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City of Winnipeg

Cornish Library Addition
Geotechnical Investigation Report

Prepared for:

Mr. Evan C. Wiebe. C.E.T.
City of Winnipeg
Planning, Property & Development Dept.
4th Floor, 185 King Street
Winnipeg, Manitoba
R3B 1J1

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www.trekgeotechnical.ca
1712 St. James Street | Winnipeg, Manitoba R3H 0L3 | Tel 1.204.975.9433 | Fax 1.204.975.9435

TREK GEOTECHNICAL
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Our File No. 0015 016 00

Mr. Evan C. Wiebe, C.E.T.
City of Winnipeg
4th Floor, 185 King Street
Winnipeg, Manitoba
R3B 1J1

RE: Cornish Library Addition
Geotechnical Investigation Report

TREK Geotechnical Inc. is pleased to submit our Final Report for the Geotechnical Investigation for the above noted project.

Please contact the undersigned if you have any questions. Thank you for the opportunity to work with you on this assignment.

Sincerely,

TREK Geotechnical Inc.
Per:

Ryan Belbas M.Sc., P.Eng
Geotechnical Engineer
Tel: 204.975.9433 ext. 113

www.trekgeotechnical.ca
1712 St. James Street | Winnipeg, Manitoba R3H 0L3 | Tel 1.204.975.9433 | Fax 1.204.975.9435

Revision History

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Authorization Signature

PROVINCE OF MANITOBA
RBelbas
R.E.J.
BELBAS
Member
Feb.1/16
REGISTERED PROFESSIONAL ENGINEER

Prepared By: ______________________

Ryan Belbas, M.Sc., P.Eng.
Geotechnical Engineer

Reviewed By: ______________________

James Blatz, Ph.D., P.Eng., FEC
Senior Geotechnical Engineer / Principal

APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Feb. 1/16

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1.0 Introduction

This report summarizes the results of the geotechnical investigation completed by TREK Geotechnical Inc. (TREK) for the proposed Cornish Library addition at 20 West Gate in Winnipeg, MB. The terms of reference for the investigation are included in our proposal to the City of Winnipeg (The City) dated August 27th, 2015. The scope of work includes a subsurface investigation, laboratory testing, and provision of recommendations for the design and construction of foundations and conceptual riverbank stabilization alternatives. Other considerations such as excavations, floor slabs, site drainage, cement specifications, materials testing and inspection requirements are also included.

2.0 Project Understanding

Based on preliminary information provided by Wolfrom Engineering Ltd. (Wolfrom), TREK understands the addition will be an elevated single storey steel-framed structure above a partial basement. The main floor will be approximately 90 square metres in size. The basement will be approximately 29 square metres in size and will match the existing basement elevation which is approximately 1.8 m below ground surface. Factored foundation loads are anticipated to consist of a single factored point load of 900 kN and four to five points loads ranging between 100 and 150 kN. Renovations to the existing building may require a grade-supported slab in the basement for an elevator or lift.

The Library is situated within the City of Winnipeg Waterways regulated zone between the Primary Dike and the Assiniboine River and therefore building the addition will require a riverbank stability assessment for a permit subject to the requirements of the City of Winnipeg. The Flood Protection Level (FPL) at the site is at Elev. 230.7 m, which is expected to be about 2 m lower than the main floor of the proposed addition.

3.0 Site Conditions

Site visits were conducted in late November and early December of 2015 to assess the general condition of the riverbank in the vicinity of the library and, in particular, to look for visual evidence of riverbank instabilities. The property is located adjacent to and south of the Maryland Bridge at the end of an outside bend of the Assiniboine River which generally flows east towards the Forks. The riverbank is landscaped with grasses and scattered shrubs and trees. A walkway and stairwell is located on the north side of the library which provides pedestrian access underneath the bridge. A brick masonry retaining wall runs along both sides of the walkway and stairwell from the upper bank to the bridge crossing.

The overall slope of the riverbank is about 3.5H:1V (Horizontal:Vertical). Erosion was observed along the toe of the bank which appears to have resulted in a scarp at the south end of the property. No evidence of instabilities upslope was observed within the riverbank, including tension cracks, slumping ground and leaning trees. Riprap is present at the toe of the bank and appears to be part of the bridge armouring; its extents and properties are unknown.

4.0 Subsurface Investigation

4.1 Drilling Program

A subsurface investigation was undertaken in early December of 2015 under the supervision of TREK personnel to evaluate the subsurface conditions at the site. Two test holes (TH15-01 and 02) were drilled using a CME-850 track-mounted drill rig equipped with 125 mm diameter solid stem augers. TH15-01 was drilled in the upper bank within the footprint of the proposed addition and TH15-02 was drilled mid-bank within the bench area. Both test holes were drilled to power auger refusal which occurred at 12.4 m below ground surface in TH15-01 and 6.8 m in TH15-02. A standpipe piezometer with a Casagrande tip was installed in the silt till at a depth of 6.8 m below ground surface (Elev. 220.6 m) in TH15-02 and a vibrating wire piezometer was installed in alluvial clay in an adjacent hole (1 m southeast of TH15-02) to a depth of 4.1 m (Elev. 223.3 m). TH15-01 was backfilled with bentonite and auger cuttings. The standpipe piezometer in TH15-02 was backfilled with sand, bentonite and auger cuttings and the vibrating wire piezometer in the adjacent hole was backfilled with bentonite grout.

Subsurface soils encountered during drilling were visually classified based on the Unified Soil Classification System (USCS). Disturbed (auger cutting and split spoon) and relatively undisturbed (Shelby tube) samples were collected during drilling. All samples retrieved were transported to TREK's testing laboratory in Winnipeg, MB for further classification and laboratory testing. Laboratory testing consisted of water content determination on all samples as well as bulk unit weight measurements and unconfined compressive strength testing on Shelby tube samples. Atterberg limit testing was also completed on a sample in the alluvial clay.

Test hole locations and elevations were surveyed by Wanless Geopoint Solutions Inc. The test hole locations are shown on Figure 01 and the elevations are provided on the test hole logs. The test hole logs provided include a description of the soil units encountered and other pertinent information such as groundwater and sloughing conditions, and a summary of the laboratory testing results.

4.2 Subsurface Conditions

4.2.1 Soil Stratigraphy

A brief description of the soil units encountered at the test hole locations are provided below. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the attached test hole logs.

The soil stratigraphy in the upper bank consists of about 2 m of alternating clay and silt layers (approximately 0.5 m thick) overlying a 9 m thick layer of lacustrine clay, which is underlain by silt till at a depth of 11.0 m below ground surface. The lacustrine clay is highly plastic and stiff to very stiff becoming softer with depth and the silt till is generally loose and wet. The stratigraphy at mid-bank consists of about 3.5 m of clay fill overlying a 2.5 m thick alluvial deposit consisting primarily of clay, which overlays silt till. The lacustrine clay is stiff becoming softer with depth, the alluvial clay is firm becoming softer with depth and the till is loose and wet.

4.2.2 Groundwater Conditions

Groundwater seepage was observed in both test holes from the silt till layer. Shortly after completion of drilling, water levels of 8.8 and 3.7 m were measured below ground surface in TH15-01 and 02, respectively. Sloughing of a thin sand layer (0.5 m thick) was observed in TH15-02 at 3.5 m depth. Both test holes remained open to the depth of exploration after completion of drilling. The piezometers were monitored on January 8, 2016. Readings from the vibrating wire piezometer indicated a groundwater level in alluvial clay lower than the depth of installation. A groundwater level of 3.1 m below ground surface (Elev. 224.2 m) was measured in the standpipe, which is approximately 0.5 m higher than the river level at the time.

The groundwater observations made during drilling are short-term and should not be considered reflective of (static) groundwater levels at the site which would require monitoring over an extended period of time to determine. It is important to recognize that groundwater conditions may vary seasonally, annually, or as a result of construction activities.

5.0 Foundation Recommendations

Based on the subsurface conditions, cast-in-place concrete friction piles and driven concrete end bearing piles are considered to be the most suitable foundation types for this site. Design and construction parameters for these pile types are provided in this section and include axial (compression and uplift) pile capacities in accordance with the National Building Code of Canada (NBCC, 2010). As requested, end bearing caissons were evaluated; however, based on the subsurface conditions, this foundation type was not considered suitable for the site.

5.1 Limit States Design

Limit states design (LSD) recommendations for deep foundations in accordance with the National Building Code of Canada (NBCC, 2010) are provided below. Limit states design requires consideration of distinct loading scenarios comparing the structural loads to the foundation bearing capacity using resistance and load factors that are based on reliability criteria. Two general design scenarios are evaluated corresponding to the serviceability and ultimate capacity requirements.

The **Ultimate Limit State (ULS)** is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation units. The ULS foundation bearing capacity is obtained by multiplying the nominal (ultimate) bearing capacity by a resistance factor (reduction factor), which is then compared to the factored (increased) structural loads. The ULS bearing capacity must be greater than or equal to the maximum factored load. Table 1 summarizes the resistance factors (ϕ) that can be used for the design of foundations as per the National Building Code of Canada (NBCC, 2010). The values of the resistance factors depend on the method of analysis and verification testing completed during construction.

The **Service Limit State (SLS)** is concerned with limiting deformation or settlement of the foundation under service loading conditions such that the integrity of the structure will not be impacted. The SLS should generally be analysed by calculating the settlement resulting from applied service loads and comparing this to the settlement tolerance of the structure. However, the settlement tolerance of the structure is typically not defined at the preliminary design stage. As such, SLS bearing capacities are often provided that are developed on the basis of limiting settlement to 25 mm or less. A more detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust the SLS pile capacity if a more stringent settlement tolerance is required or if large groups of piles are used.

Table 1. ULS Resistance Factors for Deep Foundations (NBCC, 2010)

Bearing Resistance to Axial Load for Deep Foundations (Analysis Methods)	Resistance Factor (ϕ)
Semi-empirical analysis using laboratory and *in situ* test data	0.4
Analysis using dynamic monitoring results (PDA testing and CAPWAP analysis)	0.5
Analysis using static loading test results	0.6
Uplift resistance by semi-empirical analysis.	0.3
Uplift resistance using loading test results.	0.4

5.2 Cast-in-Place Concrete Friction Piles

Cast-in-place concrete (CIPC) friction piles will derive a majority of their resistance in shaft friction (adhesion) with a relatively small contribution from end bearing. Table 2 provides the recommended ULS and SLS axial (compressive and uplift) unit resistances for shaft adhesion and end bearing. Pile settlements are expected to be less than 10 mm at the pile tip (bottom of pile). The elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement.

Table 2. Recommended ULS and SLS Resistances for CIPC Friction Piles

Pile Depth Below Ground Surface (m)	Geodetic Elevation (m)	ULS Axial Unit Resistance (kPa)			SLS Axial-Compressive Unit Resistance Shaft Adhesion (kPa)
		Compression $\phi = 0.4$		Uplift $\phi = 0.3$	
		Shaft Adhesion	End Bearing	Shaft Adhesion	
0 to 1.5	232.8 to 231.3	0	0	0	0
1.5 to 10	231.3 to 222.8	16	70	12	16

<u>CIPC Friction Pile Design Recommendations:</u>

1. The weight of the embedded portion of the pile may be neglected.
2. The piles should have a minimum shaft diameter of 406 mm.
3. For piles supporting heated structures (excluding perimeter piles), shaft adhesion in compression and uplift within the upper 1.5 m below ground surface should be neglected from design. For piles subjected to freezing conditions (including perimeter piles), shaft adhesion in compression and uplift within the upper 2.5 m below ground surface should be neglected.
4. Pile lengths should be limited to a depth of 10 m below existing ground surface (Elev. 222.8 m) to avoid penetrating the silt till and to protect against heaving at the base of the pile shaft.
5. Piles should have a minimum spacing of 3 pile diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
6. Piles require steel reinforcement designed by a qualified structural engineer for the anticipated axial (compression and uplift), lateral and bending loads induced from the structure.

<u>CIPC Friction Pile Installation Recommendations:</u>

1. Temporary steel casings (*i.e.* sleeves) should be available and used if sloughing of the pile hole occurs and/or to control groundwater seepage if encountered. Care should be taken in removing sleeves to prevent sloughing (necking) of the shaft walls and a reduction in the cross-sectional area of the pile.
2. Concrete should be placed in one continuous operation immediately after the completion of drilling the pile hole to avoid construction problems such as sloughing or caving of the pile hole and groundwater seepage. Concrete should be poured under dry conditions. If groundwater is encountered it should be controlled and removed. If water cannot be controlled and removed, the concrete should be placed using tremie methods.
3. Concrete placed by free-fall methods should be directed through the middle of the pile shaft and steel reinforcing cage to prevent striking of the drilled shaft walls to protect against soil contamination of the concrete.

5.3 Driven Precast Prestressed Concrete Hexagonal Piles

Precast prestressed concrete hexagonal (PPCH) piles driven to practical refusal will derive a majority of their capacity in end bearing with a relatively small contribution from shaft adhesion. The recommended SLS and factored ULS axial (compressive) capacities for PPCH piles driven to practical refusal are provided in Table 3. Pile settlements are expected to be less than 10 mm at the pile tip (bottom of pile). The elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement.

Table 3. Recommended ULS and SLS Capacities for Driven PPCH Piles

Pile Size (mm)	Refusal Criteria (Blows/25mm)	ULS Axial-Compressive Capacity (kN)			SLS Axial-Compressive Capacity (kN)
		$\phi = 0.4$	$\phi = 0.5$	$\phi = 0.6$	
305	5	550	690	825	445
356	8	770	965	1,155	625
406	12	990	1,240	1,485	800

The piles should be driven to at least three consecutive sets of the refusal criteria outlined in Table 3, using a diesel hammer having a minimum rated energy of 40 kJ or a hydraulic drop hammer having a minimum rated energy of 20 kJ.

Power auger refusal is often a good indicator of practical refusal depth for this type of driven pile; however, due to small diameter of the auger used to drill the test holes and the inherently variable conditions of the silt till underlying Winnipeg, the depth to practical refusal of the pile should be expected to vary across the site and may be deeper than encountered during drilling and as indicated on the TH logs.

<u>Driven PPCH Pile Design Recommendations:</u>

1. The weight of the embedded portion of the pile may be neglected.
2. Pile spacing should not be less than 2.5 pile diameters centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
3. Pre-boring should be completed to reduce ground vibrations and protect against heave of, and consequently damage to, the existing building. Pre-boring also contributes to maintaining verticality and alignment of the piles. Pre-bore diameter should be no more than 50 mm larger than the pile diameter. A typical pre-bore depth is 3 m; however, pre-bore depth should be increased to at least 6 m below the base of existing structures (grade beams) for piles to be driven directly adjacent to existing structures.
4. A factored ULS shaft adhesion of 12 kPa (a geotechnical resistance factor of 0.3 has been applied to this value) can be used to design for uplift resistance for piles in clay and till. The entire pre-

bore length should be neglected from uplift resistance. It should be noted that uplift loads will also be resisted by structural dead loads.

5. Piles should be designed by a qualified structural engineer to withstand design loads, handling stresses, driving stresses, and tensile forces induced from seasonal movements (*e.g.* frost-related movements) of the bearing soils.

Driven PPCH Pile Installation Recommendations

1. The pile-driving hammer should have the capability of adjusting the delivered energy to operate at higher settings during driving if the developed energy is not sufficient to mobilize the ultimate pile capacity. The driving system should also have the capability of adjusting the developed energy to operate at lower settings during easy driving and to prevent pile damage upon sudden pile refusal.
2. The pile-driving hammer should be equipped with a pile cushion to protect the pile head from damage during driving from direct impact with the steel driving helmet. The pile cushion should consist of a minimum of 100 mm of compressible material such as plywood or hardwood (*e.g.* oak). The pile cushion should fit tightly inside the pile helmet.
3. The piles should be cured for at least 7 days prior to driving.
4. Piles should be driven continuously once driving is initiated to the required refusal criteria.
5. Where a steel follower is required to install piles below the ground surface, the refusal criteria should be increased by 50% in order to account for additional energy losses through the use of the follower.
6. Re-driving of all piles in groups should be specified along with the requirement to monitor for pile heave. All piles exhibiting heave of 6 mm or more should be re-driven to a minimum of one set of the practical refusal criteria.
7. Pile verticality (plumbness) should be measured on all piles with adequate stick-up after practical refusal has been achieved to check if verticality is within the limits of the structural design. It is common local practice to specify a maximum acceptable percentage that the pile can be out of vertical plumbness (*e.g.* 2% out of plumb).
8. Any piles damaged, out of plumb an excessive amount, or reaching premature refusal may need to be replaced. The structural designer will have to assess non-conforming piles to determine if they are acceptable. PDA testing with CAPWAP analysis is recommended for any piles that are suspected to not meet the design capacity or to be damaged if a structural solution is not possible.

5.4 Lateral Pile Analysis

Soil response to lateral loads can be modeled in a simplified manner that assumes the soil around a pile can be simulated by a series of horizontal springs (subgrade reaction) for preliminary design of pile foundations. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral subgrade reaction modulus (K_s) as provided in Table 4. The majority of lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the relative stiffness of the pile and soil units. Void spaces surrounding piles due to pre-boring activities should be in-filled with lean-mix concrete to ensure compliance with the surrounding soil. If in-filling is not completed, the depth of the pre-bore should be neglected from lateral pile resistance calculations.

Table 4. Recommended Values for Lateral Subgrade Reaction Modulus (K_s)

Depth Below Final Grade (m)	Approximate Geodectic Elevation (m)	K_s (kN/m^3)
0 to 1.5	232.8 to 231.3	0
1.5 to 11.0	231.3 to 221.8	3,300/d*
> 11.0	< 221.8	14,000/d*

** d = pile diameter*

It should be understood that using the lateral subgrade reaction modulus assumes a linear response to lateral loading and therefore is only appropriate under the following conditions:

- maximum pile deflections are small (less than 1% of the pile diameter),
- loading is static (no dynamic cycling), and
- pile material behaviour is confirmed to be within linear elastic limits by the structural engineer.

If one or more of these conditions are not met, a more rigorous analysis that includes non-linear behavior of the piles and surrounding soil is required. In this regard, as part of detailed design, a lateral pile analysis that incorporates the material and section properties of the piles, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading should be carried out by TREK once the final design grades are determined to confirm the lateral load capacity of the piles, which is not part of our current scope of work.

5.5 Ad-freezing Effects

Piles subjected to freezing conditions should be designed to resist ad-freezing and uplift forces related to frost action acting along the vertical faces of the pile and pile cap within the depth of frost penetration (2.5 m below ground surface). In this regard, piles may be subject to an ad-freeze bond stress of 65 kPa within the depth of frost penetration. These forces will be resisted by structural dead loads and uplift resistance provided by the length of the pile below the depth of frost penetration (and pre-bore). Alternatively, measures such as flat lying rigid polystyrene insulation could be incorporated into the design to reduce frost penetration depths and thereby ad-freezing effects and uplift forces.

5.6 Grade Beams and Pile Caps

A minimum void of 150 mm underneath all grade beams and pile caps should be provided to avoid uplift pressures from developing on the underside of the pile cap as a result of swelling or frost action. The void can consist of a compressible layer (*e.g.* low density polystyrene) to permit sub-grade soil movements without engaging the grade beams or piles caps. Excavations for grade beams and pile caps should be backfilled with a non-frost susceptible soils such as sand compacted to a minimum of 95% of the Standard Proctor Maximum Dry Density (SPMDD).

5.7 Foundation Concrete

All foundation concrete should be designed by a qualified structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads from the structure. Based on local experience gathered through previous work in Winnipeg, the degree of exposure for concrete subjected to sulphate attack is classified as severe according to Table 3, CSA A23.1-09 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56-day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-09 for concrete with severe sulphate exposure (S2). Concrete that may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-09.

5.8 Foundation Inspection Requirements

In accordance with Section *4.2.2.3 Field Review* of the NBCC (2010), the designer or other suitably qualified person shall carry out a field review on:

1. a continuous basis during:
    i. the construction of all deep foundation units,
    ii. the installation and removal of retaining structures and related backfilling operations, and
    iii. during the placement of engineered fills.
2. on an as-required basis for the construction of shallow foundation units and in excavating, dewatering and other related works.

In consideration of the above and relative to this particular project, we recommend that TREK, as the geotechnical engineer of record, be retained to inspect the installation of any foundation elements. TREK is familiar with the geotechnical conditions and the basis for the foundation recommendations and can provide any design modifications deemed to be necessary should altered subsurface conditions be encountered.

6.0 Floor Slabs

6.1 Grade-supported Floor Slabs

If some movement of floor slabs can be tolerated, a grade-supported floor slab can be used. Vertical movements due to seasonal moisture and volume changes of the underlying clay soils should be expected. Although difficult to predict, these movements could be in the order of 50 mm. In this regard, the floor slab should be designed to accommodate these movements. Additionally, floor slabs in unheated areas will be subject to additional movements from freeze/thaw of the sub-grade soils.

Additional Recommendations of design and Construction of Grade Supported Floors:

1. For best long-term performance, organics, fill materials, silt and any other deleterious material should be excavated such that the sub-grade consists of native stiff silty clay.
2. Excavation should be completed with an excavator equipped with a smooth bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed sub-grade.
3. TREK personnel should inspect the final sub-grade after excavation and prior to concrete placement to verify the adequacy of the sub-grade. Areas that contain silt or are soft should be repaired as per directions provided by TREK.
4. The sub-grade should be protected from freezing, drying, or inundation with water at all times. If any of these conditions occur, the sub-grade should be scarified, moisture conditioned as appropriate, and re-compacted to a minimum of 95% of the SPMDD.
5. In heated areas, the floor slab should be placed on a 150 mm thick base layer consisting of 20 mm down crushed rock fill underlain by a 150 mm thick granular sub-base layer constructed of a 50 mm down crushed rock fill. In unheated areas the thickness of the sub-base layer should be increased to 250 mm. The granular fill should be placed in lifts no greater than 150 mm and compacted to 98% of the SPMDD.
6. All granular materials should be well-graded and well-draining.
7. A vapour barrier should be placed above the granular base and beneath the floor slab.
8. Floor slabs should be designed to resist all structural loads and to minimize slab cracking associated with movements as a result of swelling, shrinkage, and thermal expansion and contraction of the sub-grade soils.

6.2 Structural Floor Slabs

If floor slabs cannot tolerate movements that are typically associated with grade supported floor slabs, a structural floor slab will be required. A minimum void of 150 mm is recommended beneath the structural floor slab to accommodate volumetric changes in the underlying sub-grade soils. The void can consist of a compressible layer (*e.g.* low density polystyrene) to permit sub-grade soil movements without engaging the floor slab or alternatively a crawl space. A vapour barrier below the slab is also recommended to minimize long-term moisture changes within the sub-grade soils.

7.0 Excavations and Temporary Shoring

Excavations must be carried out in compliance with the appropriate regulations under the Manitoba Workplace Safety and Health Act. It is anticipated that short term stability can be maintained for open-cut excavations with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Any open-cut excavation greater than 3 m deep must be designed and sealed by a professional engineer and should be reviewed by the geotechnical engineer of record (TREK). Furthermore, maintaining the stability of the excavation slopes for the duration of construction should be the responsibility of the

Contractor. To prevent wetting or drying of the exposed excavation side slopes, they should be protected with plastic covering or similar measures. Stockpiles of excavated material and heavy equipment should be kept away from the edge of the excavation by a distance equal to or greater than the depth of excavation.

Dewatering measures should be completed as necessary to maintain a dry excavation and permit proper completion of the work. If seepage is encountered, it should be directed to a sump pit and pumped out of the excavation. If saturated silts are encountered, shoring or slope flattening may be required. Gravel buttressing could be used in conjunction with sump pits for dewatering to prevent wet silt soils from entering the excavation,. Surface water should be diverted away from the excavation and the excavation should be backfilled as soon as possible following construction.

Cantilevered (un-braced or braced) walls will be required for deep excavations where temporary shoring is necessary. Table 5 provides the recommended earth pressure coefficients and bulk unit weights of the soils for calculation of lateral earth pressures. Surcharge loads and hydrostatic water pressure should be incorporated into the design of cantilevered walls, as well as an adequate factor of safety against instability.

Table 5. Recommended Design Parameters for Cantilevered Walls

Design Parameter	Earth Pressure Coefficients and Bulk Unit Weights
	Silt / Clay
Active (K_a)	0.5
At-rest (K_o)	0.7
Passive (K_p)	1.8
Bulk Unit Weight, Υ (kN/m^3)	17.0

A certain amount of ground movement behind the shoring will occur and is largely unavoidable. The amount of movement that will occur cannot be accurately predicted, mainly because the movement is as much a function of installation procedures and workmanship as it is a function of soil mechanics considerations. It is anticipated that the design of temporary shoring will be the responsibility of the Contractor. Once the proposed shoring design is complete, it should be reviewed by TREK prior to construction to ensure the design is appropriate and to assess the need for groundwater control. Performance of the excavation system should be monitored from the onset of installation to removal of the shoring system.

8.0 Site Drainage

Positive site drainage adjacent to the addition should promote runoff away from the structure. A minimum gradient of about 2% for landscaped areas should be provided and maintained throughout the life of the structure. Water discharge from roof leaders should be directed away from the structure.

9.0 Riverbank Stability

9.1 Design Objectives

New structures situated on riverbanks within the City of Winnipeg Waterways regulated zone are typically designed based on achieving factors of safety of 1.5 under normal (long-term) groundwater and river level conditions and 1.3 under extreme (short-term) conditions, and such that the potential slip surface at these target factors of safety do not infringe upon the structure. Slope stability analyses were performed to assess the existing riverbank stability to determine if stabilization measures are required to establish a development setback for the proposed addition based on the above design criteria.

9.2 Numerical Model

The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2012 software package (Geo-Slope International Ltd.). The slope stability model used the Morgenstern-Price method of slices with the half-sine inter-slice force function to calculate factors of safety (FS) and slip surfaces were identified using a grid and radius slip surface method. A static piezometric line was used to represent groundwater and river water levels since significant seepage gradients were not observed between the till and clay layers based on the piezometer readings. The piezometric line is incorporated into the model such that it transitions gradually from the upper bank down to the river level.

9.3 Riverbank Geometry and Soil Properties

A topographic survey of the property and bathymetry of the riverbed was completed and provided by Wanless Geopoint Solutions Inc. Cross sections of the bank and riverbed (A and B) were developed through the property and Cross Section A was used for stability analyses. The locations of the cross-sections are shown in plan view on Figure 01 and the cross-sections are shown in Figure 02.

The soil units used in the model are based on the stratigraphy encountered during drilling and the site survey. The soils units include clay fill, sand, alluvial clay, lacustrine clay and silt till. Table 6 summarizes the soil properties used in the slope stability analyses for each soil unit. The clay fill and sand are considered representative of a stiff to very stiff clay and loose silty sand, respectively. The silt till layer was considered to be impenetrable and the strength properties assumed for the alluvial and lacustrine soils are considered typical for Winnipeg clays along slopes which have experienced large strains.

Table 6. Soil Properties used in the Stability Analysis

Soil Description	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (degrees)
Clay Fill	18	1	20
Sand	19	0	25
Alluvial Clay	18	2	23
Lacustrine Clay	17	5	17
Silt Till	Impenetrable		
Riprap	21	0	40

9.4 Groundwater and River Levels

The groundwater and river levels used in the analyses to represent the long and short-term design scenarios are summarized in Table 7. River levels are based on the average recorded levels at the Maryland Bridge and include the Regular Summer River Level (RSRL) and the Regulated Winter River Level (RWRL). Groundwater levels are based on assumed values in the upper bank. In consideration of groundwater levels measured in the piezometers, the assumed summer levels govern design and provide the basis of long-term conditions.

Table 7. Groundwater and River Levels used in the Stability Analysis

Design Scenario	Groundwater Level Below Ground Surface in Upper Bank (m)	Assiniboine River Elev. (m)	Factor of Safety Design Criteria
Short-Term (Extreme)	2.0	223.0 (RWRL)	1.3
Long-Term (Normal) Summer	2.5	224.0 (RSRL)	1.5

The long-term design scenario is considered to consist of a summer groundwater level in the upper bank transitioning down to the river level which is located at RSRL. The short-term case consists of a high groundwater level in the upper bank and low river level at RWRL which represents a scenario that would result in reduced stability throughout the bank, however would not exist for an extended period of time.

9.5 Stability Analysis Results

The existing stability of the riverbank was first assessed to determine if stabilization measures are required to achieve a development setback that accommodates the proposed addition, without modifying the design, for both the long-term and short-term design scenarios. The results indicate that stabilization measures are required to achieve the design objective. As such, stabilization measures were incorporated into the analysis to improve stability. It must be understood that the stabilization measures presented here are conceptual and should only be used for preliminary design purposes. TREK should be retained during the design phase to refine riprap and grading geometries and extents.

The following stability cases were analyzed to assess the existing riverbank stability and determine conceptual stabilization alternatives that can be implemented to achieve the design objective:

- Existing conditions of the riverbank under long-term conditions
- Riprap placement at riverbank toe (approx. 1 m thick) under long-term conditions
- Re-grading the upper bank to 5H:1V under long-term conditions
- Riprap placement at riverbank toe (approx. 1 m thick) and re-grading the upper bank (5H:1V) under long-term conditions
- Riprap placement at riverbank toe (approx. 1 m thick) and re-grading the upper bank (5H:1V) under short-term conditions

A summary of the analysis results of each case are provided in Table 8 and includes the design scenario and stabilization measure analyzed, the calculated critical factors of safety in the lower and upper bank, and the impact of the development setback on the proposed addition. Stability analysis outputs for each case analyzed are provided in Appendix B.

Table 8. Summary of the Stability Analysis Results

Design Scenario	Stabilization Measure	Critical FS		Does Proposed Addition Meet Required Development Setback
		Lower Bank	Upper Bank	
Long-Term (Target FS for Development Setback = 1.5)	Existing Conditions	1.16	1.35	NO
	Riprap at Riverbank Toe	n/a	1.45	NO
	Re-grading Upper Bank	1.16	1.47	NO
	Riprap at Riverbank Toe and Re-grading Upper Bank	1.50	1.58	YES
Short-Term (Target FS for Development Setback = 1.3)	Riprap at Riverbank Toe and Re-grading Upper Bank	1.31	1.44	YES

Based on the results of the stability analysis, a development setback that accommodates the proposed addition can be achieved by implementing riprap at the riverbank toe and re-grading the upper bank. It should be noted that the stabilization works (riprap and re-grading) improve the critical FS (with respect to the existing FS) in the lower riverbank by approximately 35 and 15% for the long-term and short-term conditions, respectively. The critical FS in the upper bank is improved by 23 and 9% for the long-term and short-term conditions, respectively. It is important to note that riprap is not only required to improve stability but also to provide protection against erosion along the shoreline which will result in loss of riverbank soils and a reduction in stability of the riverbank over time.

10.0 Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of the City of Winnipeg (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Figures

0015 016 00
City of Winnipeg
Cornish Library Addition

Tabloid (279mm x 432mm)

PLOT: 1/20/2016 4:50:04 PM

FILE NAME: FIG 001 2016-01-20 Site Plan 0_F_HA 0015 016 00.dwg

0 5 10 15 20m
SCALE : 1:500 (279mm x 432mm)

LEGEND :

⌖ TEST HOLE (TREK, DECEMBER 3 AND 4, 2015)

NOTES :

1. AERIAL IMAGE FROM CITY OF WINNIPEG 2013

Figure 01
Site Plan

TREK
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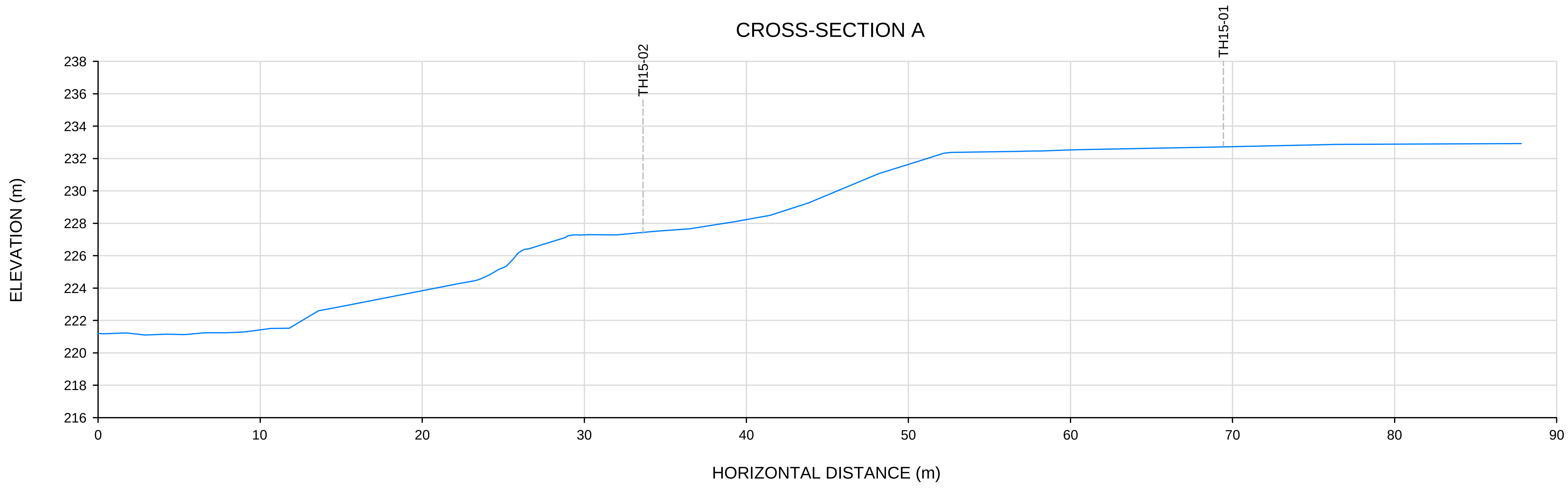
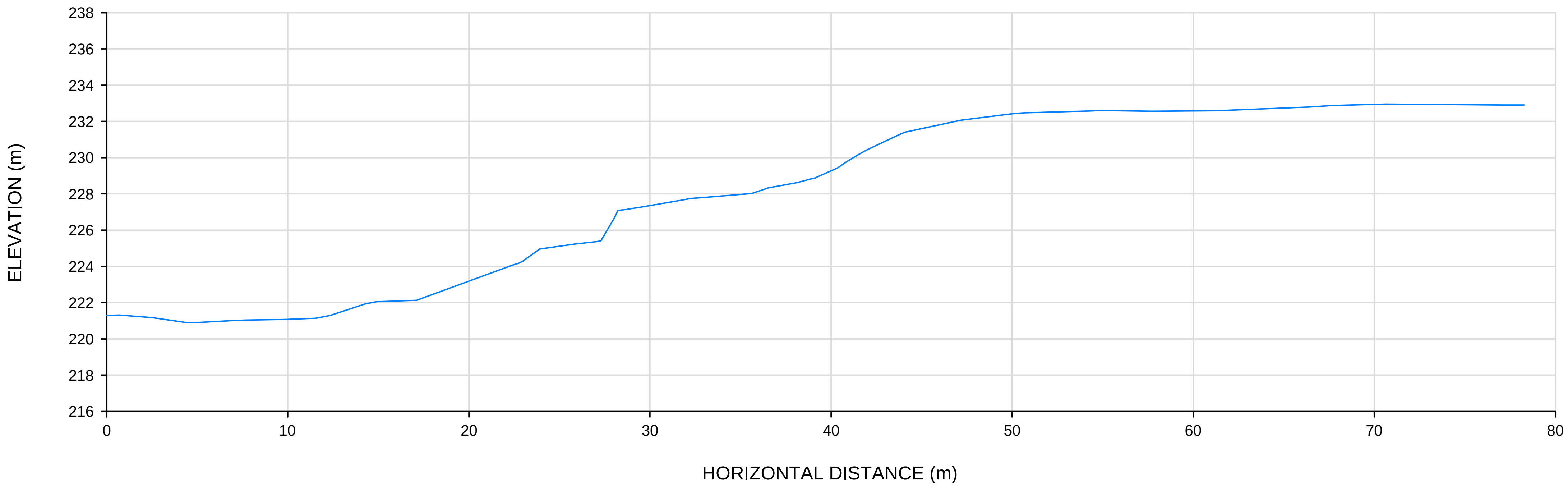
0015 016 00
City of Winnipeg
Cornish Library Addition

Tabloid (279mm x 432mm)

PLOT: 1/20/2016 4:50:47 PM

FILE NAME: FIG 001 2016-01-20 Site Plan 0_F_HA 0015 016 00.dwg


CROSS-SECTION A


CROSS-SECTION B

0 3 6 9 12m
SCALE : 1:300 (279mm x 432mm)

Figure 02
Cross Sections A and B

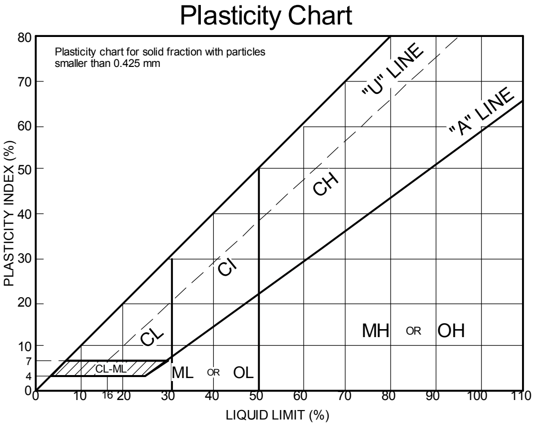
Test Hole Logs

TREK GEOTECHNICAL

EXPLANATION OF FIELD AND LABORATORY TESTING

GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. Descriptions on these test hole logs apply only at the specific test hole locations and at the time the test holes were drilled. Variability of soil and groundwater conditions may exist between test hole locations.
3. When the following classification terms are used in this report or test hole logs, the primary and secondary soil fractions may be visually estimated.

Major Divisions			USCS Classi-fication	Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW	●	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent...... GW, GP, SW, SP More than 12 percent...... GM, GC, SM, SC 6 to 12 percent...... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	○	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM	◌	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC	▧	Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than 4.75 mm)	Clean sands (Little or no fines)	SW	⁘	Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	⠒	Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM	▥	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC	▨	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML	▥	Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity			
			CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	☰	Organic silts and organic silty clays of low plasticity			
	Silts and Clays (Liquid limit greater than 50)		MH	▥	Inorganic silts, micaceous or distomaceous fine sandy or silty soils, organic silts			
			CH	▨	Inorganic clays of high plasticity, fat clays			
			OH	≋	Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt	⩊	Peat and other highly organic soils	Von Post Classification Limit	Strong colour or odour, and often fibrous texture	

Material		Particle Size mm	Particle Size ASTM Sieve sizes
Sand	Coarse	2.00 to 4.75	#10 to #4
	Medium	0.425 to 2.00	#40 to #10
	Fine	0.075 to 0.425	#200 to #40
Silt or Clay		< 0.075	< #200

Material		Particle Size mm	Particle Size ASTM Sieve Sizes
Boulders		> 300	> 12 in.
Cobbles		75 to 300	3 in. to 12 in.
Gravel	Coarse	19 to 75	3/4 in. to 3 in.
	Fine	4.75 to 19	#4 to 3/4 in.

* Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols. For example; GW-GC, well-graded gravel-sand mixture with clay binder.

Other Symbol Types

■	Asphalt	▧	Bedrock (undifferentiated)	○	Cobbles
⁙	Concrete	▦	Limestone Bedrock	●	Boulders and Cobbles
▩	Fill	☰	Cemented Shale	▥	Silt Till
		▤	Non-Cemented Shale	▨	Clay Till

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

⧨ Water Level After Drilling as Indicated on Test Hole Logs

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

TERM	EXAMPLES	PERCENTAGE
and	and CLAY	35 to 50 percent
"y" or "ey"	clayey, silty	20 to 35 percent
some	some silt	10 to 20 percent
trace	trace gravel	1 to 10 percent

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

The Standard Penetration Test blow count (N) of a non-cohesive soil can be related to compactness condition as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

The Standard Penetration Test blow count (N) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very soft	< 2
Soft	2 to 4
Firm	4 to 8
Stiff	8 to 15
Very stiff	15 to 30
Hard	> 30

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

Descriptive Terms	Undrained Shear Strength (kPa)
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

Test Hole TH15-01

1 of 2

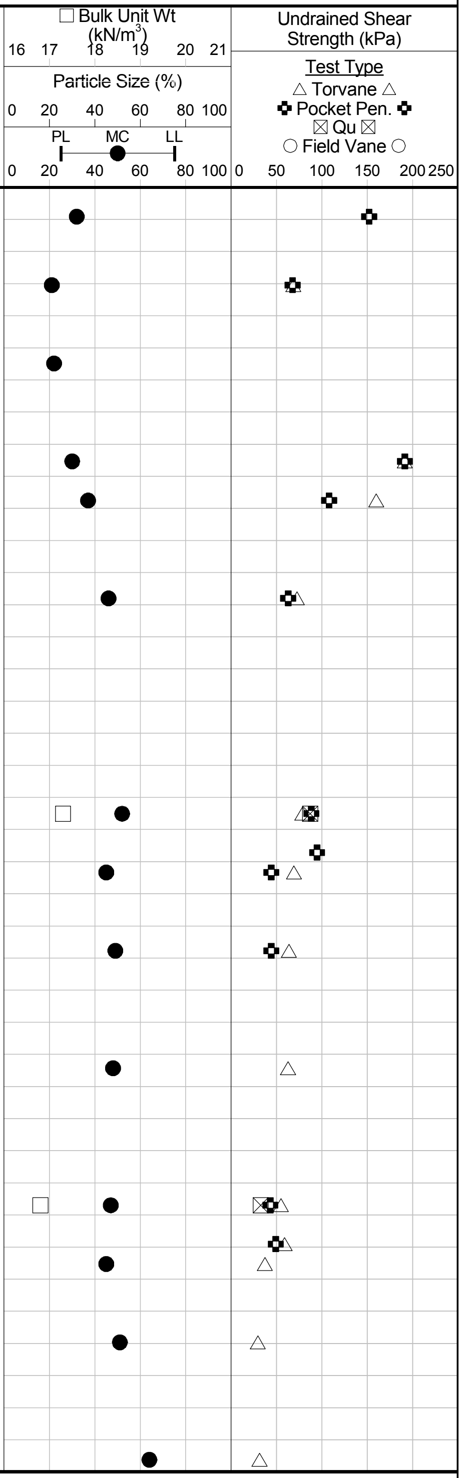
Sub-Surface Log

Client:	City of Winnipeg	**Project Number:**	0015 016 00
Project Name:	Cornish Library Addition	**Location:**	UTM N-5526730.85, E-632182.25
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	232.78 m
Method:	125 mm Solid Stem Auger, CME-850 Track Mount	**Date Drilled:**	3 December 2015

Sample Type: ◢ Grab (G) ▐▌ Shelby Tube (T) ☒ Split Spoon (SS) ⧗ Split Barrel (SB) ▐ Core (C)

Particle Size Legend: ▦ Fines ▨ Clay ▥ Silt ░ Sand ● Gravel ▢ Cobbles ■ Boulders

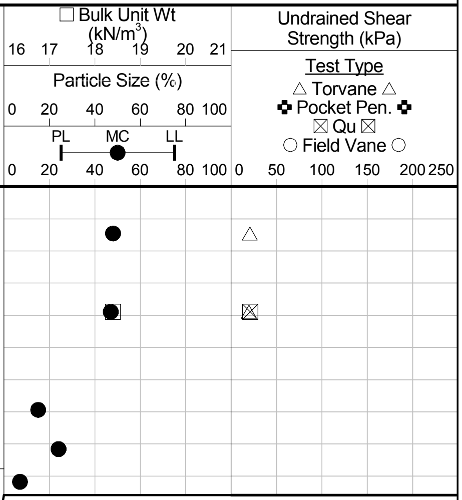
Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
		CLAY - silty, dark grey, moist, very stiff, high plasticity - frozen to 0.1 m	◢	G-01	
232.3	0.5	SILT - some clay to clayey - light brown, moist, stiff, low plasticity	◢	G-02	
231.7	1.0	CLAY - silty, sandy, trace gravel (<10 mm) - brown, dry to moist, stiff, intermediate plasticity - some organics (roots) below 1.4 m	◢	G-03	
231.3	1.5	SILT - clayey - light brown, moist, stiff, low plasticity			
230.8	2.0	CLAY (LACUSTRINE) - silty, trace silt inclusions (<5 mm diam.) - mottled brown and grey, moist, very stiff, high plasticity	◢	G-04	
	2.5		◢	G-05	
	3.0	- stiff below 3.1 m - trace precipitates from 3.1 to 4.0 m	◢	G-06	
	3.5 4.0 4.5				
	5.0		▐▌	T-07	
	5.5		◢	G-08	
	6.0		◢	G-09	
	6.5 7.0		◢	G-10	
	7.5 8.0		▐▌	T-11	
	8.5	- trace coarse sand, firm below 8.2 m	◢	G-12	
	▼ 9.0		◢	G-13	
	9.5		◢	G-14	



Logged By:	Steven Harms	**Reviewed By:**	James Blatz	**Project Engineer:**	Ryan Belbas

Sub-Surface Log

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Number	SPT (N)
		- trace gravel (<25 mm), grey, soft below 10.2 m	G-15	
221.8	11.0	SILT (TILL) - clayey, some sand, trace gravel (<50 mm) - light brown - wet, loose	T-16	
			G-17	
			G-18	
220.4		- sandy below 12.2 m	SS-19	100 / 52mm



END OF TEST HOLE AT 12.4 m IN SILT (TILL)

Notes:

1. Power auger refusal at 12.4 m below ground surface.
2. Seepage at 11.3 m below ground surface.
3. Water level in test hole at 8.8 m below ground surface immediately after drilling.
4. Test hole backfilled with bentonite and auger cuttings.

Logged By: Steven Harms **Reviewed By:** James Blatz **Project Engineer:** Ryan Belbas

Sub-Surface Log

Test Hole TH15-02

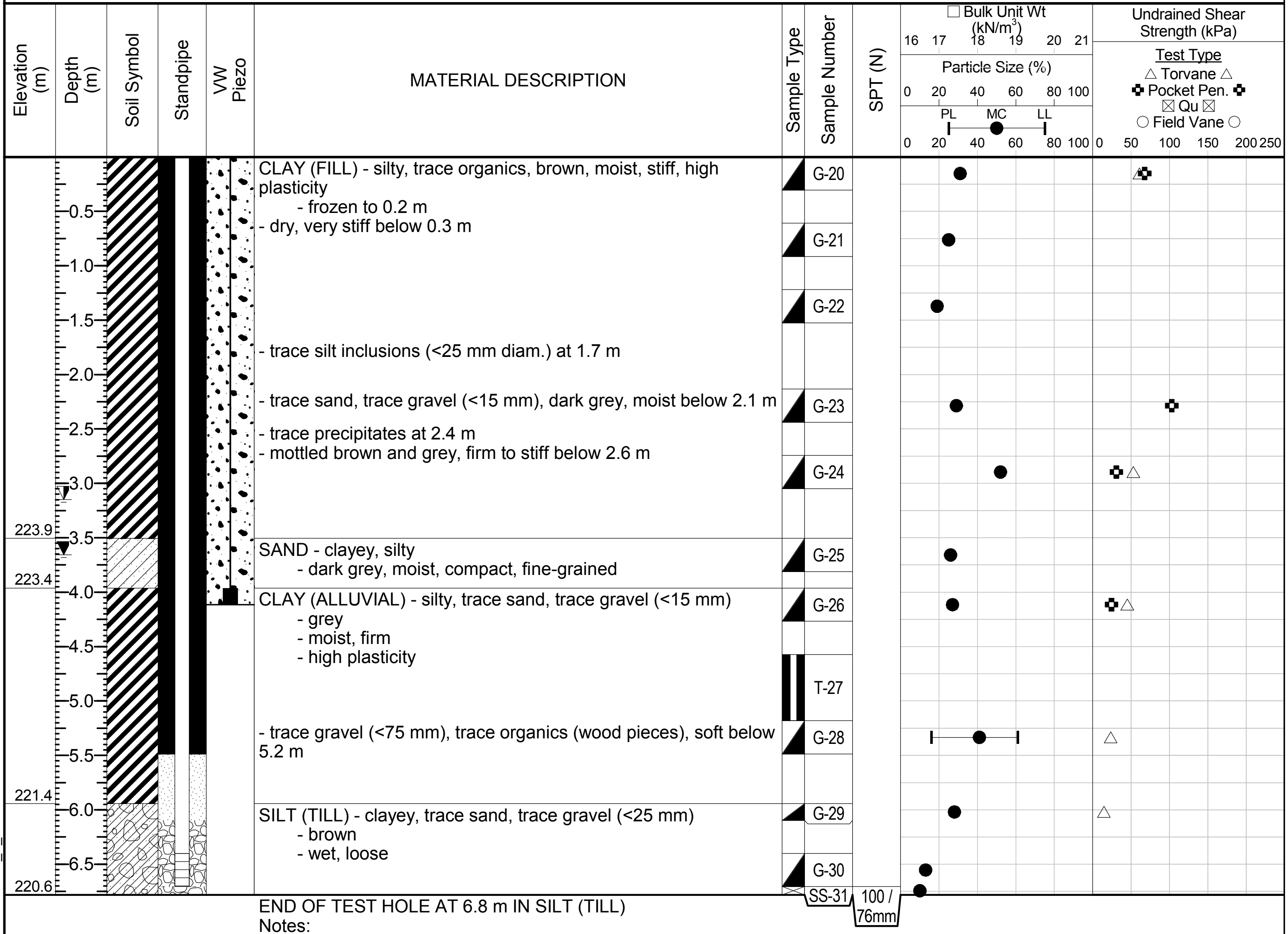
1 of 1

Client:	City of Winnipeg	**Project Number:**	0015 016 00
Project Name:	Cornish Library Addition	**Location:**	UTM N-5526707.65, E-632155.05
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	227.37 m
Method:	125 mm Solid Stem Auger, CME-850 Track Mount	**Date Drilled:**	4 December 2015

Sample Type: ◢ Grab (G) ▮ Shelby Tube (T) ☒ Split Spoon (SS) ⧗ Split Barrel (SB) ▮ Core (C)

Particle Size Legend: ▥ Fines ▨ Clay ▥ Silt ░ Sand ▚ Gravel ▢ Cobbles ■ Boulders

Backfill Legend: ■ Bentonite ▩ Cement ▨ Drill Cuttings ░ Filter Pack Sand ▚ Grout ▩ Slough



CLAY (FILL) - silty, trace organics, brown, moist, stiff, high plasticity
- frozen to 0.2 m
- dry, very stiff below 0.3 m
- trace silt inclusions (<25 mm diam.) at 1.7 m
- trace sand, trace gravel (<15 mm), dark grey, moist below 2.1 m
- trace precipitates at 2.4 m
- mottled brown and grey, firm to stiff below 2.6 m

SAND - clayey, silty
- dark grey, moist, compact, fine-grained

CLAY (ALLUVIAL) - silty, trace sand, trace gravel (<15 mm)
- grey
- moist, firm
- high plasticity
- trace gravel (<75 mm), trace organics (wood pieces), soft below 5.2 m

SILT (TILL) - clayey, trace sand, trace gravel (<25 mm)
- brown
- wet, loose

Sample Type	Sample Number	SPT (N)
◢	G-20	
◢	G-21	
◢	G-22	
◢	G-23	
◢	G-24	
◢	G-25	
◢	G-26	
▮	T-27	
◢	G-28	
◢	G-29	
◢	G-30	
☒	SS-31	100 / 76mm

END OF TEST HOLE AT 6.8 m IN SILT (TILL)

Notes:
1. Power auger refusal at 6.7 m below ground surface.
2. Sloughing at 3.5 m below ground surface.
3. Seepage at 5.9 m below ground surface.
4. Water level in test hole at 3.7 m below ground surface immediately after drilling.
5. Standpipe with Casagrande tip installed at 6.7 m below ground surface.
6. Test hole backfilled with sand and bentonite.
7. Vibrating Wire Piezometer installed at 4.1 m below ground surface in adjacent hole (1 m SE of TH15-02). Hole backfilled with bentonite grout.

Logged By: Steven Harms **Reviewed By:** James Blatz **Project Engineer:** Ryan Belbas

Appendix A
Laboratory Results

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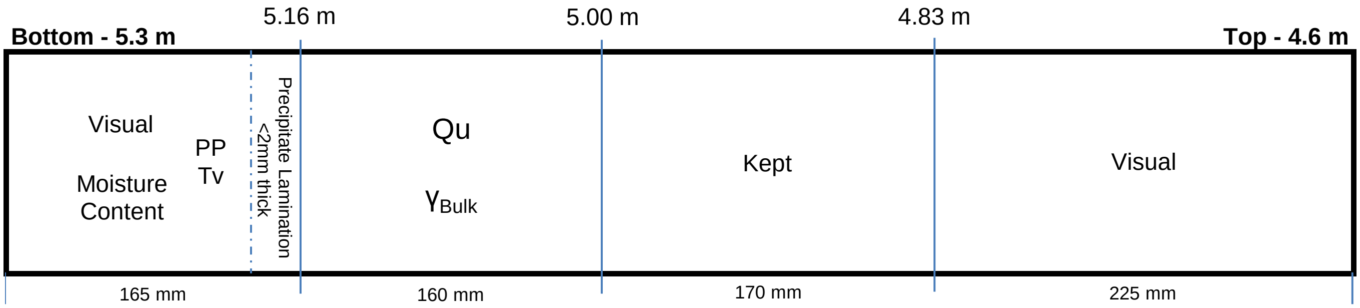
1712 St. James Street
Winnipeg, MB R3H 0L3
Tel: 204.975.9433 Fax: 204.975.9435

Shelby Tube Visual

Project No.	0015-016-00
Client	City of Winnipeg
Project	Cornish Library Addition
Test Hole	TH15-01
Sample #	T07
Depth (m)	4.6 - 5.3
Sample Date	03-Dec-15
Test Date	17-Dec-15
Technician	Daniel Wiebe

Tube Extraction

Recovery (mm) 720



Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (~<20mm ⌀)	
trace precipitates	
trace oxidation	
Color	mottled grey and brown
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	homogeneous / blocky
Gradation	

Torvane

Reading	0.80
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	78.5

Pocket Penetrometer

Reading	**1**	1.90
	2	1.80
	3	1.70
	Average	1.80
Undrained Shear Strength (kPa)		88.3

Moisture Content

Tare ID	R106
Mass tare (g)	4.5
Mass wet + tare (g)	351.2
Mass dry + tare (g)	231.9
Moisture %	52.5%

Unit Weight

Bulk Weight (g)		1116.8
Length (mm)	**1**	153.93
	2	153.93
	3	153.97
	4	154.02
Average Length (m)		0.154
Diam. (mm)	**1**	72.23
	2	72.52
	3	72.55
	4	72.48
Average Diameter (m)		0.072
Volume (m^3)		6.35E-04
Bulk Unit Weight (kN/m^3)		17.3
Bulk Unit Weight (pcf)		109.9
Dry Unit Weight (kN/m^3)		11.3
Dry Unit Weight (pcf)		72.1

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Winnipeg, MB R3H 0L3
Tel: 204.975.9433 Fax: 204.975.9435

Unconfined Compressive Strength

ASTM D2166

Project No.	0015-016-00
Client	City of Winnipeg
Project	Cornish Library Addition

Test Hole	TH15-01
Sample #	T07
Depth (m)	4.6 - 5.3
Sample Date	3-Dec-15
Test Date	17-Dec-15
Technician	Daniel Wiebe

Unconfined Strength

	kPa	ksf
Max q_u	174.2	3.6
Max S_u	87.1	1.8

Specimen Data

Description CLAY - silty, trace silt inclusions (~<20mm ⌀), trace precipitates, trace oxidation, mottled grey and brown, moist, stiff, high plasticity, homogeneous / blocky.

Length	154.0	(mm)
Diameter	72.4	(mm)
L/D Ratio	2.1	
Initial Area	0.00412	(m^2)
Load Rate	1.00	(%/min)

Moisture %	52%	
Bulk Unit Wt.	17.3	(kN/m^3)
Dry Unit Wt.	11.3	(kN/m^3)
Liquid Limit	-	
Plastic Limit	-	
Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.80	78.5	1.64
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.90	93.2	1.95
	1.80	88.3	1.84
	1.70	83.4	1.74
Average	**1.80**	**88.3**	**1.84**

Failure Geometry

Sketch:

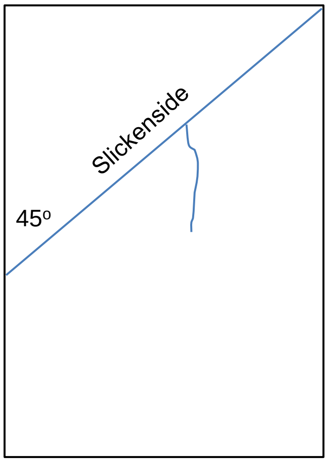


Photo:

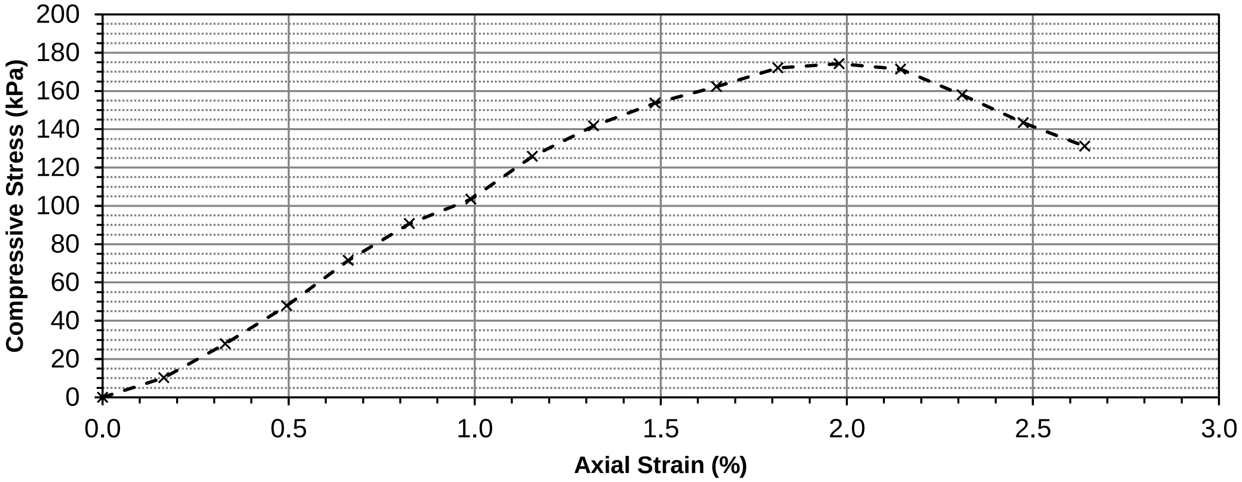
Project 0015-016-00
Location Cornish Library
Hole No. TH15-01 Sample No. T07
Depth 15-17' Date Dec 3/15
Technician SMH

Unconfined Compressive Strength

ASTM D2166

Project No. 0015-016-00
Client City of Winnipeg
Project Cornish Library Addition

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004122	0.0	0.00	0.00
10	13	0.2540	0.16	0.004129	42.5	10.30	5.15
20	35	0.5080	0.33	0.004136	115.4	27.90	13.95
30	60	0.7620	0.49	0.004142	197.8	47.75	23.88
40	90	1.0160	0.66	0.004149	296.7	71.51	35.76
50	114	1.2700	0.82	0.004156	376.9	90.67	45.34
60	130	1.5240	0.99	0.004163	430.7	103.46	51.73
70	158	1.7780	1.15	0.004170	525.0	125.90	62.95
80	178	2.0320	1.32	0.004177	592.4	141.81	70.91
90	193	2.2860	1.48	0.004184	642.9	153.65	76.83
100	204	2.5400	1.65	0.004191	680.4	162.34	81.17
110	216	2.7940	1.81	0.004198	722.2	172.03	86.02
120	219	3.0480	1.98	0.004205	732.7	174.23	87.11
130	216	3.3020	2.14	0.004212	722.2	171.45	85.73
140	200	3.5560	2.31	0.004219	666.5	157.95	78.98
150	182	3.8100	2.47	0.004227	605.8	143.34	71.67
160	167	4.0640	2.64	0.004234	555.4	131.17	65.59

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1712 St. James Street
Winnipeg, MB   R3H 0L3
Tel: 204.975.9433   Fax: 204.975.9435

Shelby Tube Visual

Project No.	0015-016-00
Client	City of Winnipeg
Project	Cornish Library Addition
Test Hole	TH15-01
Sample #	T11
Depth (m)	7.6 - 8.3
Sample Date	03-Dec-15
Test Date	16-Dec-15
Technician	Daniel Wiebe

Tube Extraction

Recovery (mm) 720

Bottom - 8.3 m | 8.16 m | 7.99 m | 7.82 m | **Top - 7.6 m**

Kept	Visual Moisture Content PP Tv	Qu γ_{Bulk}	Visual
160 mm	170 mm	170 mm	220 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (~<20mm ⌀)	
trace sand	
trace gravel (~<10mm ⌀)	
trace precipitates	
trace oxidation	
Color	mottled grey and brown
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	homogeneous / blocky
Gradation	

Torvane

Reading	0.56
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	54.9

Pocket Penetrometer

Reading	**1**	1.25
	2	0.80
	3	0.60
	Average	0.88
Undrained Shear Strength (kPa)		43.3

Moisture Content

Tare ID	D48
Mass tare (g)	8.4
Mass wet + tare (g)	372.1
Mass dry + tare (g)	255.6
Moisture %	47.1%

Unit Weight

Bulk Weight (g)		1080.0
Length (mm)	**1**	153.06
	2	152.88
	3	152.59
	4	152.64
Average Length (m)		0.153
Diam. (mm)	**1**	72.00
	2	72.31
	3	72.77
	4	72.92
Average Diameter (m)		0.073
Volume (m^3)		6.31E-04
Bulk Unit Weight (kN/m^3)		16.8
Bulk Unit Weight (pcf)		106.9
Dry Unit Weight (kN/m^3)		11.4
Dry Unit Weight (pcf)		72.7

Unconfined Compressive Strength

ASTM D2166

Project No.	0015-016-00
Client	City of Winnipeg
Project	Cornish Library Addition

Test Hole	TH15-01
Sample #	T11
Depth (m)	7.6 - 8.3
Sample Date	3-Dec-15
Test Date	16-Dec-15
Technician	Daniel Wiebe

Unconfined Strength

	kPa	**ksf**
Max q_u	65.4	1.4
Max S_u	32.7	0.7

Specimen Data

Description CLAY - silty, trace silt inclusions (~<20mm ø), trace sand, trace gravel (~<10mm ø), trace precipitates, trace oxidation, mottled grey and brown, moist, firm, high plasticity, homogeneous / blocky.

Length	152.8	(mm)	**Moisture %**	47%	
Diameter	72.5	(mm)	**Bulk Unit Wt.**	16.8	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	11.4	(kN/m^3)
Initial Area	0.00413	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

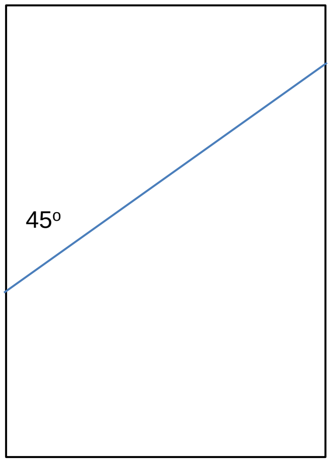
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.56	54.9	1.15
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.25	61.3	1.28
	0.80	39.2	0.82
	0.60	29.4	0.61
Average	**0.88**	**43.3**	**0.90**

Failure Geometry

Sketch:



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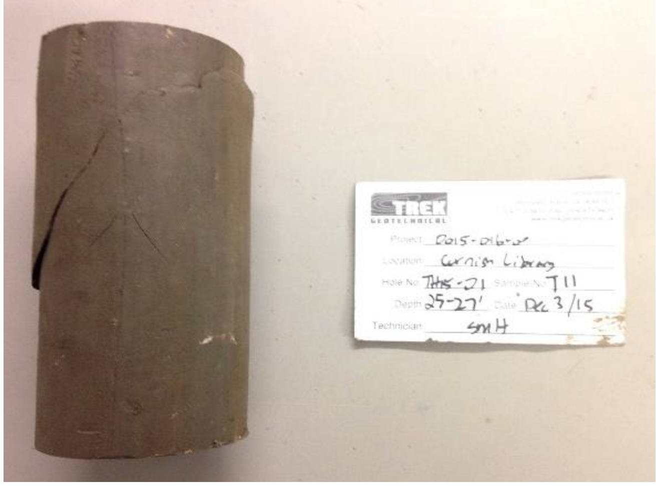
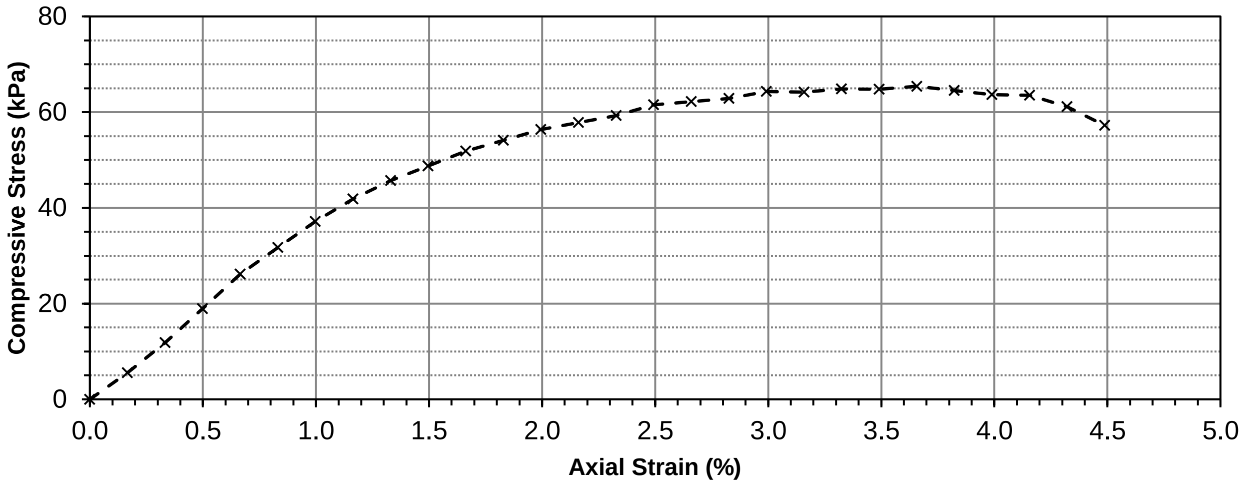


Unconfined Compressive Strength

ASTM D2166

Project No. 0015-016-00
Client City of Winnipeg
Project Cornish Library Addition

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004128	0.0	0.00	0.00
10	7	0.2540	0.17	0.004135	22.9	5.53	2.77
20	15	0.5080	0.33	0.004142	49.1	11.85	5.93
30	24	0.7620	0.50	0.004149	78.6	18.95	9.48
40	33	1.0160	0.66	0.004156	108.8	26.18	13.09
50	40	1.2700	0.83	0.004163	131.9	31.68	15.84
60	47	1.5240	1.00	0.004170	155.0	37.17	18.58
70	53	1.7780	1.16	0.004177	174.7	41.83	20.92
80	58	2.0320	1.33	0.004184	191.2	45.71	22.85
90	62	2.2860	1.50	0.004191	204.4	48.77	24.39
100	66	2.5400	1.66	0.004198	217.6	51.84	25.92
110	69	2.7940	1.83	0.004205	227.5	54.10	27.05
120	72	3.0480	1.99	0.004212	237.4	56.36	28.18
130	74	3.3020	2.16	0.004219	244.0	57.82	28.91
140	76	3.5560	2.33	0.004227	250.6	59.28	29.64
150	79	3.8100	2.49	0.004234	260.4	61.51	30.76
160	80	4.0640	2.66	0.004241	263.8	62.20	31.10
170	81	4.3180	2.83	0.004248	267.1	62.87	31.43
180	83	4.5720	2.99	0.004256	273.7	64.30	32.15
190	83	4.8260	3.16	0.004263	273.7	64.19	32.10
200	84	5.0800	3.32	0.004270	276.9	64.86	32.43
210	84	5.3340	3.49	0.004278	276.9	64.74	32.37
220	85	5.5880	3.66	0.004285	280.2	65.40	32.70
230	84	5.8420	3.82	0.004292	276.9	64.52	32.26

Project No. 0015-016-00
Client City of Winnipeg
Project Cornish Library Addition

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	83	6.0960	3.9897	0.004300	273.7	63.64	31.82
250	83	6.3500	4.16	0.004307	273.7	63.53	31.77
260	80	6.6040	4.32	0.004315	263.8	61.13	30.57
270	75	6.8580	4.49	0.004322	247.3	57.21	28.61

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Shelby Tube Visual

Project No.	0015-016-00
Client	City of Winnipeg
Project	Cornish Library Addition
Test Hole	TH15-01
Sample #	T16
Depth (m)	10.7 - 11.4
Sample Date	03-Dec-15
Test Date	17-Dec-15
Technician	Daniel Wiebe

Tube Extraction

Recovery (mm) 700

Bottom - 11.4 m		11.24 m	⇐ **B**	11.07 m **A** ⇒	10.91 m	**Top - 10.7 m**
Visual CLAY (TILL) - silty, trace silt inclusions (~<30mm ø), trace to some sand, trace gravel (~<40mm ø), grey, moist to wet, soft, intermediate plasticity	Moisture Content	Kept		Qu γ_{Bulk}	PP Tv	Visual Moisture Content
160 mm		170 mm		160 mm		210 mm

Visual Classification - A

Material	CLAY
Composition	silty
trace silt inclusions (~<30mm ø)	
trace sand (well graded)	
trace gravel (~<30mm ø)	
Color	grey
Moisture	moist to wet
Consistency	soft
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

	B	A
Reading		0.20
Vane Size (s,m,l)		m
Undrained Shear Strength (kPa)		19.6

Pocket Penetrometer

Reading	**1**	0.25
	2	
	3	
	Average	
Undrained Shear Strength (kPa)		12.3

Moisture Content

	B	A
Tare ID	Z113	E85
Mass tare (g)	8.5	8.6
Mass wet + tare (g)	423.2	358.3
Mass dry + tare (g)	331.9	247.2
Moisture %	28.2%	46.6%

Unit Weight

		B	A
Bulk Weight (g)			1156.4
Length (mm)	**1**		151.59
	2		151.95
	3		151.70
	4		151.53
Average Length (m)			0.152
Diam. (mm)	**1**		71.99
	2		71.52
	3		71.89
	4		72.00
Average Diameter (m)			0.072
Volume (m^3)			6.15E-04
Bulk Unit Weight (kN/m^3)			18.4
Bulk Unit Weight (pcf)			117.4
Dry Unit Weight (kN/m^3)			12.6
Dry Unit Weight (pcf)			80.1

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Unconfined Compressive Strength

ASTM D2166

Project No.	0015-016-00
Client	City of Winnipeg
Project	Cornish Library Addition
Test Hole	TH15-01
Sample #	T16
Depth (m)	10.7 - 11.4
Sample Date	3-Dec-15
Test Date	17-Dec-15
Technician	Daniel Wiebe

Unconfined Strength

	kPa	**ksf**
Max q_u	42.1	0.9
Max S_u	21.1	0.4

Specimen Data

Description CLAY - silty, trace silt inclusions (~<30mm ⌀), trace sand (well graded), trace gravel (~<30mm ⌀), grey, moist to wet, soft, high plasticity, homogeneous.

Length	151.7	(mm)	**Moisture %**	47%	
Diameter	71.9	(mm)	**Bulk Unit Wt.**	18.4	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	12.6	(kN/m^3)
Initial Area	0.00405	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.20	19.6	0.41
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**

Average

Failure Geometry

Sketch:

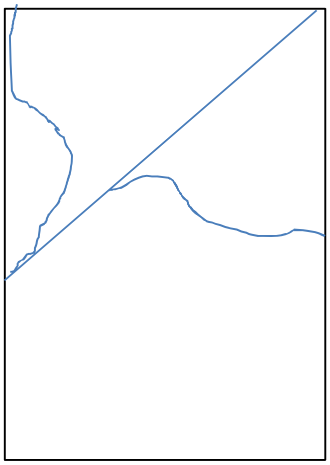

Photo:

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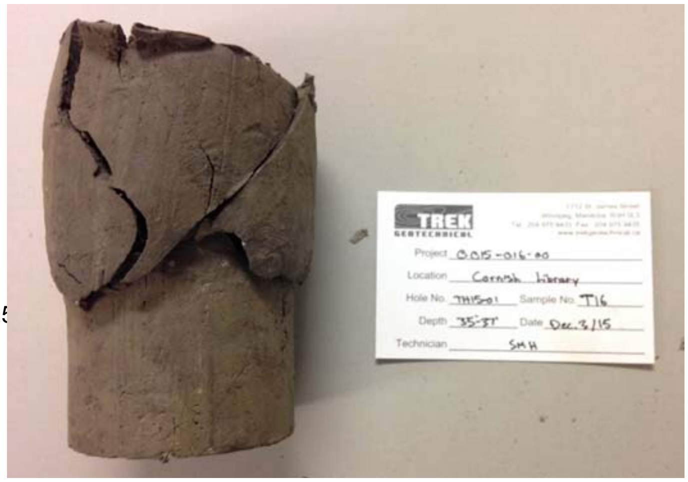


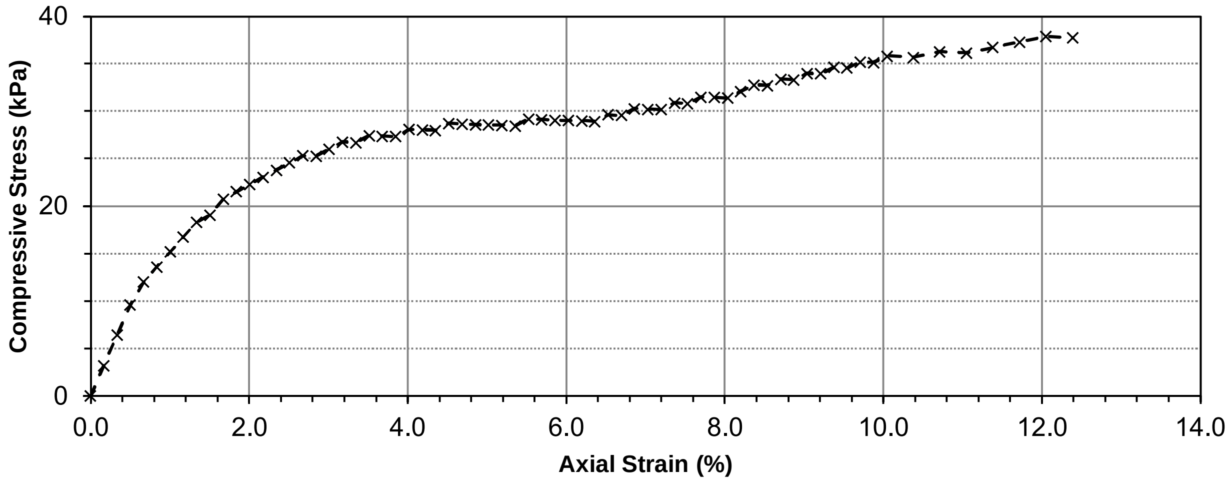
Project 0015-016-00
Location Cornish Library
Hole No. TH15-01 Sample No. T16
Depth 35'-37' Date Dec.3/15
Technician SMH

Unconfined Compressive Strength

ASTM D2166

Project No. 0015-016-00
Client City of Winnipeg
Project Cornish Library Addition

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004055	0.0	0.00	0.00
10	4	0.2540	0.17	0.004061	13.1	3.22	1.61
20	8	0.5080	0.33	0.004068	26.2	6.43	3.22
30	12	0.7620	0.50	0.004075	39.3	9.63	4.82
40	15	1.0160	0.67	0.004082	49.1	12.03	6.01
50	17	1.2700	0.84	0.004089	55.7	13.61	6.81
60	19	1.5240	1.00	0.004096	62.2	15.19	7.59
70	21	1.7780	1.17	0.004103	68.8	16.76	8.38
80	23	2.0320	1.34	0.004110	75.3	18.33	9.17
90	24	2.2860	1.51	0.004117	78.6	19.10	9.55
100	26	2.5400	1.67	0.004124	85.7	20.79	10.39
110	27	2.7940	1.84	0.004131	89.0	21.55	10.77
120	28	3.0480	2.01	0.004138	92.3	22.31	11.15
130	29	3.3020	2.18	0.004145	95.6	23.06	11.53
140	30	3.5560	2.34	0.004152	98.9	23.83	11.91
150	31	3.8100	2.51	0.004159	102.2	24.58	12.29
160	32	4.0640	2.68	0.004166	105.5	25.33	12.66
170	32	4.3180	2.85	0.004173	105.5	25.28	12.64
180	33	4.5720	3.01	0.004181	108.8	26.03	13.01
190	34	4.8260	3.18	0.004188	112.1	26.77	13.38
200	34	5.0800	3.35	0.004195	112.1	26.72	13.36
210	35	5.3340	3.52	0.004202	115.4	27.46	13.73
220	35	5.5880	3.68	0.004210	115.4	27.41	13.71
230	35	5.8420	3.85	0.004217	115.4	27.36	13.68

Unconfined Compressive Strength

ASTM D2166

Project No. 0015-016-00
Client City of Winnipeg
Project Cornish Library Addition

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	36	6.0960	4.0187	0.004224	118.7	28.09	14.05
250	36	6.3500	4.19	0.004232	118.7	28.05	14.02
260	36	6.6040	4.35	0.004239	118.7	28.00	14.00
270	37	6.8580	4.52	0.004247	122.0	28.72	14.36
280	37	7.1120	4.69	0.004254	122.0	28.67	14.34
290	37	7.3660	4.86	0.004261	122.0	28.62	14.31
300	37	7.6200	5.02	0.004269	122.0	28.57	14.29
310	37	7.8740	5.19	0.004277	122.0	28.52	14.26
320	37	8.1280	5.36	0.004284	122.0	28.47	14.24
330	38	8.3820	5.53	0.004292	125.3	29.20	14.60
340	38	8.6360	5.69	0.004299	125.3	29.15	14.57
350	38	8.8900	5.86	0.004307	125.3	29.09	14.55
360	38	9.1440	6.03	0.004315	125.3	29.04	14.52
370	38	9.3980	6.20	0.004322	125.3	28.99	14.50
380	38	9.6520	6.36	0.004330	125.3	28.94	14.47
390	39	9.9060	6.53	0.004338	128.6	29.65	14.82
400	39	10.1600	6.70	0.004346	128.6	29.59	14.80
410	40	10.4140	6.87	0.004353	131.9	30.30	15.15
420	40	10.6680	7.03	0.004361	131.9	30.24	15.12
430	40	10.9220	7.20	0.004369	131.9	30.19	15.09
440	41	11.1760	7.37	0.004377	135.2	30.88	15.44
450	41	11.4300	7.53	0.004385	135.2	30.83	15.41
460	42	11.6840	7.70	0.004393	138.5	31.52	15.76
470	42	11.9380	7.87	0.004401	138.5	31.46	15.73
480	42	12.1920	8.04	0.004409	138.5	31.41	15.70
490	43	12.4460	8.20	0.004417	141.8	32.10	16.05
500	44	12.7000	8.37	0.004425	145.1	32.78	16.39
510	44	12.9540	8.54	0.004433	145.1	32.72	16.36
520	45	13.2080	8.71	0.004441	148.3	33.40	16.70
530	45	13.4620	8.87	0.004449	148.3	33.34	16.67
540	46	13.7160	9.04	0.004458	151.7	34.03	17.01
550	46	13.9700	9.21	0.004466	151.7	33.97	16.98
560	47	14.2240	9.38	0.004474	155.0	34.64	17.32
570	47	14.4780	9.54	0.004482	155.0	34.57	17.29
580	48	14.7320	9.71	0.004491	158.3	35.24	17.62
590	48	14.9860	9.88	0.004499	158.3	35.18	17.59
600	49	15.2400	10.05	0.004507	161.6	35.84	17.92
620	49	15.7480	10.38	0.004524	161.6	35.71	17.85
640	50	16.2560	10.72	0.004541	164.9	36.30	18.15
660	50	16.7640	11.05	0.004558	164.9	36.16	18.08
680	51	17.2720	11.39	0.004576	168.1	36.75	18.37
700	52	17.7800	11.72	0.004593	171.4	37.33	18.66
720	53	18.2880	12.06	0.004610	174.7	37.90	18.95
740	53	18.7960	12.39	0.004628	174.7	37.75	18.88

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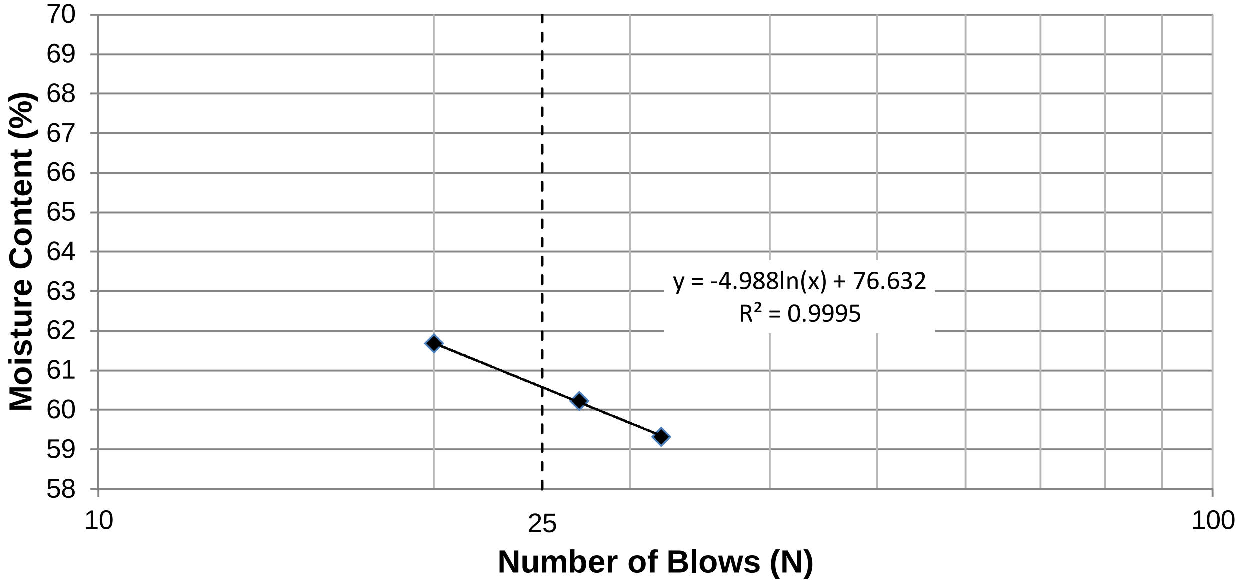
Atterberg Limits
ASTM D4318

Project No.	0015 015 00
Client	City of Winnipeg
Project	Cornish Library Addition
Test Hole	TH15-02
Sample #	G28
Depth (m)	5.18 - 5.48
Sample Date	08-Dec-15
Test Date	26-Jan-16
Technician	J.B / L.I

Liquid Limit	61
Plastic Limit	16
Plasticity Index	44

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	32	27	20		
Mass Wet Soil + Tare (g)	24.434	23.387	23.400		
Mass Dry Soil + Tare (g)	20.643	20.032	19.933		
Mass Tare (g)	14.253	14.461	14.312		
Mass Water (g)	3.791	3.355	3.467		
Mass Dry Soil (g)	6.390	5.571	5.621		
Moisture Content (%)	59.327	60.223	61.679		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	17.818	16.290	16.457		
Mass Dry Soil + Tare (g)	17.285	15.982	16.123		
Mass Tare (g)	14.251	13.915	14.124		
Mass Water (g)	0.533	0.308	0.334		
Mass Dry Soil (g)	3.034	2.067	1.999		
Moisture Content (%)	17.568	14.901	16.708		

Analysis B
Slope Stability Results

Cornish Library Addition
0015 016 00

Existing Conditions
GWL at approx. 2.5 m below ground surface
River Level at Elev. 224.0 (RSRL)
Long-Term (Normal Summer)

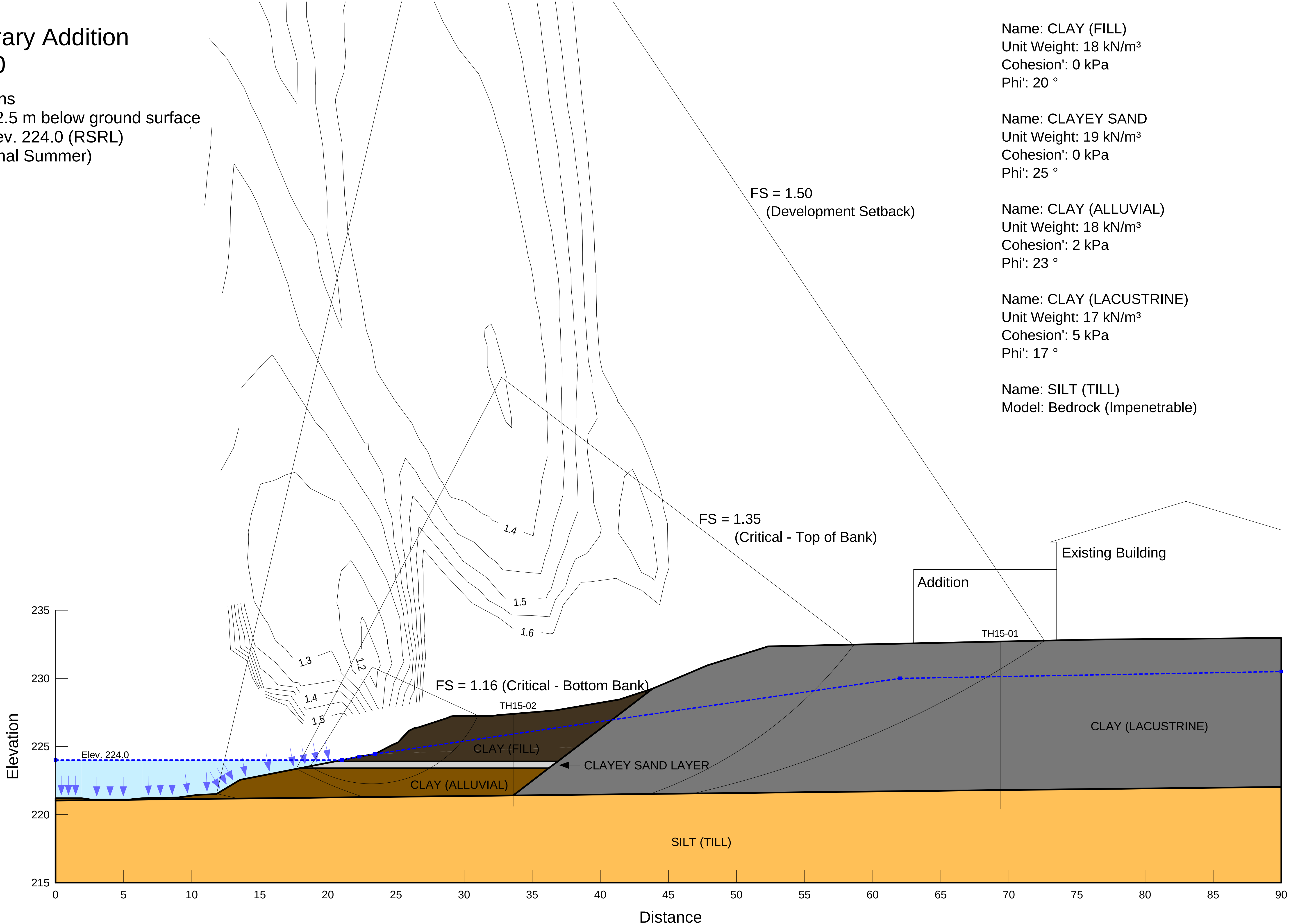
Name: CLAY (FILL)
Unit Weight: 18 kN/m³
Cohesion': 0 kPa
Phi': 20 °

Name: CLAYEY SAND
Unit Weight: 19 kN/m³
Cohesion': 0 kPa
Phi': 25 °

Name: CLAY (ALLUVIAL)
Unit Weight: 18 kN/m³
Cohesion': 2 kPa
Phi': 23 °

Name: CLAY (LACUSTRINE)
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: SILT (TILL)
Model: Bedrock (Impenetrable)



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Date: 1/28/2016

Cornish Library Addition
0015 016 00

Riprap at Bank Toe - approx. 1 m thick
GWL at approx. 2.5 m below ground surface
River Level at Elev. 224.0 m (RSRL)
Long-Term (Normal Summer)

Name: RIP RAP
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 40 °

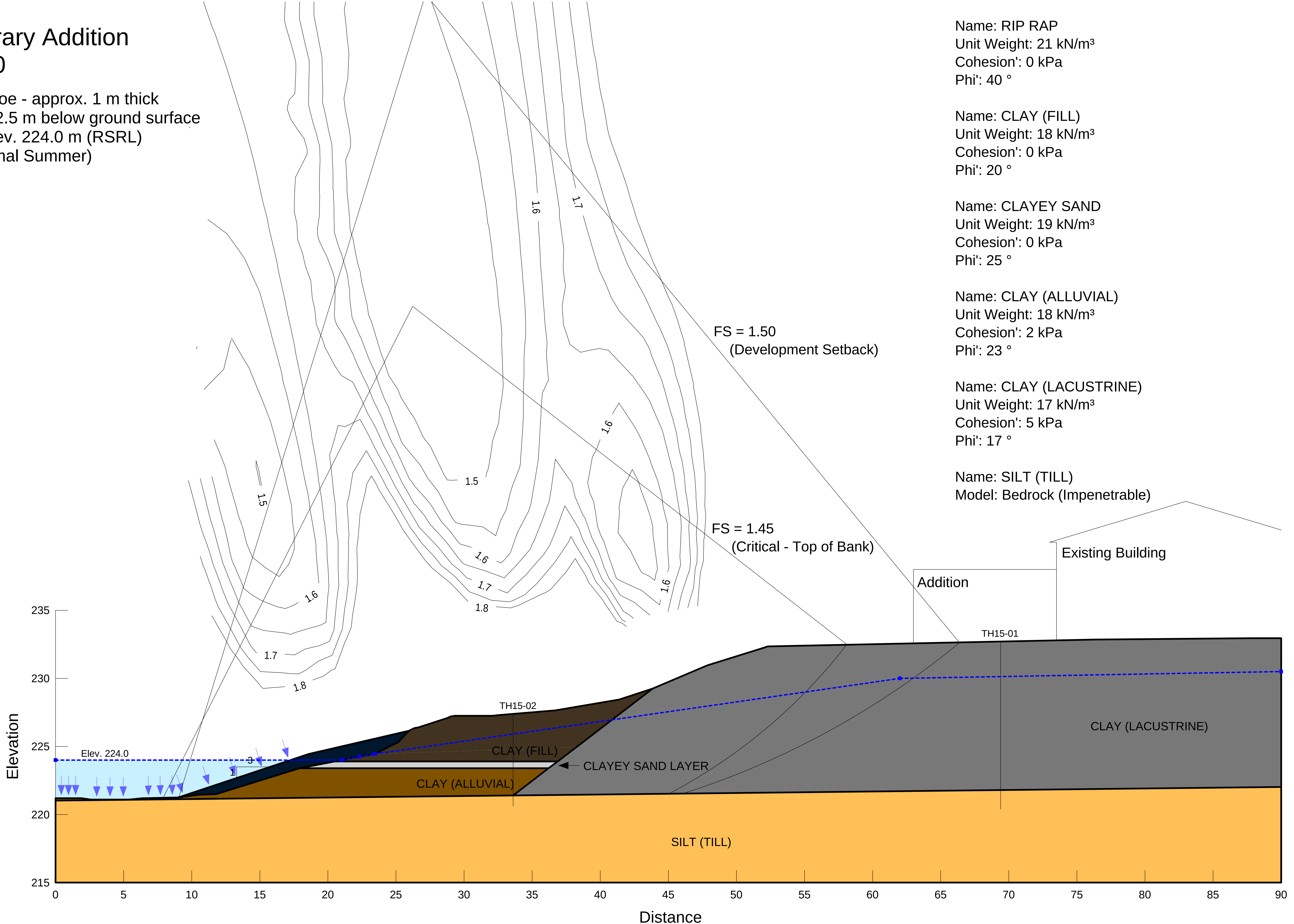
Name: CLAY (FILL)
Unit Weight: 18 kN/m³
Cohesion': 0 kPa
Phi': 20 °

Name: CLAYEY SAND
Unit Weight: 19 kN/m³
Cohesion': 0 kPa
Phi': 25 °

Name: CLAY (ALLUVIAL)
Unit Weight: 18 kN/m³
Cohesion': 2 kPa
Phi': 23 °

Name: CLAY (LACUSTRINE)
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: SILT (TILL)
Model: Bedrock (Impenetrable)



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Date: 1/28/2016

Cornish Library Addition
0015 016 00

Regrade Top of Bank
GWL at approx. 2.5 m below ground surface
River Level at Elev. 224.0 m (RSRL)
Long-Term (Normal Summer)

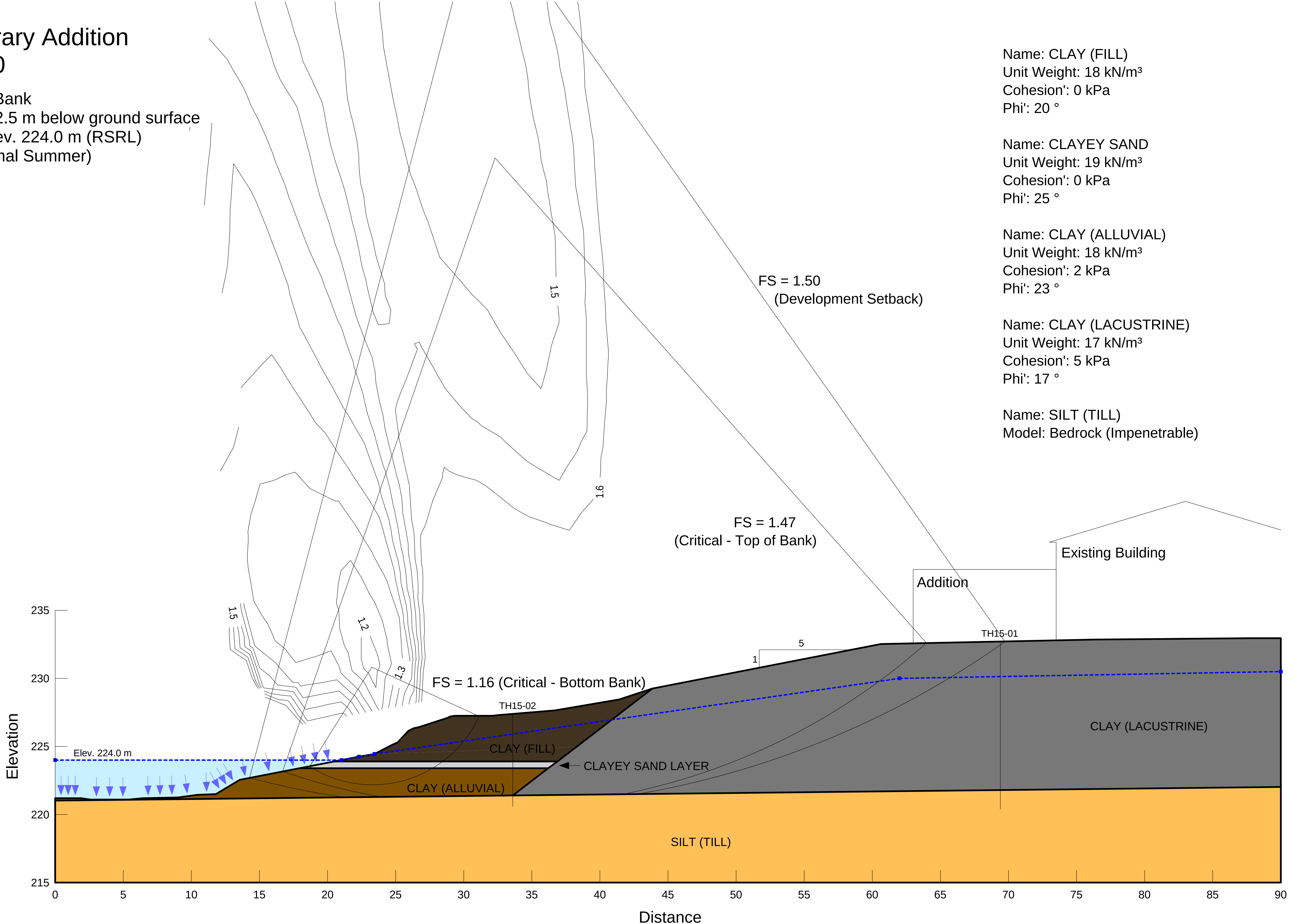
Name: CLAY (FILL)
Unit Weight: 18 kN/m³
Cohesion': 0 kPa
Phi': 20 °

Name: CLAYEY SAND
Unit Weight: 19 kN/m³
Cohesion': 0 kPa
Phi': 25 °

Name: CLAY (ALLUVIAL)
Unit Weight: 18 kN/m³
Cohesion': 2 kPa
Phi': 23 °

Name: CLAY (LACUSTRINE)
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: SILT (TILL)
Model: Bedrock (Impenetrable)



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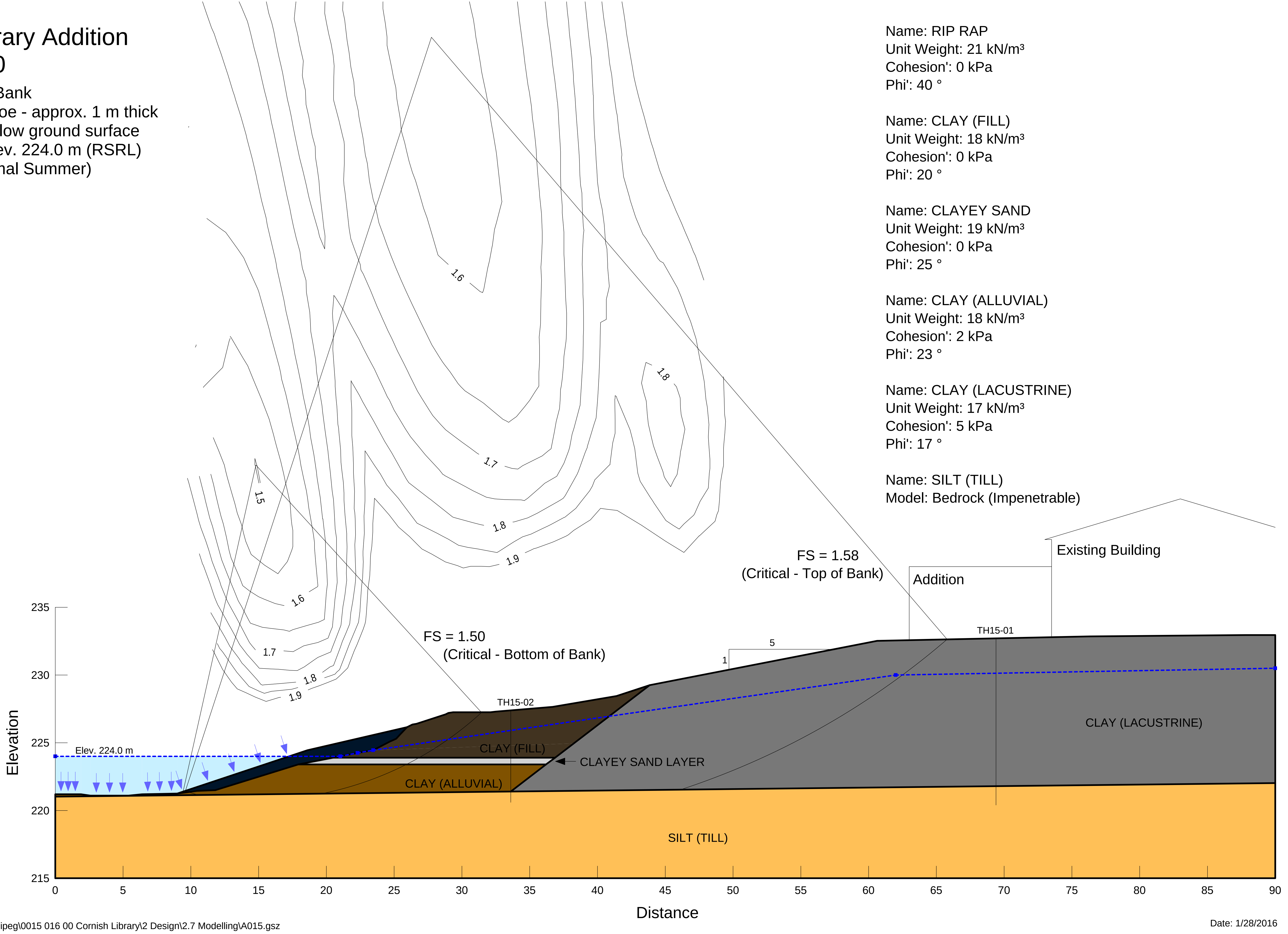
Date: 1/28/2016

Cornish Library Addition
0015 016 00

Regrade Top of Bank
Riprap at Bank Toe - approx. 1 m thick
GWL at 2.5 m below ground surface
River Level at Elev. 224.0 m (RSRL)
Long-Term (Normal Summer)



Name: RIP RAP
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 40 °

Name: CLAY (FILL)
Unit Weight: 18 kN/m³
Cohesion': 0 kPa
Phi': 20 °

Name: CLAYEY SAND
Unit Weight: 19 kN/m³
Cohesion': 0 kPa
Phi': 25 °

Name: CLAY (ALLUVIAL)
Unit Weight: 18 kN/m³
Cohesion': 2 kPa
Phi': 23 °

Name: CLAY (LACUSTRINE)
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: SILT (TILL)
Model: Bedrock (Impenetrable)

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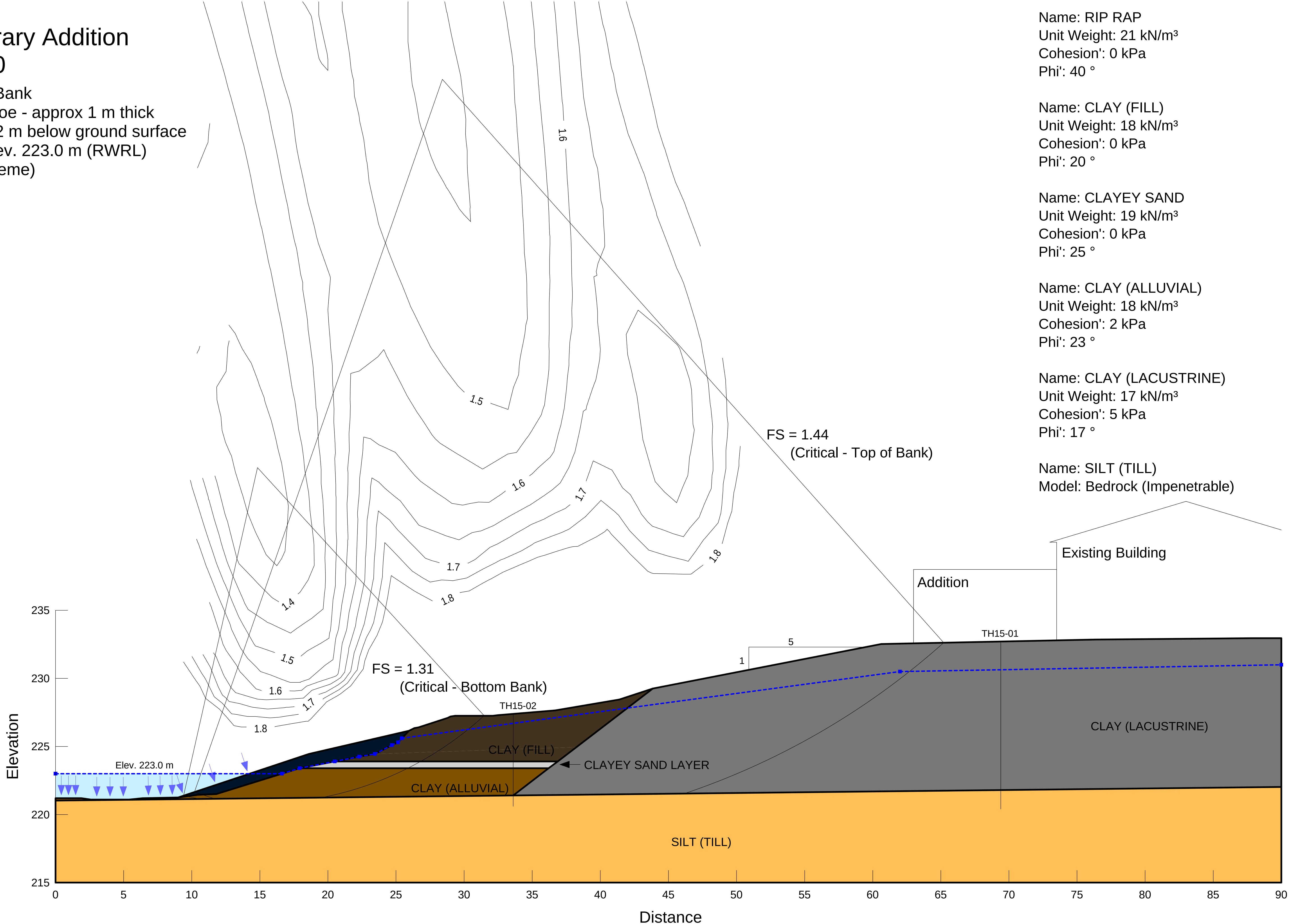
Date: 1/28/2016

Cornish Library Addition
0015 016 00

Regrade Top of Bank
Riprap at Bank Toe - approx 1 m thick
GWL at approx. 2 m below ground surface
River Level at Elev. 223.0 m (RWRL)
Short-Term (Extreme)



Name: RIP RAP
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 40 °

Name: CLAY (FILL)
Unit Weight: 18 kN/m³
Cohesion': 0 kPa
Phi': 20 °

Name: CLAYEY SAND
Unit Weight: 19 kN/m³
Cohesion': 0 kPa
Phi': 25 °

Name: CLAY (ALLUVIAL)
Unit Weight: 18 kN/m³
Cohesion': 2 kPa
Phi': 23 °

Name: CLAY (LACUSTRINE)
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: SILT (TILL)
Model: Bedrock (Impenetrable)

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Date: 1/28/2016