105.5	**(mm^2)**
SAMPLE LENGTH, (Lo):	153.83	**(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)**



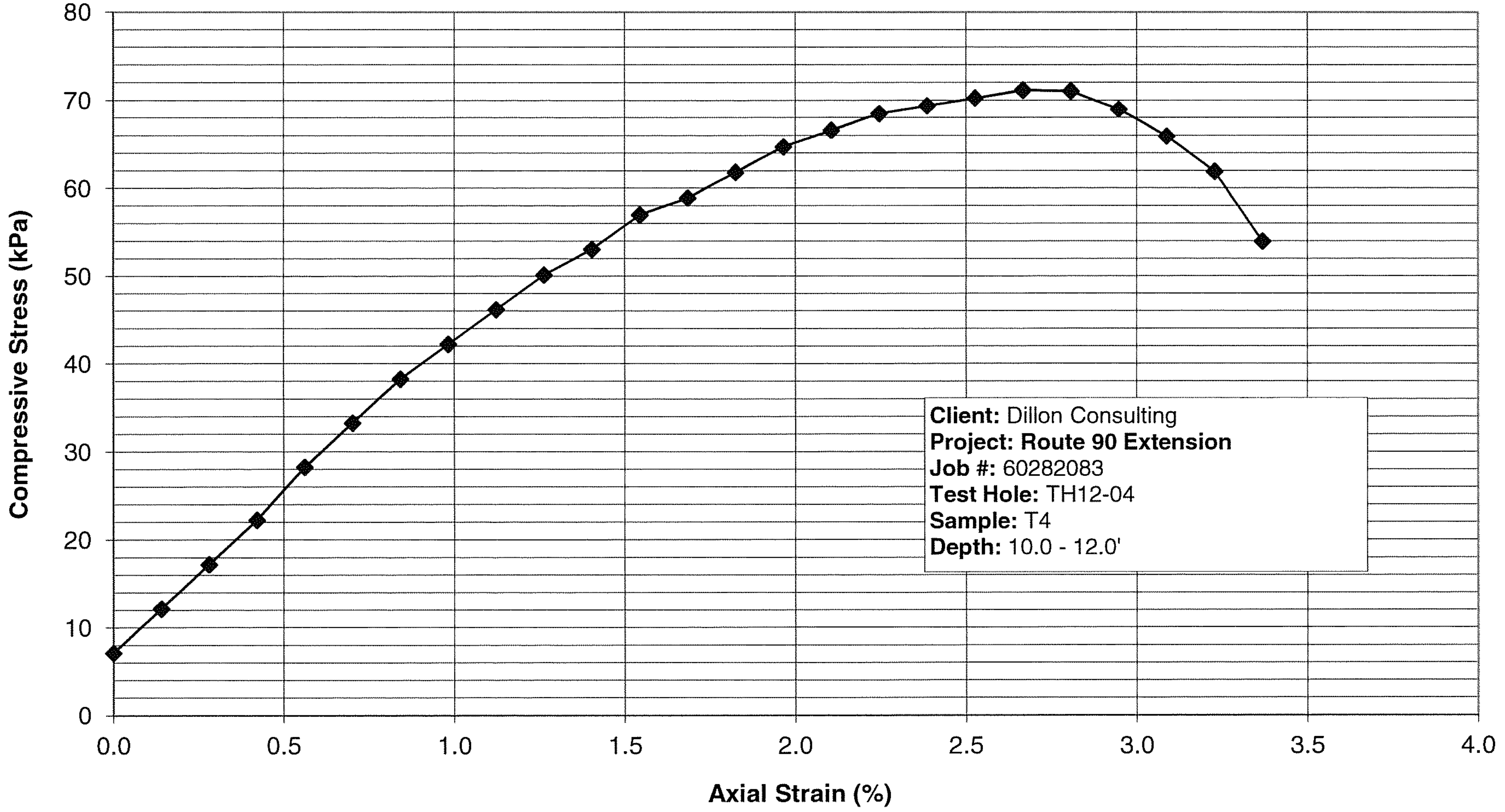
FAILURE SKETCH

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	0.0007	0.00	6.36	6.56	1.03	0.148	7.1
0.02	0.0012	0.14	6.37	11.24	1.76	0.254	12.2
0.03	0.0017	0.28	6.38	15.93	2.50	0.359	17.2
0.03	0.0022	0.42	6.39	20.61	3.23	0.465	22.2
0.04	0.0028	0.56	6.40	26.24	4.10	0.590	28.3
0.05	0.0033	0.70	6.41	30.92	4.82	0.695	33.3
0.06	0.0038	0.84	6.42	35.61	5.55	0.799	38.3
0.07	0.0042	0.98	6.43	39.35	6.12	0.882	42.2
0.08	0.0046	1.12	6.44	43.10	6.70	0.964	46.2
0.09	0.0050	1.26	6.44	46.85	7.27	1.047	50.1
0.09	0.0053	1.40	6.45	49.66	7.69	1.108	53.1
0.10	0.0057	1.54	6.46	53.41	8.26	1.190	57.0
0.11	0.0059	1.68	6.47	55.28	8.54	1.230	58.9
0.12	0.0062	1.82	6.48	58.09	8.96	1.291	61.8
0.13	0.0065	1.96	6.49	60.91	9.38	1.351	64.7
0.14	0.0067	2.11	6.50	62.78	9.66	1.391	66.6
0.14	0.0069	2.25	6.51	64.65	9.93	1.430	68.5
0.15	0.0070	2.39	6.52	65.59	10.06	1.449	69.4
0.16	0.0071	2.53	6.53	66.53	10.19	1.467	70.3
0.17	0.0072	2.67	6.54	67.46	10.32	1.486	71.1
0.18	0.0072	2.81	6.55	67.46	10.30	1.484	71.0
0.19	0.0070	2.95	6.56	65.59	10.00	1.440	69.0
0.20	0.0067	3.09	6.57	62.78	9.56	1.377	65.9
0.20	0.0063	3.23	6.58	59.03	8.98	1.293	61.9
0.21	0.0055	3.37	6.59	51.54	7.83	1.127	54.0

UNCONFINED COMPRESSIVE STRENGTH, q_u:	71.15	kPa
(based on maximum q_u value)	1.486	ksf
UNDRAINED SHEAR STRENGTH, S_u:	35.57	kPa
(based on maximum q_u value)	0.743	ksf

NOTES:
Sample condition was poor after pushing, but could still be tested.

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: Dillon Consulting
Project: Route 90 Extension
Job #: 60282083
Test Hole: TH12-04
Sample: T4
Depth: 10.0 - 12.0'

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

CLIENT: Dillon Consulting
PROJECT: Route 90 Extension
JOB NO.: 60282083

TEST HOLE NO.:	**TH12-02**
SAMPLE NO.:	**T26**
SAMPLE DEPTH:	10.0 - 12.0'
DATE TESTED:	5-Dec-12
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.90
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**88.3**
Undrained Shear Strength (ksf)	1.84
POCKET PENETROMETER	
Reading - Qu (tsf)	2.00
Undrained Shear Strength (kPa)	**95.8**
Reading - Qu (tsf)	1.75
Undrained Shear Strength (kPa)	**83.8**
Reading - Qu (tsf)	1.75
Undrained Shear Strength (kPa)	**83.8**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**n/a**
Unconfined compressive strength (ksf)	**n/a**
Undrained Shear Strength (kPa)	**n/a**
Undrained Shear Strength (ksf)	**n/a**
MOISTURE CONTENT	
Tare Number	MAC2
Wt. Sample wet + tare (g)	604.8
Wt. Sample dry + tare (g)	461.6
Wt. Tare (g)	8.4
Moisture Content %	**31.6**
BULK DENSITY	
Sample Wt. (g)	1097.6
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.23
Diameter 3 (cm)	7.27
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	14.40
Length 2 (cm)	14.30
Length 3 (cm)	14.33
Avg. Length (cm)	**14.34**
Volume (cm^3)	589.4
Moisture content (%)	31.6
Bulk Density (g/cm^3)	1.862
Bulk Density (kN/m^3)	**18.3**
Bulk Density (pcf)	**116.3**
Dry Density (kN/m^3)	**13.88**

AECOM

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

CLIENT:	Dillon Consulting
PROJECT:	Route 90 Extension
JOB NO.:	60282083

TEST HOLE NO.:	TH12-02
SAMPLE NO.:	T26
SAMPLE DEPTH:	10.0 - 12.0'
SAMPLE DATE:	26-Nov-12
TEST DATE:	5-Dec-12

SOIL DESCRIPTION:

CLAY; some silt, brown, moist, firm to stiff, high plasticity,
silt lenses

MOISTURE CONTENT: 31.6

SAMPLE DIAM.(Do):	72.33	(mm)	**INITIAL AREA, Ao:**	4109.3	(mm^2)
SAMPLE LENGTH, (Lo):	143.43	(mm)	**PISTON RATE:**	0.051	(inches / minute)
L / D RATIO:	1.98	(2 < L/D < 2.5)	**AXIAL STRAIN RATE, R:**	n/a	(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)

UNCONFINED COMPRESSIVE STRENGTH, q_u:	0.00	kPa
(based on maximum q_u value)	0.000	ksf
UNDRAINED SHEAR STRENGTH, S_u:	0.00	kPa
(based on maximum q_u value)	0.000	ksf

NOTES:
Sample was not tested due to breakage after trimming.

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

CLIENT: Dillon Consulting
PROJECT: Route 90 Extension
JOB NO.: 60282083

TEST HOLE NO.:	**TH12-02**
SAMPLE NO.:	**T33A**
SAMPLE DEPTH:	35.0 - 37.0'
DATE TESTED:	5-Dec-12
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)	0.75
Undrained Shear Strength (kPa)	**35.9**
Reading - Qu (tsf)	1.00
Undrained Shear Strength (kPa)	**47.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**51.5**
Unconfined compressive strength (ksf)	**1.1**
Undrained Shear Strength (kPa)	**25.8**
Undrained Shear Strength (ksf)	**0.538**
MOISTURE CONTENT	
Tare Number	AK13
Wt. Sample wet + tare (g)	488.8
Wt. Sample dry + tare (g)	326.8
Wt. Tare (g)	8.5
Moisture Content %	**50.9**
BULK DENSITY	
Sample Wt. (g)	941.2
Diameter 1 (cm)	7.10
Diameter 2 (cm)	6.73
Diameter 3 (cm)	6.98
Avg. Diameter (cm)	**6.94**
Length 1 (cm)	15.29
Length 2 (cm)	15.24
Length 3 (cm)	15.29
Avg. Length (cm)	**15.27**
Volume (cm^3)	577.2
Moisture content (%)	50.9
Bulk Density (g/cm^3)	1.631
Bulk Density (kN/m^3)	**16.0**
Bulk Density (pcf)	**101.8**
Dry Density (kN/m^3)	**10.60**

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

AECOM

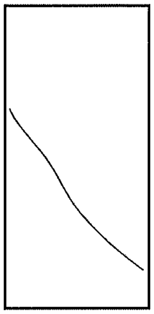
CLIENT:	Dillon Consulting
PROJECT:	Route 90 Extension
JOB NO.:	60282083

TEST HOLE NO.:	TH12-02
SAMPLE NO.:	T33A
SAMPLE DEPTH:	35.0 - 37.0'
SAMPLE DATE:	
TEST DATE:	5-Dec-12

SOIL DESCRIPTION:

CLAY; some silt, brown, moist, firm, high plasticity, homogeneous, slickensides

MOISTURE CONTENT: 50.9



FAILURE SKETCH

SAMPLE DIAM.(Do):	69.37	(mm)	**INITIAL AREA, Ao:**	3779.1	(mm^2)
SAMPLE LENGTH, (Lo):	152.73	(mm)	**PISTON RATE:**	0.051	(inches / minute)
L / D RATIO:	2.20	(2 < L/D < 2.5)	**AXIAL STRAIN RATE, R:**	0.85	(0.5<R<2 % / minute)

TEST DATA - DIAL READINGS

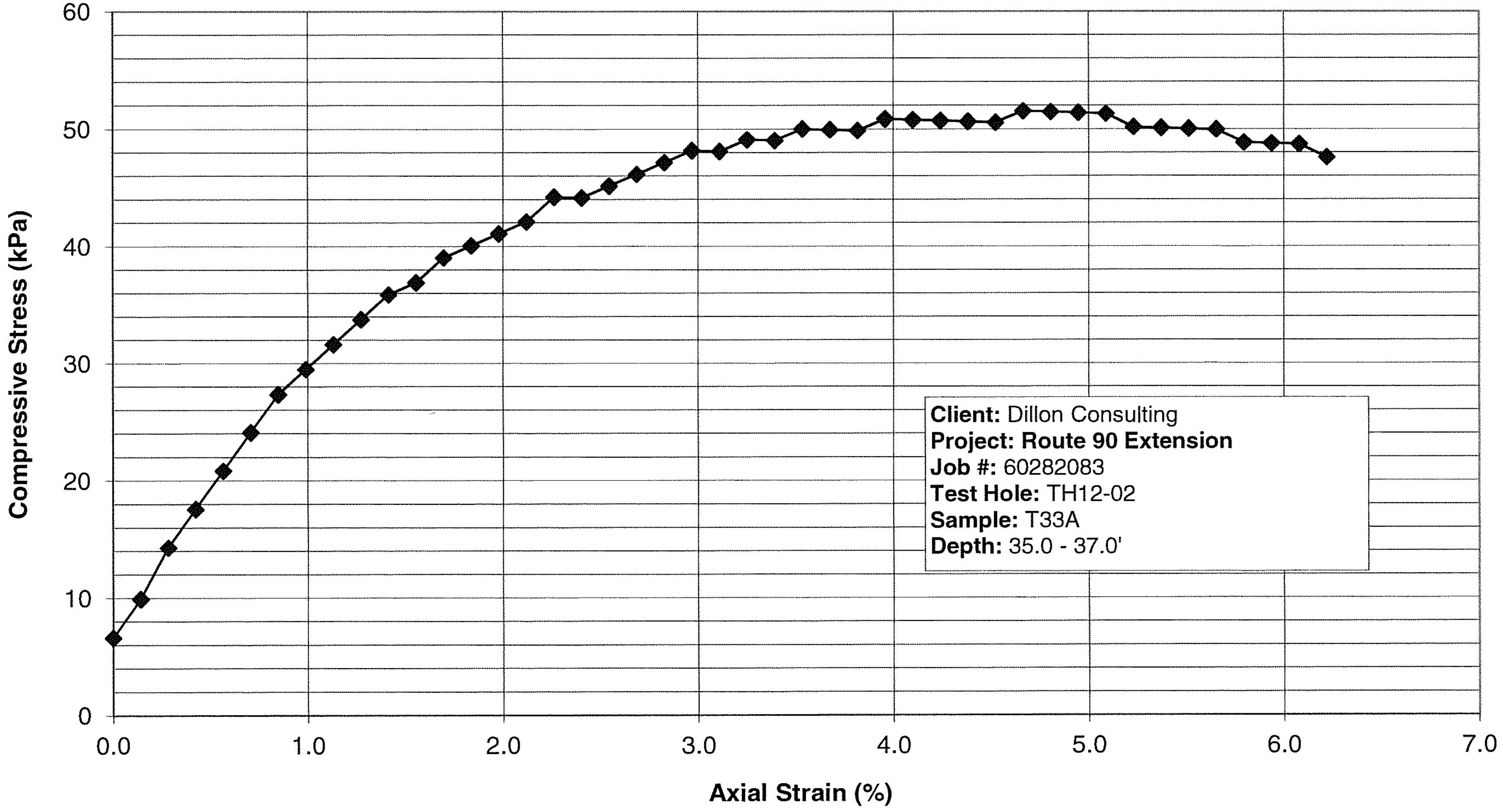
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_c		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0006	0.00	5.86	5.62	0.96	0.138	6.6
0.02	0.0009	0.14	5.87	8.43	1.44	0.207	9.9
0.03	0.0013	0.28	5.87	12.18	2.07	0.299	14.3
0.03	0.0016	0.42	5.88	14.99	2.55	0.367	17.6
0.04	0.0019	0.57	5.89	17.80	3.02	0.435	20.8
0.05	0.0022	0.71	5.90	20.61	3.49	0.503	24.1
0.06	0.0025	0.85	5.91	23.43	3.97	0.571	27.3
0.07	0.0027	0.99	5.92	25.30	4.28	0.616	29.5
0.08	0.0029	1.13	5.92	27.17	4.59	0.660	31.6
0.09	0.0031	1.27	5.93	29.05	4.90	0.705	33.8
0.09	0.0033	1.41	5.94	30.92	5.20	0.749	35.9
0.10	0.0034	1.55	5.95	31.86	5.35	0.771	36.9
0.11	0.0036	1.70	5.96	33.73	5.66	0.815	39.0
0.12	0.0037	1.84	5.97	34.67	5.81	0.837	40.1
0.13	0.0038	1.98	5.98	35.61	5.96	0.858	41.1
0.14	0.0039	2.12	5.98	36.54	6.11	0.879	42.1
0.14	0.0041	2.26	5.99	38.42	6.41	0.923	44.2
0.15	0.0041	2.40	6.00	38.42	6.40	0.922	44.1
0.16	0.0042	2.54	6.01	39.35	6.55	0.943	45.1
0.17	0.0043	2.69	6.02	40.29	6.69	0.964	46.2
0.18	0.0044	2.83	6.03	41.23	6.84	0.985	47.2
0.19	0.0045	2.97	6.04	42.17	6.98	1.006	48.2
0.20	0.0045	3.11	6.05	42.17	6.97	1.004	48.1
0.20	0.0046	3.25	6.05	43.10	7.12	1.025	49.1
0.21	0.0046	3.39	6.06	43.10	7.11	1.024	49.0
0.22	0.0047	3.53	6.07	44.04	7.25	1.044	50.0
0.23	0.0047	3.68	6.08	44.04	7.24	1.043	49.9
0.24	0.0047	3.82	6.09	44.04	7.23	1.041	49.9
0.25	0.0048	3.96	6.10	44.98	7.37	1.062	50.8
0.26	0.0048	4.10	6.11	44.98	7.36	1.060	50.8
0.26	0.0048	4.24	6.12	44.98	7.35	1.059	50.7
0.27	0.0048	4.38	6.13	44.98	7.34	1.057	50.6
0.28	0.0048	4.52	6.14	44.98	7.33	1.056	50.5
0.29	0.0049	4.66	6.14	45.91	7.47	1.076	51.5
0.30	0.0049	4.81	6.15	45.91	7.46	1.074	51.4
0.31	0.0049	4.95	6.16	45.91	7.45	1.073	51.4
0.31	0.0049	5.09	6.17	45.91	7.44	1.071	51.3
0.32	0.0048	5.23	6.18	44.98	7.28	1.048	50.2
0.33	0.0048	5.37	6.19	44.98	7.27	1.046	50.1
0.34	0.0048	5.51	6.20	44.98	7.25	1.045	50.0
0.35	0.0048	5.65	6.21	44.98	7.24	1.043	49.9
0.36	0.0047	5.80	6.22	44.04	7.08	1.020	48.8
0.37	0.0047	5.94	6.23	44.04	7.07	1.018	48.8
0.37	0.0047	6.08	6.24	44.04	7.06	1.017	48.7
0.38	0.0046	6.22	6.25	43.10	6.90	0.994	47.6

UNCONFINED COMPRESSIVE STRENGTH, q_u:	51.52	kPa
(based on maximum q_u value)	1.076	ksf
UNDRAINED SHEAR STRENGTH, S_u:	25.76	kPa
(based on maximum q_u value)	0.538	ksf

NOTES:
Sample condition was poor after pushing, but could still be tested.

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)

AECOM

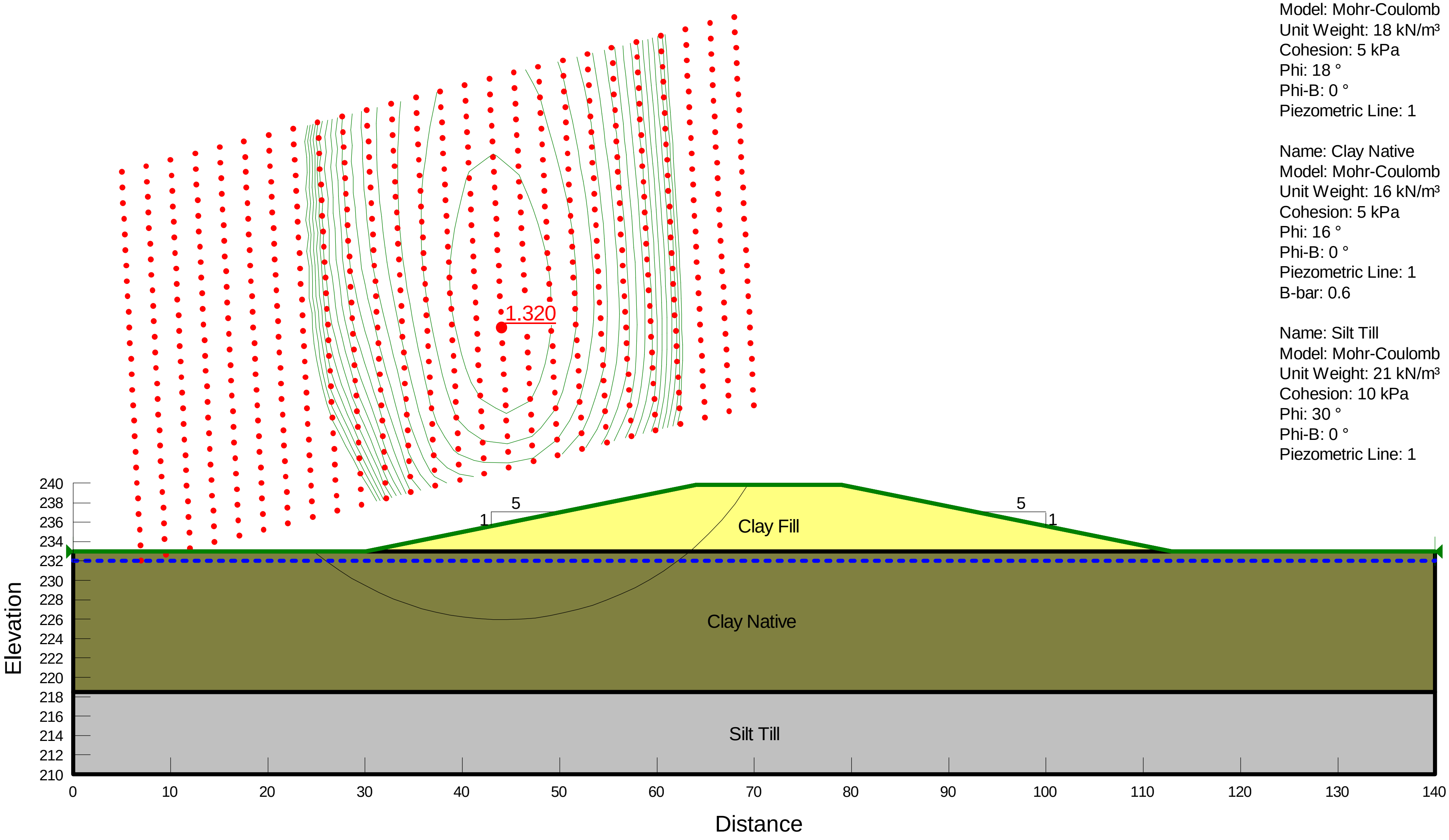


Client: Dillon Consulting
Project: Route 90 Extension
Job #: 60282083
Test Hole: TH12-02
Sample: T33A
Depth: 35.0 - 37.0'

Appendix D

Stability Analysis Results

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Exist. PWP)
Side Slope at Max. Embankment Fill



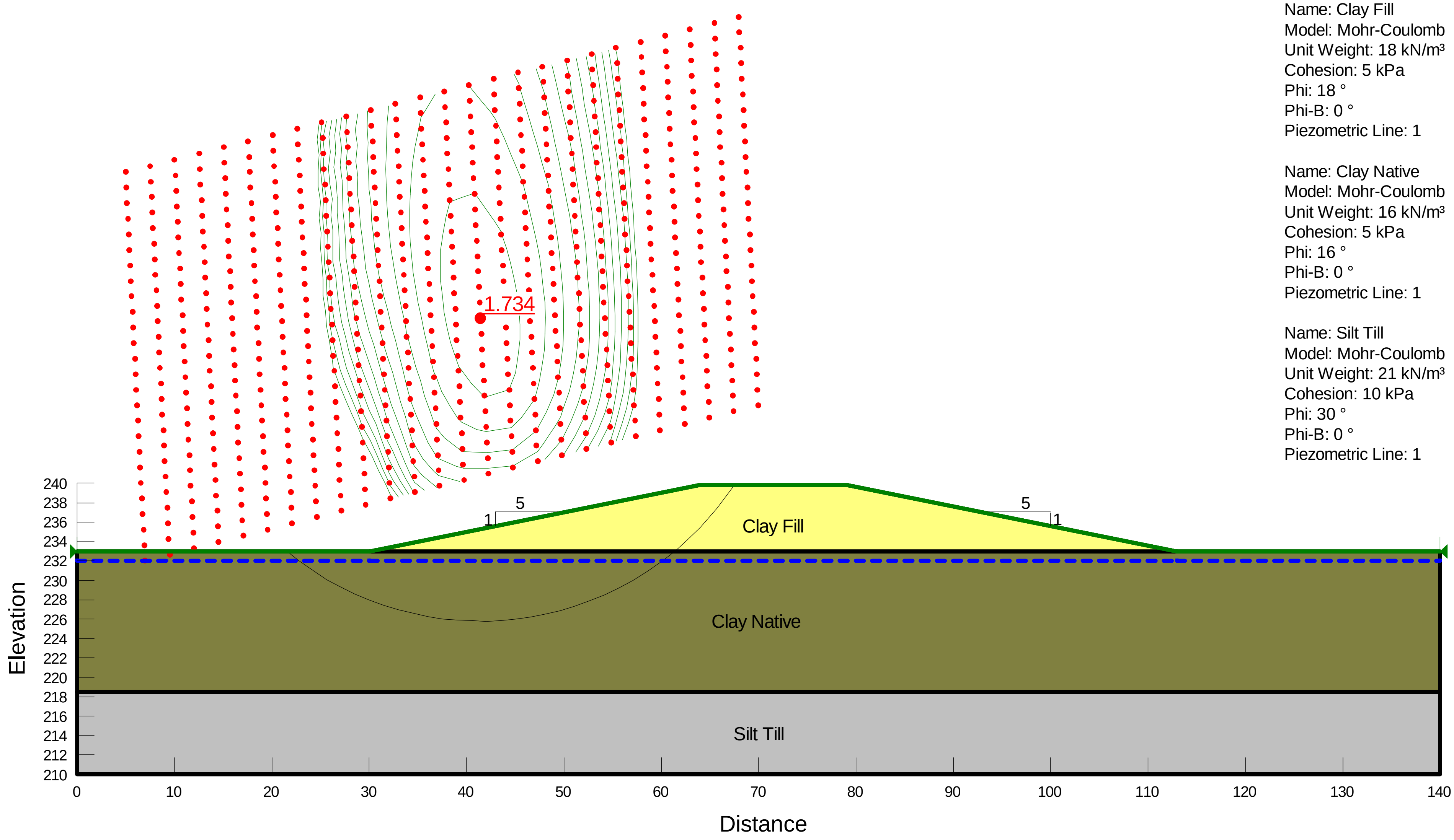
Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1
B-bar: 0.6

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

Figure 001

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Long Term)
Side Slope at Max. Embankment Fill



Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

Figure 002

Kenaston Blvd and Bishop Grandin Blvd
Head Slope Embankment - Slope Stability (Exist. PWP)
Concrete Box Abutment - East Abutment

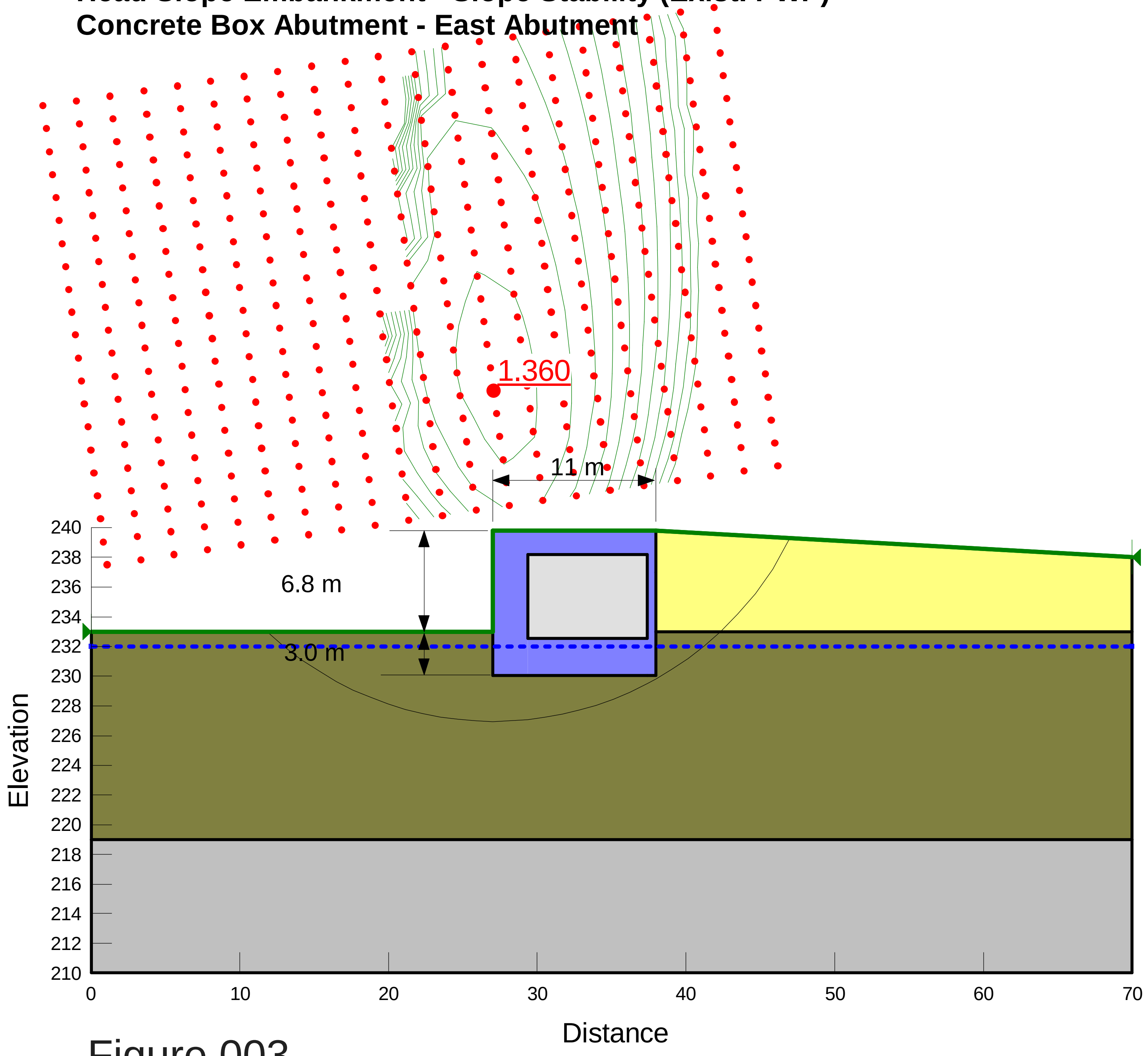


Figure 003

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1
B-bar: 0.6

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1
B-bar: 0

Name: Concrete Box
Model: Mohr-Coulomb
Unit Weight: 23.5 kN/m³
Cohesion: 100 kPa
Phi: 35 °
Phi-B: 0 °

Name: Abutment Room
Model: (None)

Kenaston Blvd and Bishop Grandin Blvd
Head Slope Embankment - Slope Stability (Long Term)
Concrete Box Abutment - East Abutment

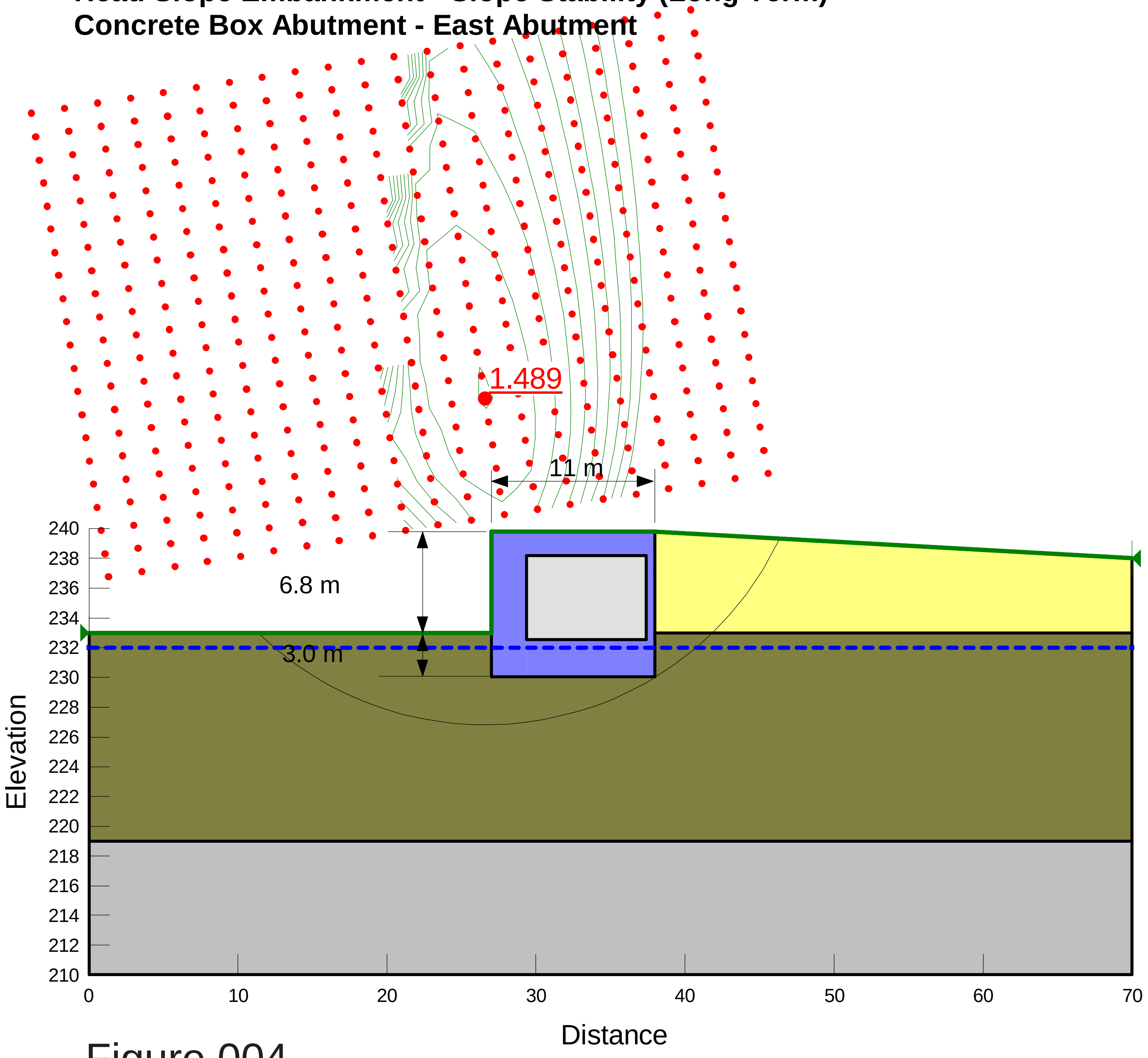


Figure 004

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

Name: Concrete Box
Model: Mohr-Coulomb
Unit Weight: 23.5 kN/m³
Cohesion: 100 kPa
Phi: 35 °
Phi-B: 0 °

Name: Abutment Room
Model: (None)

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Exist. PWP) - North Side
Retaining Wall Location - Near Hydro Tower

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1
B-bar: 0.6

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

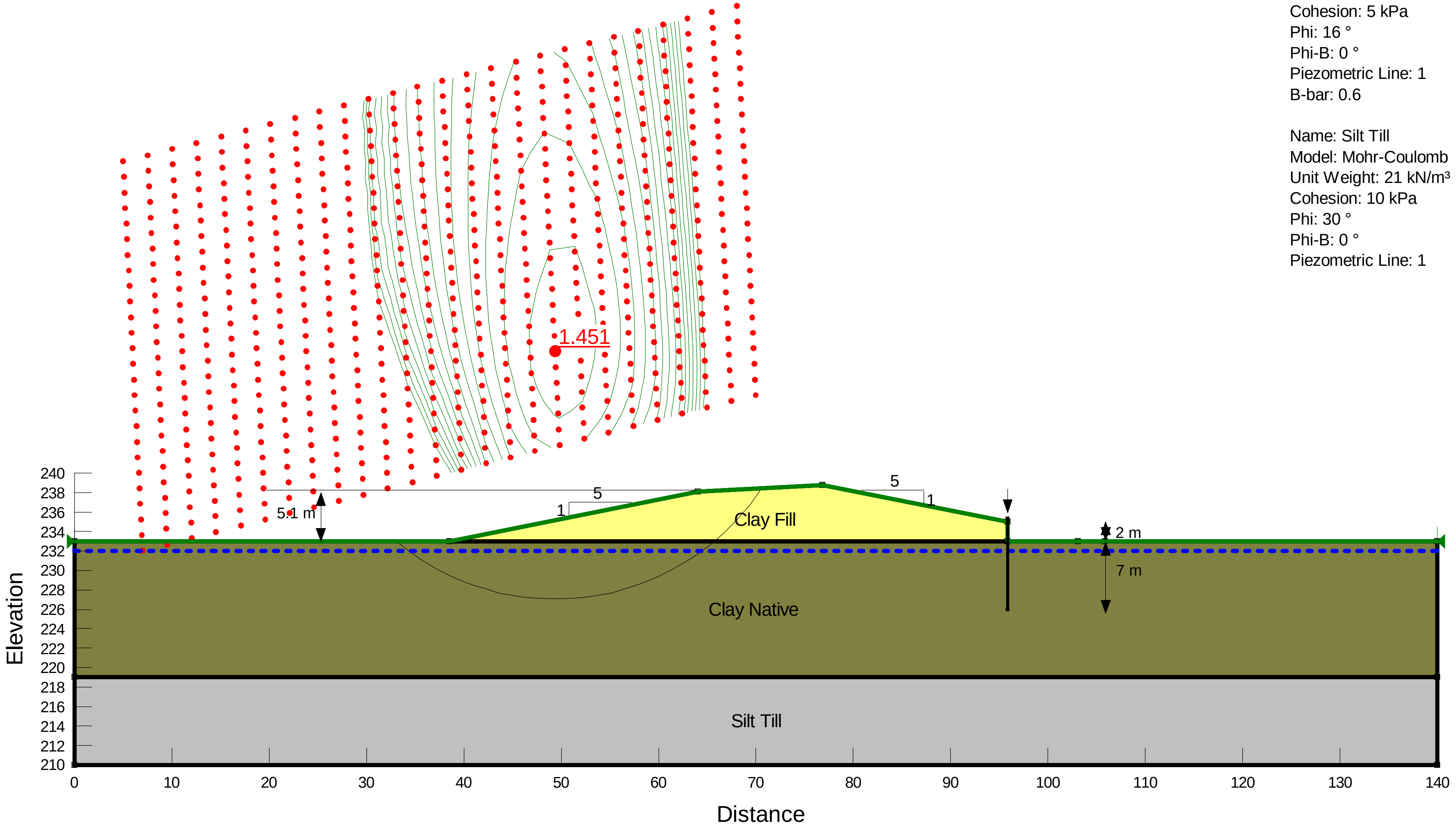


Figure 005

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Long Term) - North Side
Retaining Wall Location - Near Hydro Tower

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

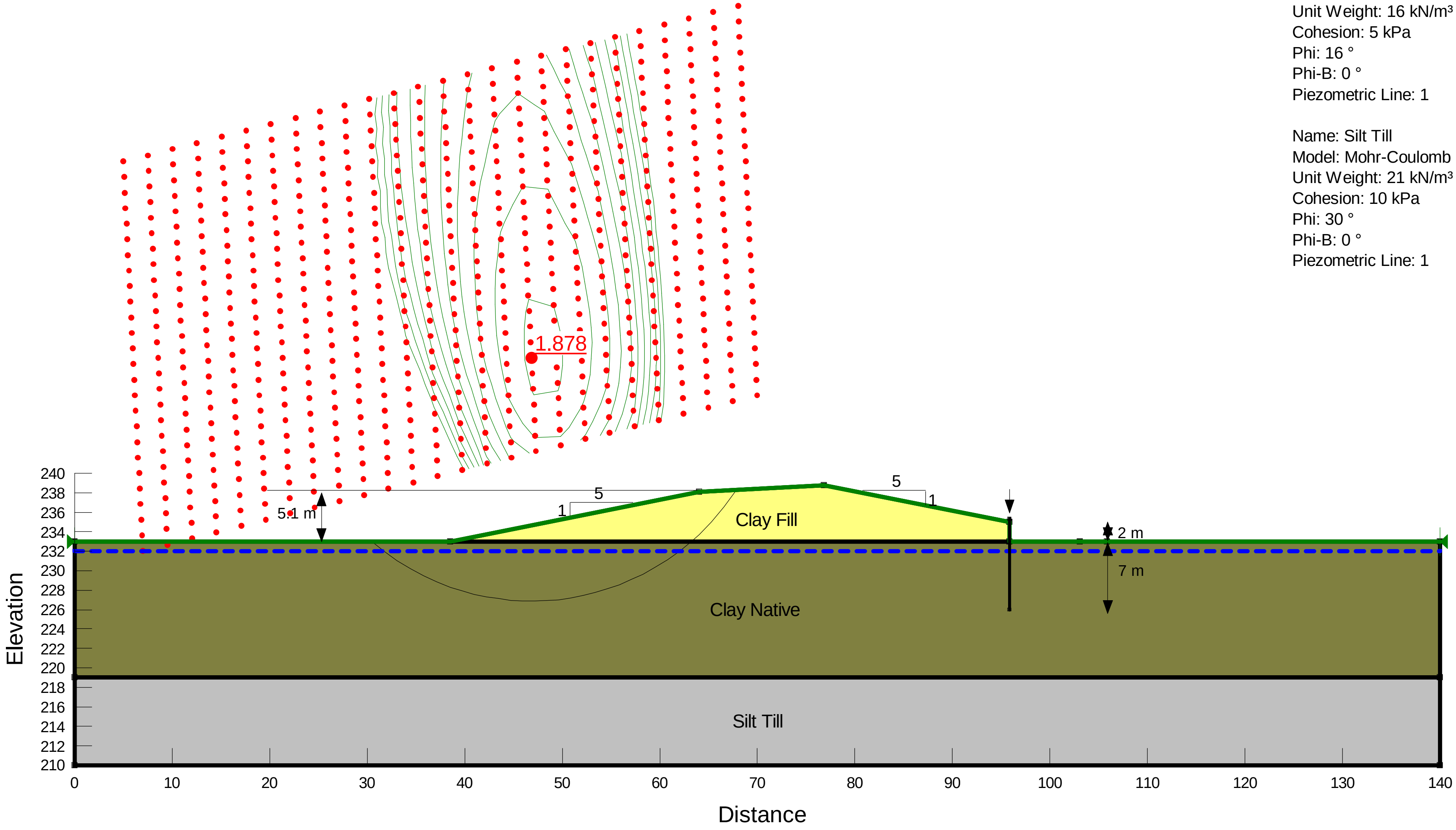


Figure 006

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Exist. PWP) - South Side
Retaining Wall Location - Near Hydro Tower

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1
B-bar: 0.6

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

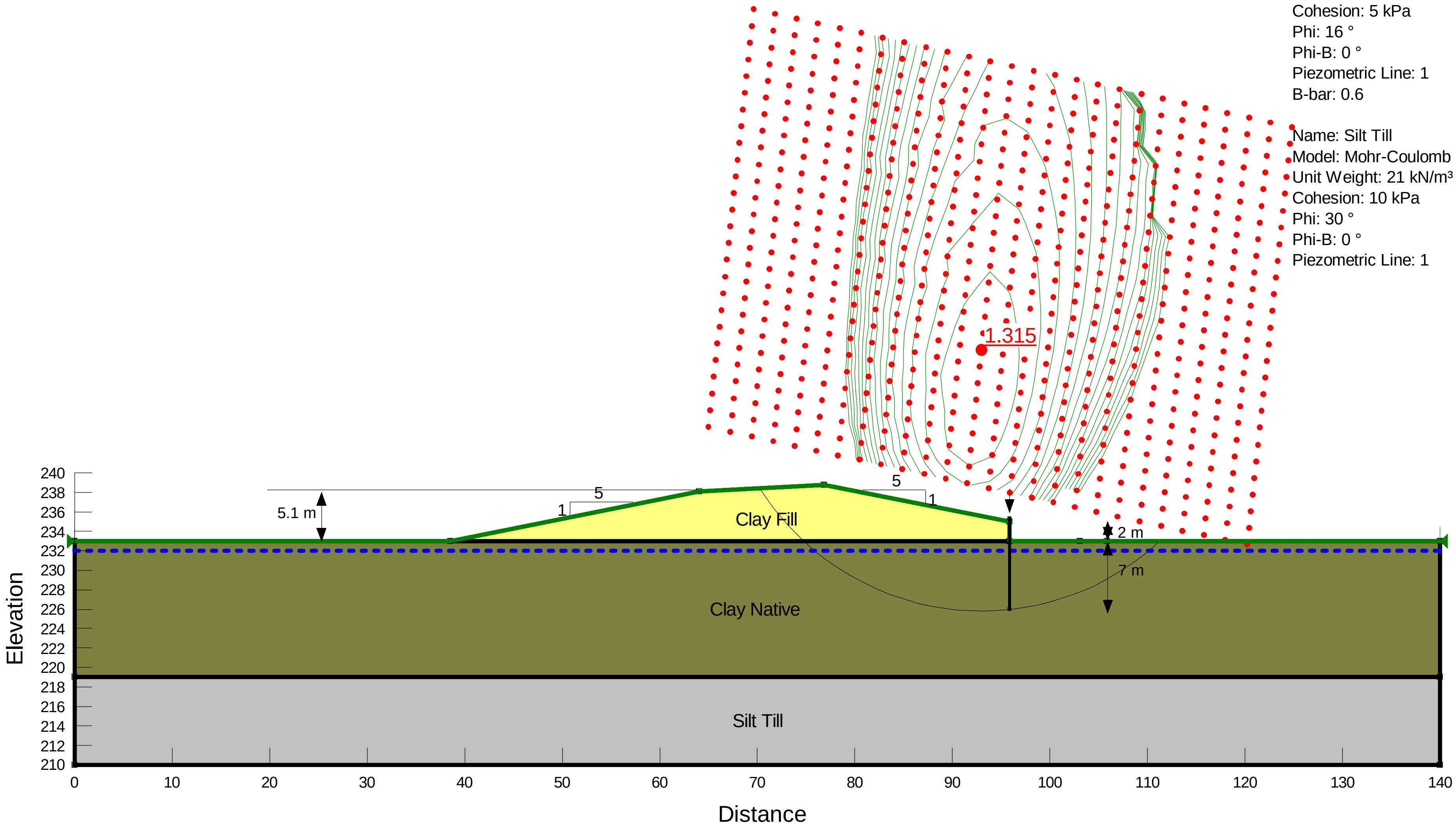


Figure 007

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Long Term) - South Side
Retaining Wall Location - Near Hydro Tower

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Phi-B: 0 °
Piezometric Line: 1

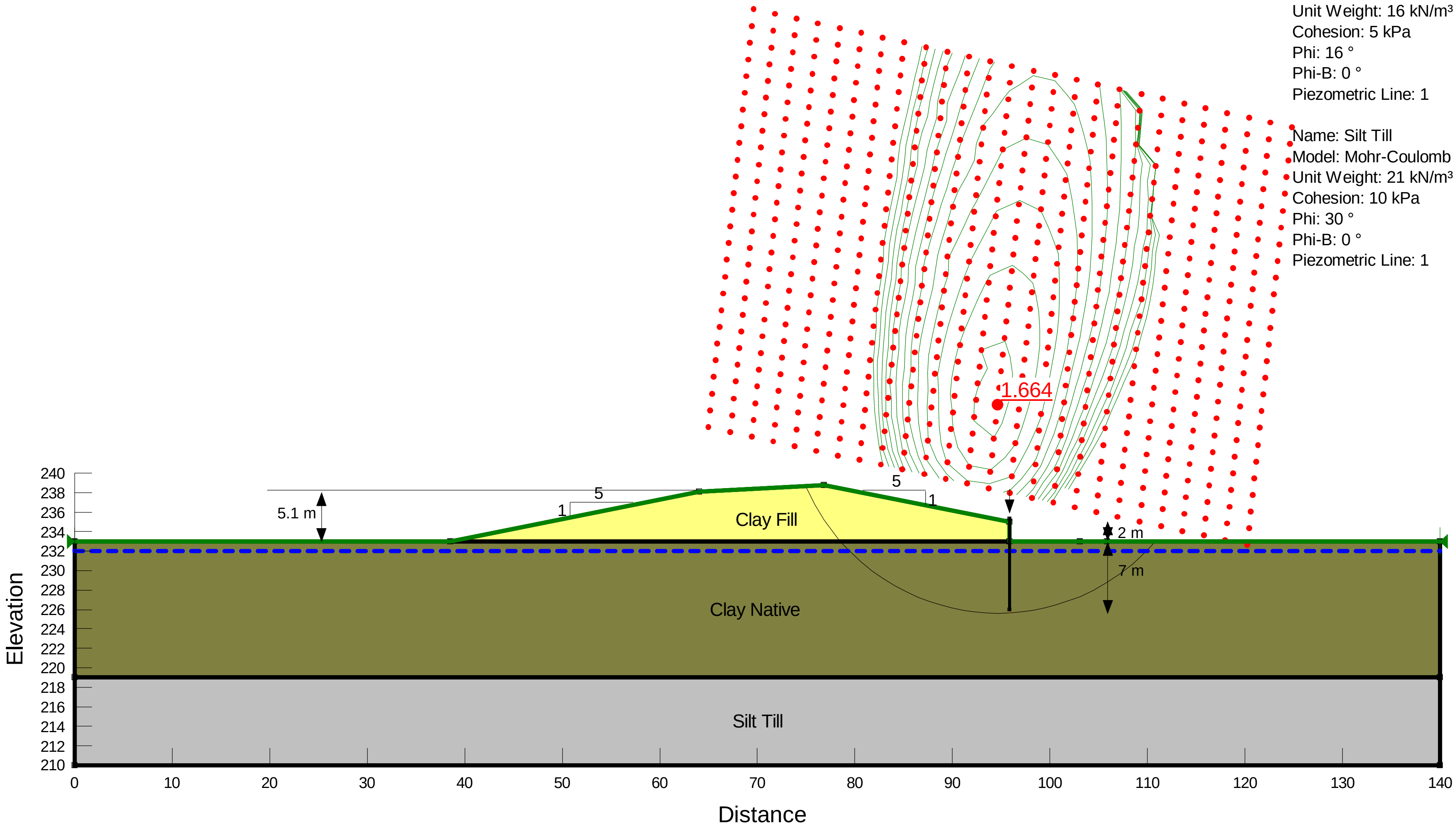
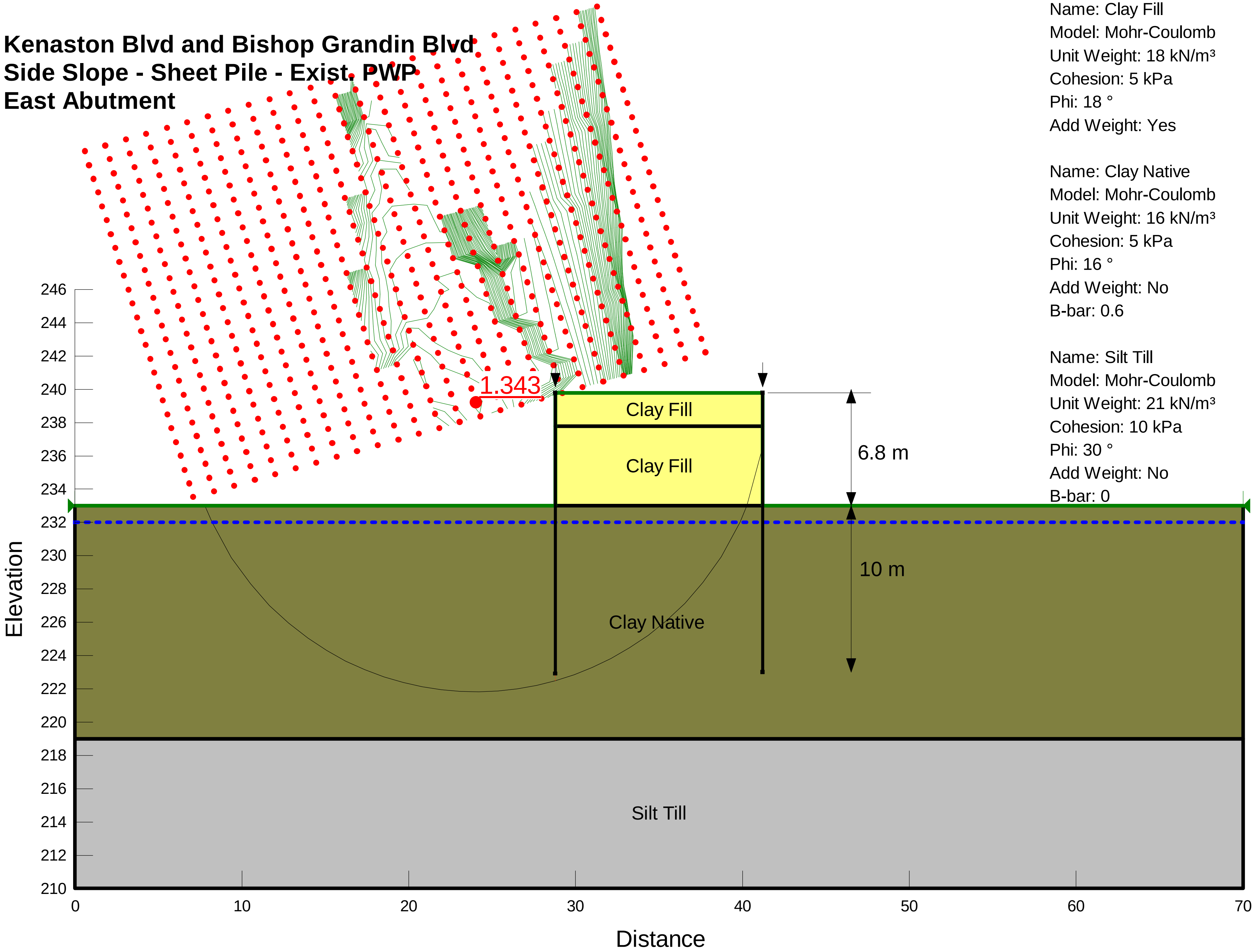


Figure 008



Kenaston Blvd and Bishop Grandin Blvd
Side Slope - Sheet Pile - Long Term
East Abutment

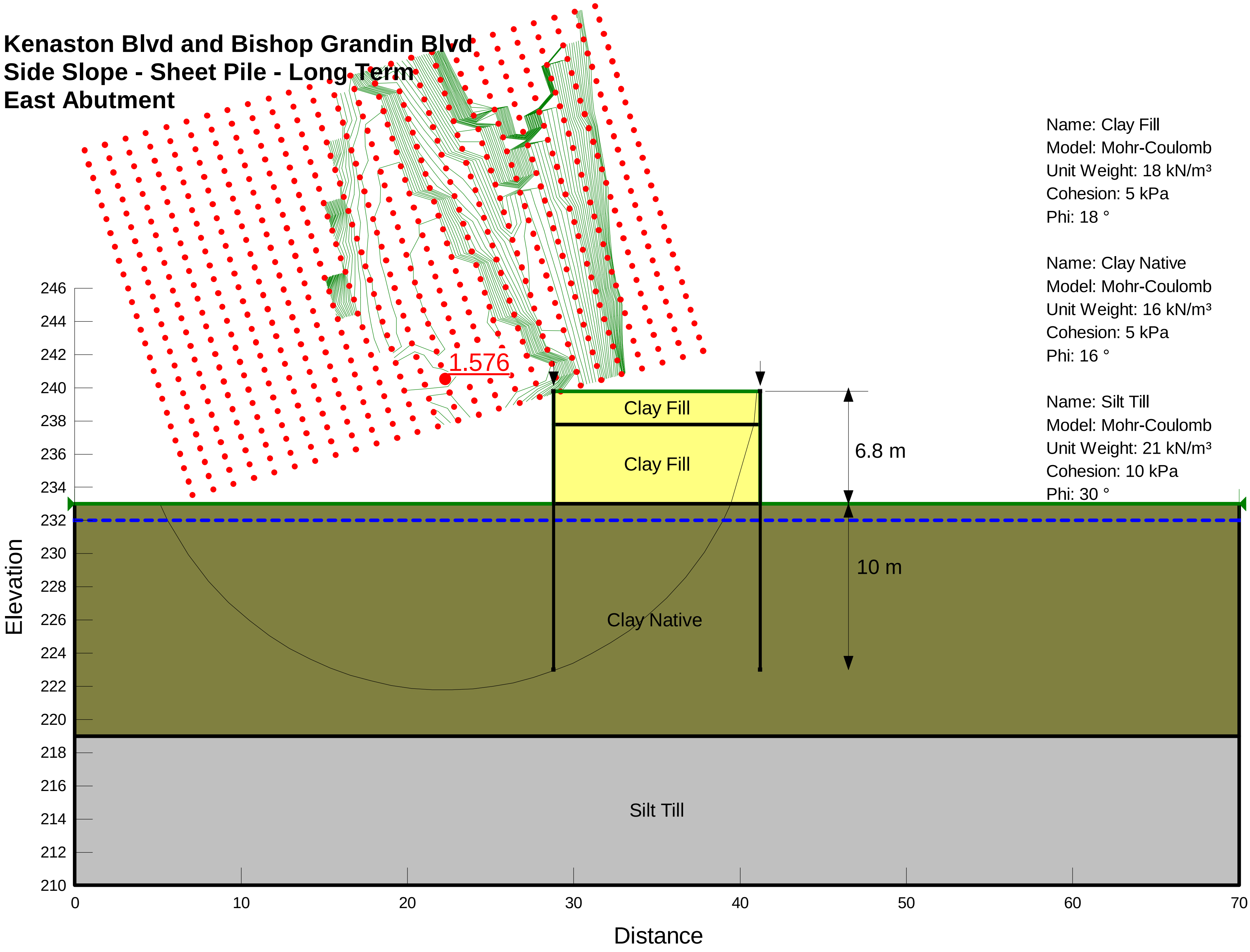


Figure 010

Kenaston Blvd and Bishop Grandin Blvd
West Flyover Embankment - Slope Stability (Exist. PWP)
Side Slope at Max. Embankment Fill

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Piezometric Line: 1
B-bar: 0
Add Weight: Yes

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Piezometric Line: 1
B-bar: 0.6
Add Weight: No

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Piezometric Line: 1
Add Weight: No

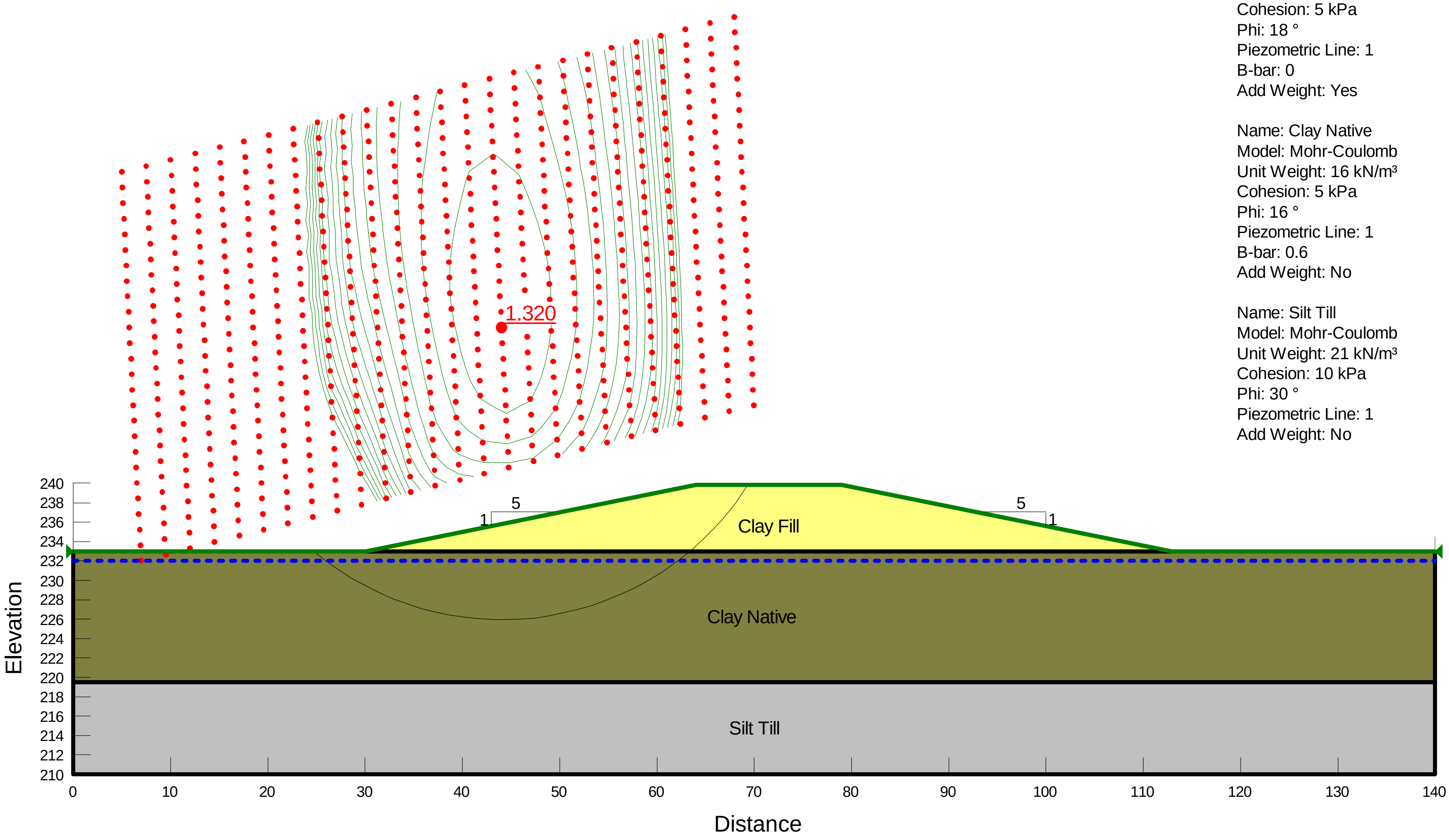
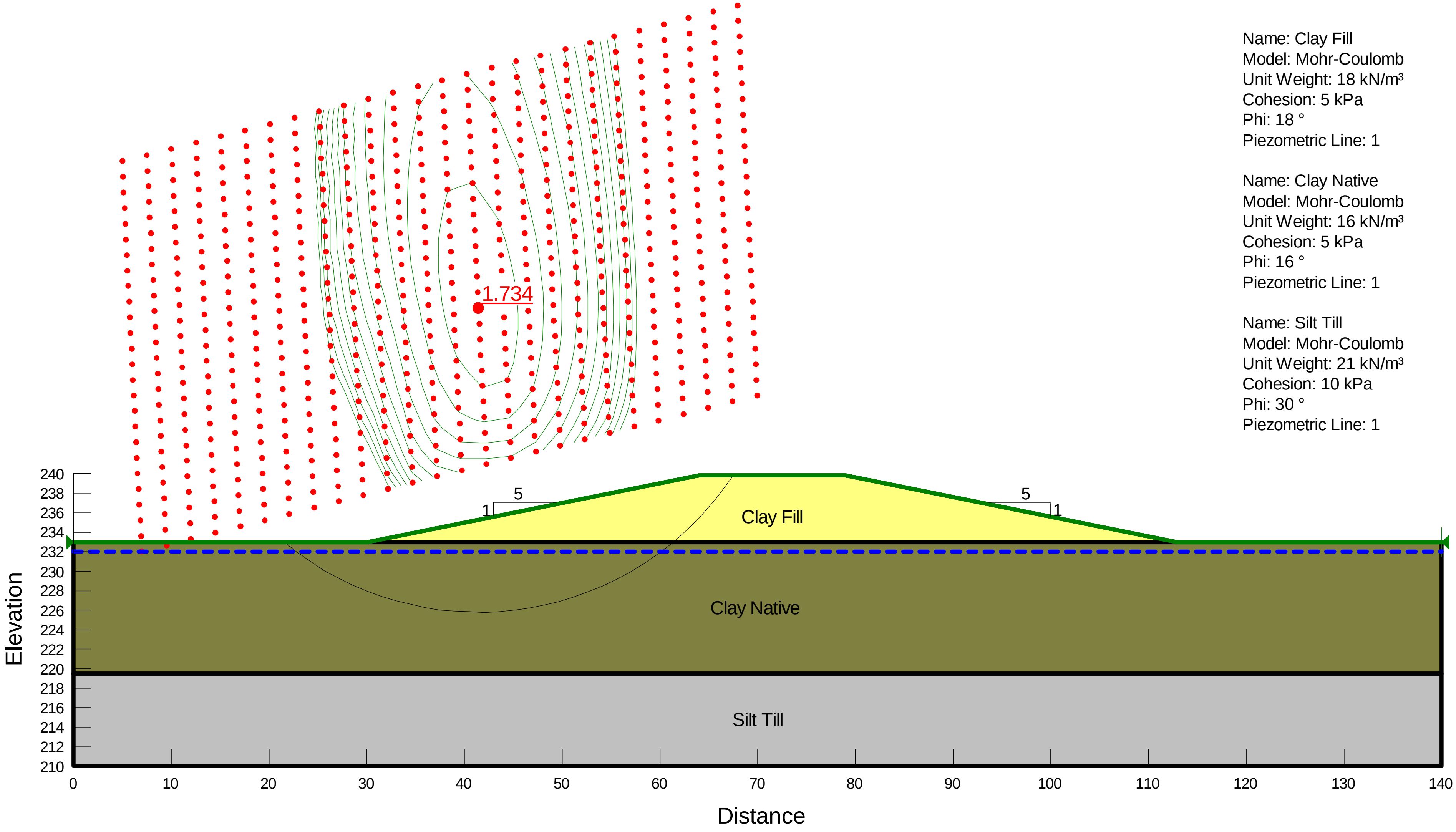


Figure 011

Kenaston Blvd and Bishop Grandin Blvd
West Flyover Embankment - Slope Stability (Long Term)
Side Slope at Max. Embankment Fill



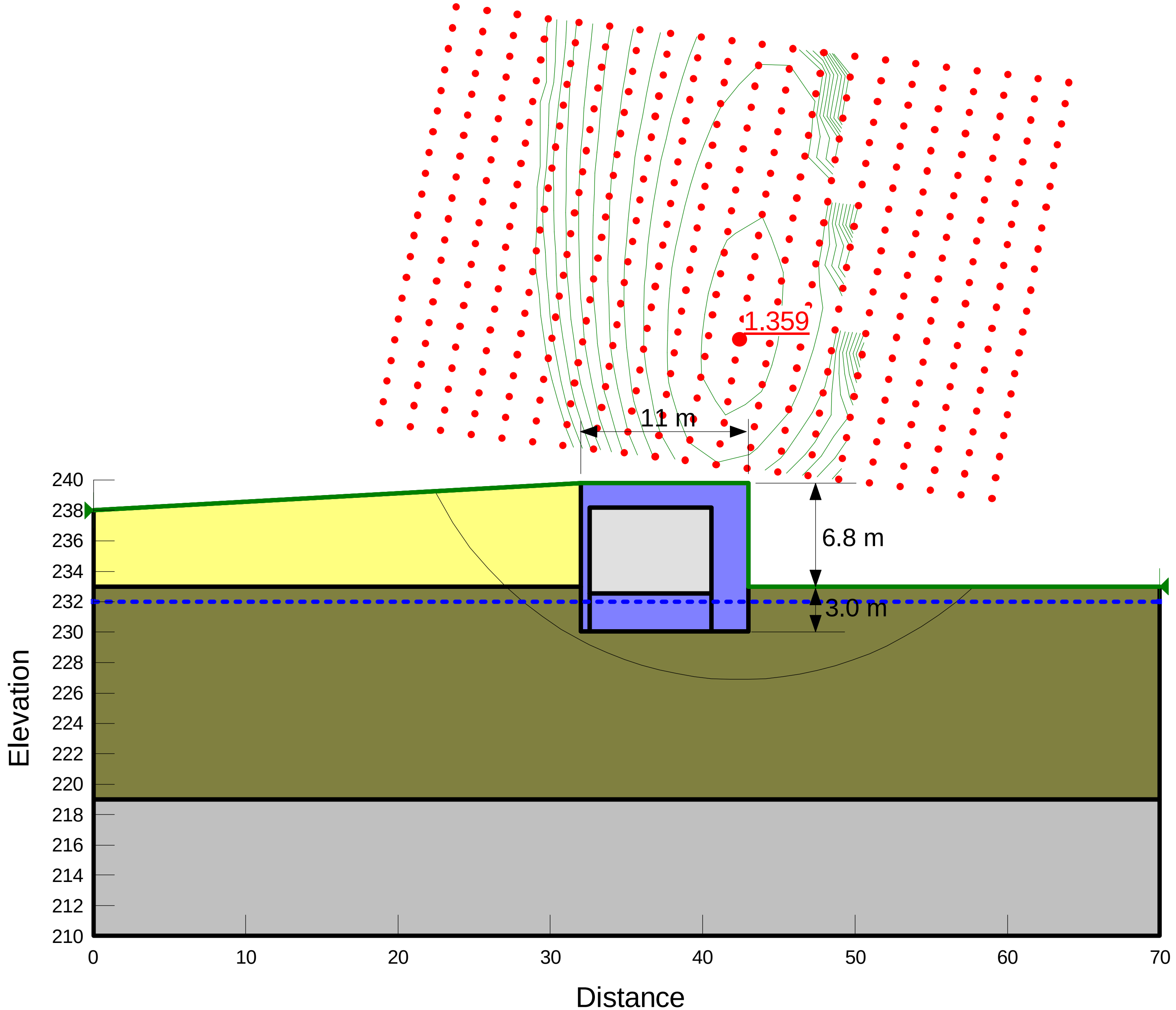
Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Piezometric Line: 1

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Piezometric Line: 1

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Piezometric Line: 1

Figure 012

Kenaston Blvd and Bishop Grandin Blvd
Head Slope Embankment - Slope Stability (Exist. PWP)
Concrete Box Abutment - West Abutment



Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
B-bar: 0

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
B-bar: 0.6
Piezometric Line: 1

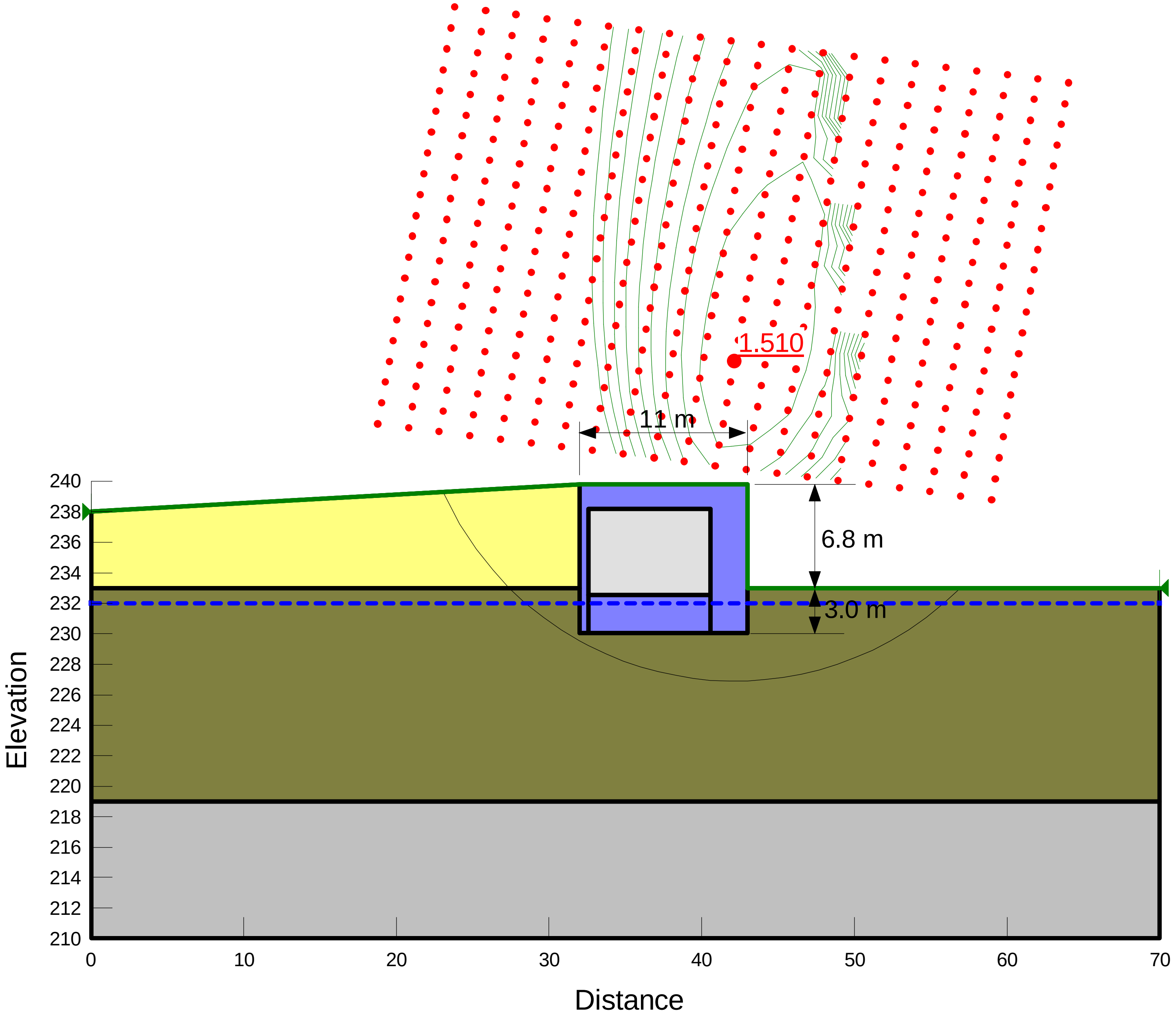
Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Piezometric Line: 1

Name: Concrete Box
Model: Mohr-Coulomb
Unit Weight: 23.5 kN/m³
Cohesion: 100 kPa
Phi: 35 °

Name: Abutment Room
Model: (None)

Figure 013

Kenaston Blvd and Bishop Grandin Blvd
Head Slope Embankment - Slope Stability (Long Term)
Concrete Box Abutment - West Abutment



Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Piezometric Line: 1

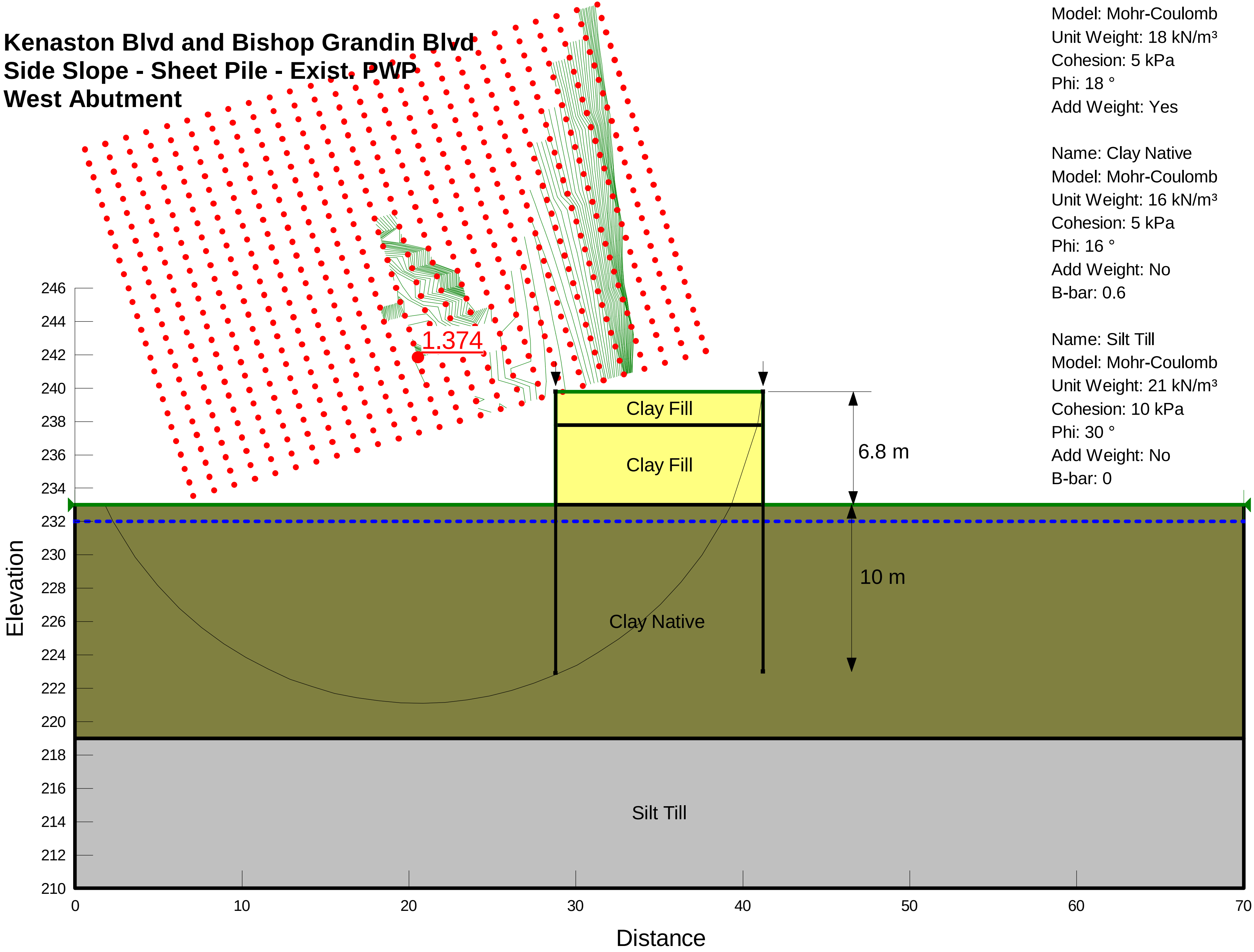
Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Piezometric Line: 1

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Piezometric Line: 1

Name: Concrete Box
Model: Mohr-Coulomb
Unit Weight: 23.5 kN/m³
Cohesion: 100 kPa
Phi: 35 °

Name: Abutment Room
Model: (None)

Figure 014



Kenaston Blvd and Bishop Grandin Blvd
Side Slope - Sheet Pile - Exist. PWP
West Abutment

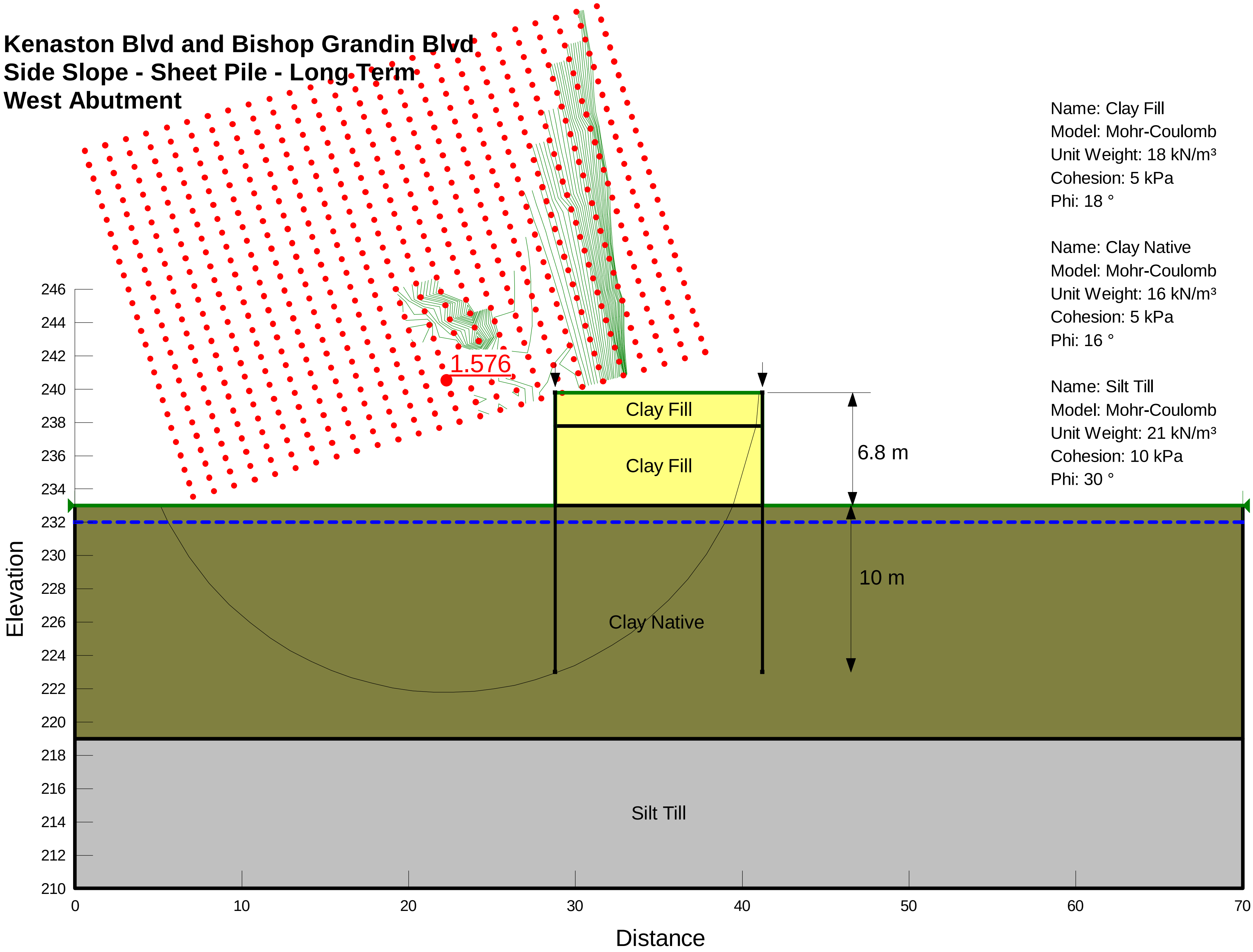
Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °
Add Weight: Yes

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °
Add Weight: No
B-bar: 0.6

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °
Add Weight: No
B-bar: 0

Figure 015

Kenaston Blvd and Bishop Grandin Blvd
Side Slope - Sheet Pile - Long Term
West Abutment



Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

Figure 016

Kenaston Blvd and Bishop Grandin Blvd
East Flyover Embankment - Slope Stability (Long Term) - North Side
Retaining Wall Location - Future Toe Walls

Name: Clay Fill
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °

Name: Clay Native
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion: 5 kPa
Phi: 16 °

Name: Silt Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

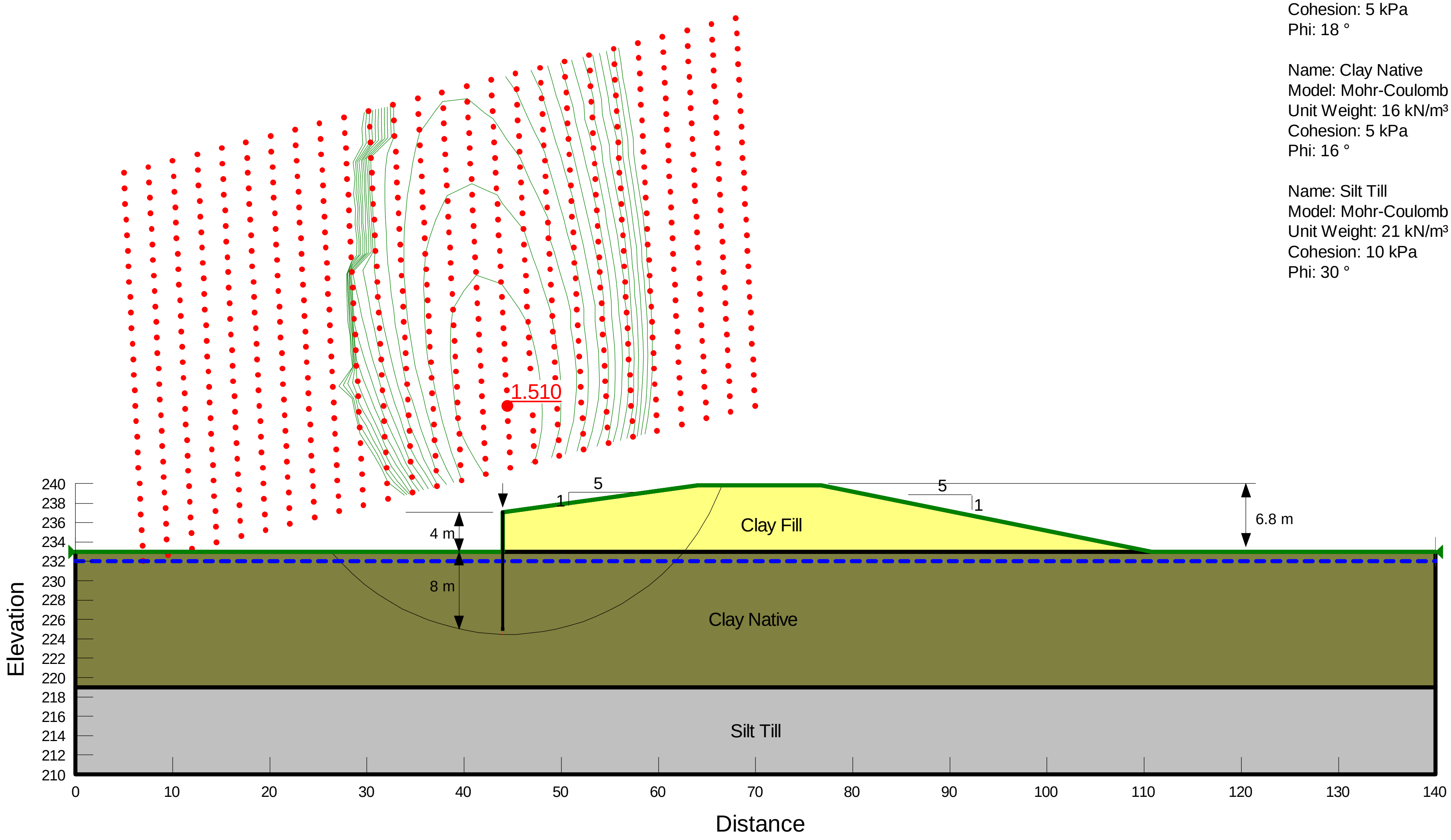


Figure 017

Kenaston Blvd and Bishop Grandin Blvd
West Flyover Embankment - Slope Stability (Long Term) - South Side
Retaining Wall Location - Future Toe Walls

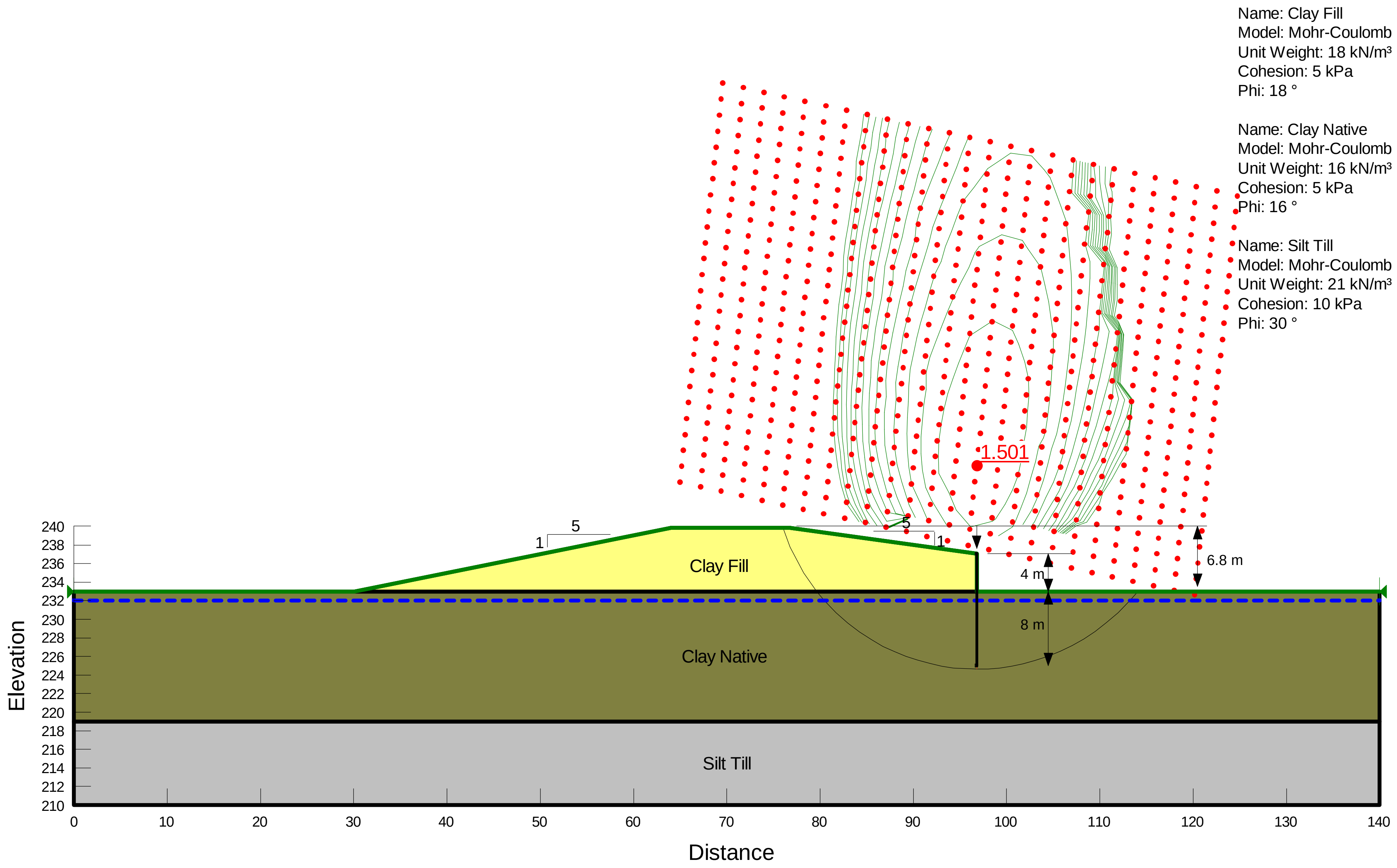


Figure 018

Appendix E

Settlement of the Embankment

Consolidation Settlement Vs. Time - 6.8m Height Embankment
TIME, year
SETTLEMENT, mm
Total Settlement (mm)
0
100
200
300
400
500
600
700
800
0
20
40
60
80
100
0, 0
0.5, 56
1, 79
2, 112
5, 178
10, 251
20, 354
30, 426
40, 478
50, 516
60, 543
70, 563
80, 577
90, 587
100, 595