ATTACHMENT – 6

GEOTECHNICAL ANALYSIS

THURBER ENGINEERING LTD.

CRYSTAL POOL & WELLNESS CENTRE REPLACEMENT PROJECT
GEOTECHNICAL DESIGN REPORT

Report

to

City of Victoria

PROFESSIONAL
PROVINCE
S. M. BEAN
BRITISH
COLUMBIA
ENGINEER

Stephen Bean

Oct. 26/18

Stephen Bean, M.Eng., P. Eng.
Review Principal

Caleb Scott, M.Eng., EIT
Project Engineer

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1. INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Thurber Engineering Ltd. (Thurber) for the proposed Crystal Pool & Wellness Centre replacement project for the City of Victoria (the City). It also provides geotechnical recommendations to support the design of the structure.

Our scope of work was described in our proposal letter dated May 22, 2018. Authorization to proceed with the work was provided by Work Order No. 216785 dated July 4, 2018.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions

2. PROJECT DESCRIPTION

We understand that the City plans to replace the aging Crystal Pool with a new facility located in the southwest quadrant of Central Park.

Thurber was provided with architectural and schematic structural design drawings by HCMA Architecture and Design (HCMA) and Fast+Epp structural engineers dated June 7, 2018. HCMA also provided a street level site plan in AutoCAD format dated August 8th, 2018. The drawings show an 'L' shaped structure approximately 95 m long in the east-west direction and ranging from 60 m to 75 m wide in the north-south direction. The structure will be setback approximately 7 m from the west property line (Quadra Street right of way) and 3 m to 8 m from the south property line (Pembroke Street right of way).

To the east and north the site borders the sport fields and green spaces of Central Park. The new structure is setback approximately 25 m from the existing Crystal Pool structure to the north. We understand that the City prefers to retain many of the existing trees along the south, north and east perimeters of the proposed structure.

The structure will include 2 storeys above ground and approximately 2 storeys below ground. The foundation plan shows a foundation depth which varies within the building footprint. We understand that the top of the slab-on-grade will be approximately 5.6 m below current ground surface for the majority of the footprint. In the southeast corner (approximately 1/3 of the footprint), the top of the slab will be approximately 3.7 m below ground surface. The site and surrounding area are relatively level. The site has an approximate elevation of 22 m geodetic, with approximately 2 m elevation gain from north to south.

3. DRILLING INVESTIGATIONS

3.1 Preliminary Investigation June 2017

Golder Associates Ltd. completed a preliminary geotechnical investigation in June 2017 which is described in their report dated August 15, 2017. That investigation included 4 test holes drilled using the sonic method. Golder's test hole logs and laboratory data from that investigation are provided in Appendix B for reference. The preliminary investigation results indicated that bedrock is relatively shallow and generally increases in depth from west to east. The investigation results also indicated that soil conditions are generally relatively favourable. However, soft soil of limited thickness was encountered at the east end of the site. Shallow perched groundwater was encountered during drilling, but stable groundwater levels were not characterized.

3.2 Current Investigation

Thurber recently conducted a geotechnical drilling investigation to support detailed design of the proposed structure. The investigation included auger drilling and bedrock probing methods. Prior to drilling, a BC One Call and a private utility locate were completed to check for underground services in the area. The test holes were logged in the field by a Thurber representative. The locations of these test holes are shown on Dwg. 22952-1.

The results of the drilling and laboratory testing were used to compile test hole logs which are included in Appendix A. The test hole locations were located via handheld GPS and are shown in Dwg. 22952-1 along with interpreted bedrock depths from surface. The test hole locations were surveyed by McElhanney Associates Land Surveying Ltd. on July 26, 2018 and September 28, 2018; however, this data has not yet been provided to Thurber.

<u>Bedrock Probing</u>

To delineate the variability in the depth to bedrock, 8 test holes (including 2 additional holes requested by the City) were drilled on July 26, 2018 using a track mounted top-drive hydraulic rock drill operated by Western Grater Contracting Ltd. (Photo 1). These holes were drilled until bedrock was encountered. This method does not provide characterization of soils.

A supplemental bedrock probing investigation was requested by the City to follow up on the significant depth of bedrock in the east portion of the site to reduce uncertainty in the foundation and shoring design. The supplemental investigation consisted of 9 additional bedrock probe test holes completed on September 28, 2018, using the same method and contractor.

<u>Auger Drilling & In Situ Testing</u>

To fill gaps in the characterization of subsurface soils and groundwater, 4 test holes (including 1 additional hole) were drilled on August 1, 2018 using a track mounted Geo-Probe auger drill operated by Drillwell Enterprises Ltd (Photo 2). These holes were typically drilled to auger refusal in glacial till or at inferred bedrock.

The additional auger hole (TH18-12) was required adjacent to bedrock probe hole TH18-03 where bedrock was found to be significantly deeper than anticipated based on the 2017 preliminary investigation. Vane shear tests were completed at TH18-12 near the elevation of the proposed foundation to characterize the undrained shear strength of the native grey clay soil.

Dynamic cone penetration tests (DCPT) were driven from surface to practical refusal in the glacial till or at bedrock at each auger hole location. DCPT's provide a qualitative estimate of in-situ density for granular soil and are useful for identifying stiffness and strength contrasts within and between different strata. The DCPT tip is similar in size and shape to the Standard Penetration Test (SPT) split spoon sampler and is driven using the same hammer. However, the DCPT is not a standardized test and cannot be used to infer the in-situ density of granular soil or to assess liquefaction potential. The DCPT results are not included on the test hole logs due to significant rod friction being evident in the recorded blow counts within the native clay soil which may be misleading. The results of the vane shear tests should be used in preference.

A 50 mm diameter standpipe piezometer was installed at TH18-11 at the completion of drilling for monitoring of groundwater levels in this area.

Photo 1. Hydraulic Rock Drill used to probe for Bedrock (location TH18-01)

Photo 2. Auger Drill at location TH18-12 with existing Crystal Pool in background

Laboratory Testing

Disturbed soil samples were collected from the augers and returned to our laboratory for routine visual identification (ASTM D2488) and moisture content (ASTM 4959) determination. Atterberg Limit (ASTM D4318) tests were conducted on 2 selected samples of the native grey clay soil to determine the plasticity. Passing No. 200 Sieve (ASTM C117) tests were conducted on 2 selected samples of the glacial till soil to determine the fines content. The results of the laboratory testing are provided in Appendix A.

Test Hole Closure

All test holes were backfilled with soil cuttings and bentonite seals in accordance with the 2016 BC Groundwater Protection Regulations. In the tennis court at TH18-06, an asphalt cold patch was installed at the surface.

4. SUBSURFACE CONDITIONS

4.1 Regional Geology

The BC Geological Survey has mapped the site as 'Thin Soil Cover with Scattered Bedrock Outcrop' (BCGS Map 2000-2), which they define as less than 5 m of Victoria Clay overlying thin older Pleistocene deposits (e.g. glacial till) or bedrock. Thicker clay deposits are mapped northeast of the site. The map indicates that it is based on widely spaced subsurface information.

The Victoria Clay is a common soil deposit found in low-lying portions of the Greater Victoria area. It is a glacio-marine silty clay deposited during the recession of glacial ice at the conclusion of the Fraser Glaciation (circa 10,000 years ago). The Upper Facies of the Victoria Clay is typically stiff to very stiff and brown due to desiccation. The Lower Facies of the Victoria Clay is typically softer and grey where it has not been exposed to desiccation. The transition between the two facies varies significantly but commonly occurs over several metres. Particular attention is drawn to the decreasing shear strength with depth in the Victoria Clay deposit.

The Canadian Geological Survey has mapped the bedrock geology in the area as the metamorphic Wark Gneiss (GSC Map 1553A), comprised of massive and gneissic metadiorite, metagabbro, and amphibolite. This formation is common in the greater Victoria area.

4.2 Soil Conditions

A generalized description of the soil and groundwater conditions encountered in the current (2018) and 2017 test holes is provided below. The reader should, however, refer to the test hole logs in Appendix A and Appendix B for a detailed description of the soil and groundwater conditions at each test hole location. The investigation results were generally consistent with the mapped regional geology, except that the encountered Victoria Clay deposits were thicker than mapped by the BCGS within the northeast corner of the site.

Fill / Topsoil

No mineral fills were encountered in the test holes. Topsoil thickness varied from 0.2 m to 0.6 m and it is possible that some of this may be imported topsoil used to level the site and to support lawn growth.

Victoria Clay – Upper Facies

The Upper Facies of the Victoria Clay deposit was encountered at all auger and sonic test hole locations, except at TH18-09 where bedrock was very shallow. It had a typical thickness of approximately 3 m to 4 m (TH18-11, TH18-12 and BH17-05), which is typical for the Victoria area based on the depth of the effects of desiccation. The Upper Facies was thinner or even absent where constrained by shallow glacial till or bedrock.

The Upper Facies of the Victoria Clay is comprised of silty clay and typically contains only minor fractions of sand and gravel. However, the 2017 investigation results at several locations indicate that the clay can contain significant sand and gravel fractions and be difficult to distinguish from the glacial till where bedrock is relatively shallow. It is typically brown in colour, or brown mottled with grey. Atterberg Limit tests were conducted (by others) on 5 samples from BH17-04, BH17-05, BH17-06 and BH17-07. The plasticity index ranged from 8 to 25 corresponding to a relatively low to moderate plasticity.

The Upper Facies was generally stiff to very stiff to hard in consistency based on a review of disturbed soil samples and on uncorrected DCPT blow count values. However at BH17-05, a single SPT blow count of ‘5’ was reported at approximately 3 m depth, indicating a firm consistency.

In portions of the site where bedrock is relatively shallow, the Upper Facies of the Victoria Clay directly overlies glacial till and bedrock. Where bedrock is deeper (e.g. TH18-12), the Upper Facies overlies the softer Lower Facies with a gradual transition between the two.

Victoria Clay - Lower Facies

The Lower Facies of the Victoria Clay deposit was encountered where bedrock was relatively deep, including at TH18-11, TH18-12 and possibly at BH17-05 (see below). It is also inferred to be present at bedrock probe holes TH18-4, TH18-15, TH18-16, TH18-17, TH18-18, TH18-19 and TH18-20 based on the encountered depth to bedrock and visual observations of drilling resistance. The thickness of the Lower Facies increased as the depth to bedrock increased, ranging from approximately 0.75 m thick (BH17-05) to greater than 3 m thick (TH18-12).

The Lower Facies is the same in its gradational components to the Upper Facies (i.e. silty clay). It is typically grey in colour but can be mottled with brown as it transitions from the Upper Facies. Atterberg Limit tests were conducted on 2 samples selected from TH18-12. The plasticity index ranged from 32 to 41 corresponding to a relatively high plasticity.

Based on the 2017 investigation results, the Lower Facies was not anticipated at this site. An SPT blow count of ‘2’ was recorded in a clayey gravelly sand deposit encountered at BH17-05. This deposit could be glacial till (disturbed by drilling) or a local variation of the Lower Facies clay.

The consistency of the Lower Facies typically decreases with depth from stiff to soft. Two shear vane tests were completed within the Lower Facies of the clay at TH18-12 which indicated peak undrained shear strengths of approximately 75 kPa (stiff) at 5.2 m and approximately 45 kPa (firm) at 6.7 m depth. It is therefore much more compressible than the other materials encountered at this site and has relatively low bearing strength.

The Lower Facies of the Victoria Clay overlies glacial till (where present) and bedrock.

Glacial Till

Glacial till was encountered in auger and sonic test holes immediately overlying bedrock at BH17-07, TH18-10, and TH18-11. Where encountered, the thickness of the glacial till varied from about 0.2 m (BH17-07) to 2.6 m (TH18-11). It is also inferred to be present in the deeper bedrock probe holes but could not be confirmed with that method.

The glacial till varies from sandy clay to silty sand and gravel and is much more permeable than the overlying clay. High groundwater pressures are sometimes encountered within the till layer.

The glacial till is typically relatively dense where not disturbed. DCPT refusal was encountered within the till at several locations.

Glacial till deposits in the Victoria area and at this site are highly variable in composition, thickness, and density.

<u>Other Soils Near Surface</u>

At TH18-09, a 1 m thick layer of relatively dense sand was encountered near ground surface overlying shallow bedrock (possible Capilano beach deposit). This sand layer was not encountered in any of the 7 other sonic and auger test hole locations. This highlights the local variability of soil conditions and potential for unidentified soil conditions to be encountered during excavation, especially near ground surface.

4.3 Bedrock Conditions

The depth to bedrock was identified in the sonic and bedrock probe holes. The depth to bedrock cannot be decisively interpreted from the auger test hole data; however, we infer that auger refusal occurred within glacial till or at the bedrock surface. The interpreted depths to bedrock and auger refusals are provided on the test hole logs and on Dwg. 22952-1.

Bedrock was relatively shallow across the west portion of the site. The depth to bedrock increases significantly toward the north-east end of the site, reaching a maximum encountered depth at TH18-3 of 12.2 m within the proposed building footprint and 13.1 m east of the site at TH18-18. The depth to bedrock at this site is very irregular and varies significantly between test hole locations (e.g. increasing in depth by 6.1 m between TH18-15 and TH18-16 which are separated by approximately 7 m).

Bedrock in this area is mapped as Wark Gneiss, which is typically hard, strong to very strong and fractured. The 2017 investigation recovered disturbed rock samples which were identified as fresh to slightly weathered meta-diorite. Highly weathered rock was encountered within 0.5 m of the bedrock surface at BH17-07.

4.4 Groundwater Conditions

In June 2017, groundwater was encountered during sonic drilling and hydro-vacuum operations at BH17-05, BH17-06 and BH17-07 at depths ranging from 1.0 m to 1.4 m below ground surface. In 2018, groundwater was not encountered in the auger test holes or bedrock probe holes at the time of drilling. However, the Lower Facies of the Victoria Clay was wet, especially below about 6 m depth. The glacial till was also wet at TH18-11 at around 5.5 m depth.

A standpipe piezometer was installed at TH18-11 with a screened interval within the glacial till to characterize the groundwater conditions. The groundwater level in the standpipe was measured during installation and on August 7, August 14, 2018, and September 25, 2018. The groundwater levels are shown on the test hole logs and summarized in Table 1 below. It appears that the groundwater level at this location has stabilized approximately 2.3 m to 2.4 m below ground surface, which is higher than wet soil conditions were encountered during drilling. This indicates a mild upward hydraulic gradient between the glacial till and overlying Victoria Clay. A recommended groundwater depth for the structural design of basement walls is provided in Section 5.7.

TABLE 1
GROUNDWATER LEVELS

Test Hole	**Water Level Below Ground Surface (m)**			
	August 1, 2018	August 7, 2018	August 14, 2018	September 25, 2018
TH18-11	4.84	2.43	2.39	2.29

It should be noted that groundwater levels can vary in response to seasonal factors and precipitation, hence the actual groundwater conditions at the time of construction could vary from those recorded during this investigation. The groundwater level in the standpipe piezometer should be measured again prior to construction.

5. GEOTECHNICAL RECOMMENDATIONS

The recommendations provided below are based on the results of the drilling investigation and our understanding of the proposed development at this early stage of design. Changes to the proposed design, building layout or loading may require modifications to the recommendations provided herein.

5.1 Foundation General Arrangement

The available geotechnical information indicates that bedrock will be encountered within the excavation to the proposed foundation elevations within the majority of the site. The proposed raft slab foundation is therefore generally considered geotechnically feasible, except as noted below.

In the northeast portions of the site, the depth of the bedrock surface increases significantly. In this portion of the site, bedrock may not be encountered within the excavation and soils including Lower Facies Victoria Clay or glacial till are anticipated at the proposed raft foundation elevations. The Lower Facies of the Victoria Clay is much more compressible than bedrock or dense glacial till which introduces significant potential for differential settlement. Therefore, piles are recommended to support the slab where Victoria Clay is encountered at foundation elevation.

We understand that the depth of the proposed foundation slab and the anticipated loading of the structure vary significantly. As described above, the subsurface conditions also vary significantly within the building footprint. The result is a relatively complex geotechnical design scenario.

To manage this complexity, we have subdivided the building footprint into 6 geotechnical design zones to organize our recommendations. The locations of the zones and their approximate boundaries are provided in Dwg. 22952-2.

The location of the boundaries between Zone 2 and Zone 4, and between Zone 3 and Zone 5 are approximate interpretations based on the available drilling data and are therefore subject to confirmation during excavation. The distinctive features of the zones are summarized in Table 2.

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TABLE 2
GEOTECHNICAL DESIGN ZONES SUMMARY

Zone	Depth to Base of Foundation Slab (HCMA Dwg.S100)	Encountered Depth to Bedrock Thurber Dwg. 22952-1	Structural Loading Description	Foundation Type Geotechnical Recommendation
1	6.1 m	1.2 m to 4.3 m	Relatively Heavy	Raft Slab on Bedrock
2	6.1 m	0.7 m to 5.2 m	Moderate Load with Uplift	Raft Slab on Bedrock with Uplift Anchors
3	4.1 m	0.7 m to 4.3 m	Moderate Load with Uplift	Raft Slab on Bedrock with Uplift Anchors
4	6.1 m	8.1 m to 12.2 m	Moderate Load with Uplift	Rock Socketed Piles*
5	4.1 m	5.2 m to 11.3 m	Moderate Load with Uplift	Rock Socketed Piles*
6	(At grade)	1.8 m to 4.2 m	Relatively Light	Spread and Strip Footings

* Where glacial till or bedrock exposed at subgrade, Raft Slab with Uplift Anchors.

5.2 Seismic Hazards

The 2012 British Columbia Building Code specifies a seismic hazard with a 2% probability of exceedance in 50 years. In accordance with the seismic hazard values for the 2015 National Building Code of Canada (NBC), the peak ground acceleration (PGA) at the site for the 2,475-year event is 0.58 g for Site Class C conditions. The 2015 NBC Seismic Hazard Calculation output is attached in Appendix C for reference.

<u>Seismic Site Classification</u>

The determination of site classification for seismic site response (BCBC 2012, Table 4.1.8.4.A) is complex at this site where the conditions at the underside of the foundations are anticipated to vary significantly. Site Class B is applicable where bedrock is encountered within 3 m of the foundation elevation as is anticipated to be the case for most of the site. However, Site Class E is applicable near TH18-12 and TH18-19 (Zone 4) and TH18-16 (Zone 5) where more than 3 m thickness of Lower Facies Victoria Clay with plasticity index greater than 20 is anticipated below the foundation elevation.

For seismic design of the structure, we recommend that the BCBC amplification factors (Fa, Fv) be selected as the greater of either Site Class B or Site Class E at each period of interest (i.e. Fa is 1.0 and Fv is 1.7).

This composite Site Class is necessary to avoid reliance on degradation of seismic loads at short periods which may not be realized for significant portions of the site where soft soils are not present. We consider this approach to be appropriately conservative given the complexity of this site and the significance of the proposed structure.

<u>Liquefaction & Cyclic Softening</u>

Liquefaction is not considered to be a potential hazard at this site. However, the Lower Facies of the Victoria Clay deposit will likely soften following a design level earthquake and post-seismic settlements could occur. This settlement would likely be in the order of 0.5% of the thickness of the grey clay and would occur relatively slowly as the excess pore pressures dissipate. The consequences of post seismic settlements in the clay are anticipated to be mitigated by the use of piled foundations where the Lower Facies clay is encountered at foundation elevation.

5.3 Foundation Design – Raft Slab

Design Parameters

Bearing capacity is not likely to be a limiting consideration for the raft slab where bedrock or glacial till are encountered at foundation subgrade. These conditions are anticipated within Zone 1, Zone 2, and Zone 3, and may also be applicable to portions of Zone 4, Zone 5 and Zone 6 if geotechnical review of the subgrade conditions indicates that glacial till or bedrock have been exposed (see 'Raft Slab Subgrade Preparation').

An Ultimate Limit State (ULS) bearing pressure of 250 kPa is considered conservative. This ULS resistance includes a geotechnical resistance factor of 0.5. A raft slab constructed on a layer of engineered fill placed directly on bedrock or glacial till surfaces prepared per our recommendations is anticipated to experience a maximum of 10 mm of post-construction settlement.

The modulus of subgrade reaction is a conceptual linear spring that does not have a unique value. The modulus is not a soil property but does depend on soil properties and other factors. Some factors affecting the modulus include the subsurface profile, soil properties, the size and shape of the loaded area, the magnitude of the loads, and the stiffness of the foundations.

For the design of raft slab foundations on glacial till or bedrock, we recommend using a modulus of subgrade reaction of 50 MPa/m. The recommended modulus of subgrade reaction is not applicable to the pile supported portion of the slab. This modulus of subgrade reaction should be doubled along the edges of foundations and quadrupled at the corners. As recommended in the American Concrete Institute's document 336.2R-88 (1993), the modulus of subgrade reaction should be varied from one-half to five times to assess the sensitivity of the foundation design to the modulus. These moduli are intended for structural modelling of the foundation only and are not suitable for evaluating settlement.

Depth to Groundwater

For the structural design of the raft slab, we recommend assuming a depth to groundwater of 3.5 m below existing grade. We have relied on the installation of a perforated perimeter drain at 3.0 m depth to mitigate the buildup of groundwater in the excavation backfill during intense precipitation events (Section 5.8). If a drain pipe is not installed, a groundwater depth of 2.3 m should be assumed.

During the investigations, shallow perched groundwater was encountered in 2017 at locations where bedrock is shallow. The standpipe piezometer at TH18-11 indicates that groundwater pressures within the glacial till may be higher than the long-term groundwater table. However, the excavation and installation of free-draining backfill is anticipated to modify these conditions. We anticipate that the long-term groundwater table within the backfill will equalize at a depth consistent with the desiccation transition between the Upper and Lower Facies of the Victoria Clay (i.e. 3.5 m depth). During construction, groundwater may be shallower due to hydraulic pressures built up in the glacial till.

Raft Slab Subgrade Preparation

Before placing any fill, the excavated footprint of the raft slab must be prepared. The subgrade surface should be as dry and smooth as possible, and clear of any soft/loose, wet or other deleterious debris. The subgrade conditions are anticipated to vary significantly across the footprint from bedrock to glacial till to clay. The bedrock surface should be blasted to allow for the placement of a minimum 300 mm thick layer of sub-slab fill.

Where the exposed subgrade is blasted bedrock, it should be cleared and cleaned with compressed air to remove loose blasting debris and mud. Where the exposed subgrade is granular glacial till, it should be well compacted. Where the subgrade comprises fine grained material such as silt or clay, it should be cleaned with the flat blade of an excavator, such that all excessively soft, wet, or deleterious materials are removed.

Once the subgrade is prepared, it should be reviewed by Thurber to confirm that the subgrade conditions conform to those anticipated (Geotechnical Hold Point). The subgrade should then be protected from disturbance or water softening.

Sub-slab Backfill

A layer of free-draining, 25 mm minus crushed gravel (with less than 5% passing the 0.075 mm sieve) should be placed directly beneath the slab to provide a level free draining working surface during construction. Following subgrade review by Thurber, we recommend a minimum 300 mm thick layer be placed on prepared subgrades comprised of bedrock or glacial till. This thickness will reduce the potential for cracks to form in slabs due to stiffness contrasts at transitions between bedrock and soil exposed at foundation subgrade.

A thicker working pad is required in portions of the site where the Lower Facies grey clay (firm to stiff) is exposed at foundation subgrade to facilitate pile installation and slab construction (i.e. Zone 4 and Zone 5). We recommend a 750 mm deep sub-excavation of the clay to be replaced with coarse free draining backfill (e.g. controlled blast rock per Section 5.7). A non-woven geotextile (Nilex 4545 or equivalent) should be used to separate the clay from the backfill in order to mitigate punching of the fill into the clay which will increase fill volumes. It is important that the exposed clay subgrade be covered with the geotextile and gravel to mitigate softening and disturbance, immediately after excavation and after review by Thurber. The pad thickness may need to be increased if heavy equipment such as cranes are required in this area.

Sub-slab fill should be compacted to at least 98% of SPMDD in lifts no thicker than 300 mm.

5.4 Foundation Design – Uplift Anchors

Design Parameters

Uplift anchors may be required to resist buoyancy uplift loads on the raft slab, where this load is not resisted by piles. This is anticipated to be applicable in Zone 2 and Zone 3 and potentially in portions of Zone 4 and Zone 5 if till or bedrock are exposed at raft slab subgrade or if dug caissons are used.

Uplift anchors typically consist of double corrosion protected (DCP) Dywidag threadbar grouted into the bedrock. The pullout resistance of an uplift anchor is typically governed by the pullout of a cone of bedrock around the anchor because of structural discontinuities in the rock mass.

The anchor configuration will depend on the design loads and group effects. Thurber should review the uplift anchor design to check the design assumptions, anchor length requirements, and the potential for group effects.

Anchor Installation & Testing

Drilled anchor holes should be flushed thoroughly with compressed air prior to anchor installation. The Contractor should provide mill certifications for anchor bars and should complete minimum daily Quality Control testing of the cement grout used in the anchor installations.

All anchors should be proof tested to 130% of the design load, and at least 10% of the anchors should be performance tested (Geotechnical Hold Point). It is recommended that seismic and buoyancy anchors be locked off at 10% of the design load to reduce slack in the system.

5.5 Foundation Design – Piles

Design Parameters

The raft slab should be supported by piles where the relatively compressible Lower Facies Victoria Clay is encountered at slab subgrade, which is anticipated in the northeast portion of the site (i.e. Zone 4 and Zone 5). Pile lengths will vary with the bedrock surface undulations. The investigation results indicate that bedrock is up to approximately 7 m below the slab elevation (e.g. TH18-3 and TH18-16).

Rock-socketed caissons (drilled piles) are a common foundation type in the Victoria area where uplift loads are anticipated. They consist of a drilled shaft cased with steel pipe which is advanced approximately 1 m into rock to achieve a suitable cut-off from the overlying soils. The shaft is advanced beyond the casing into bedrock to achieve the design socket length, which is measured from the base of the steel casing. A typical detail showing the configuration of rock-socketed piles is provided in Appendix D.

In transition zones where bedrock is relatively shallow below the base of the excavation, ‘dug caissons’ combined with uplift anchors could also be used. The construction of dug caissons involves sub-excavation of clay to a suitable bearing stratum (glacial till or bedrock) and installing a large diameter steel pipe (e.g. corrugated steel pipe) which is filled with concrete. Sub-excavation adjacent to the excavation perimeter will be constrained by shoring requirements. Safe access to the base of the caisson pipe will have to be provided for geotechnical field review of the prepared glacial till or bedrock foundation, prior to filling the pipe with concrete. Uplift anchors could be drilled through the concrete caisson into bedrock.

The ULS in axial compression for rock socketed piles and dug caissons seated on rock is typically greater than the structural capacity of the pile and is typically not limiting to design. We recommend a minimum socket length of 1.5 m. The recommended socket length should be reviewed when the diameter of the pile and loads on the socket have been determined. Uplift resistances can be provided once the pile geometry and loading requirements are provided. Settlement of the clay soils surrounding the pile will likely add a negative skin friction load which should be considered. Longer sockets will be required if the base of the socket is not thoroughly cleaned (see ‘Pile Cleanout Requirements’).

For piles seated on rock or socketed into rock, the pile settlement required to achieve the specified loads is expected to be less than 10 mm (in addition to the elastic compression of the pile). This settlement estimate is dependent on pile installation and cleanout consistent with geotechnical requirements.

<u>Pile Cleanout Requirements</u>

Where piles are installed, every attempt should be made to make the hole as clean as possible as any end bearing capacity greatly increases the factor of safety. Because of the hardness of the rock, it is unlikely that the steel casing can be advanced sufficiently far into the rock to affect a total cut-off of seepage into the hole. For cases where water is seeping into the hole, it is good practice to fill the casing with water and place the concrete using tremie techniques. Placing of the concrete in a socket with infiltrating water would otherwise result in the cement being washed from the mix.

If it is apparent during installation that the rock slope is steep, that there is a considerable amount of muck that cannot be cleaned from the socket, or that the concrete will be contaminated for one reason or another, then the socket should be lengthened accordingly.

Thurber will review the rock socket construction during drilling and after completion of the drilling to confirm that the rock conditions are as anticipated, that the sidewalls of the rock socket have been adequately cleaned, and that the required length of the socket has been obtained (Geotechnical Hold Point). The need to increase the socket length due to steeply sloping rock or inadequate cleaning of the base of the socket can only be made by an experienced field reviewer. The visual inspection requires the use of an underwater colour camera. Generally, the Contractor provides the camera and operates the equipment to provide a video of the socket conditions for approval.

Additional foundation design requirements will need to be discussed with the structural engineer during final design.

5.6 Foundation Design – Shallow Foundations

Design Parameters

In Zone 6, the provided general arrangement indicates slab on grade construction (i.e. no basement or crawlspace) with 3 m by 3 m wide spread footings and 1.8 m wide strip footings at the perimeter wall.

We recommend a minimum 0.6 m depth from finished grade to the underside of footings for frost protection. Bedrock in Zone 6 was encountered between 1.8 m and 4.2 m depth. Therefore, we anticipate that the very stiff Upper Facies Victoria Clay will be exposed in the excavations for these footings.

We recommend an Ultimate Limit State (ULS) bearing pressure of 225 kPa, which includes a geotechnical resistance factor of 0.5. A Serviceability Limit State (SLS) bearing pressure of 150 kPa should be assumed to limit the potential for differential settlements between the foundations on bedrock and the shallow foundations on very stiff clay. A shallow foundation constructed on a layer of engineered fill placed on very stiff clay surfaces prepared per our recommendations is anticipated to experience a maximum of 25 mm of post-construction settlement.

These preliminary recommendations should be confirmed by Thurber prior to final design when the loads on these foundations are available.

Subgrade Preparation

Before placing any fill, the excavated footprint of the footings must be prepared. The subgrade surface should be as dry and smooth as possible, and clear of any soft/loose, wet or other deleterious debris. The subgrade conditions in Zone 6 are anticipated to be very stiff Upper Facies Victoria Clay.

If bedrock is encountered, the bedrock surface should be blasted to allow for the placement of a minimum 300 mm thick layer of sub-slab fill. Where the exposed subgrade is blasted bedrock, it should be cleared and cleaned with compressed air to remove loose blasting debris and mud. Sub-excavation of the Upper Facies Victoria Clay may be required if bedrock is exposed within only part of the excavation for a spread footing.

Once the subgrade is prepared, it should be reviewed by Thurber to confirm that the subgrade conditions conform to those anticipated (Geotechnical Hold Point). The subgrade should then be protected from disturbance or water softening.

<u>Backfill</u>

After the subgrade has been reviewed by Thurber, a minimum 300 mm thick layer of free-draining, 25 mm minus crushed gravel (with less than 5% passing the 0.075 mm sieve) should be placed directly beneath the footings and slab to protect the prepared clay surface from disturbance and to provide a level free draining working surface during construction.

5.7 Basement Wall Design

Permanent basement walls will be constructed above the proposed raft slab. Based on the information provided, we understand that the height of these walls will range from about 3.7 m to 6.1 m. Accordingly, we have calculated the passive, at rest and active lateral earth pressure coefficients for yielding and non-yielding walls under static and seismic conditions which are provided below. Active and passive pressures are only applicable to yielding walls as described below.

The ULS non-seismic sliding resistance of shallow foundations can be taken as 0.4 times the effective stress at the underside of the foundation and the ULS seismic sliding resistance can be taken as 0.5 times the effective stress. The vertical effective stress can be calculated assuming unit weights of 20 kN/m^3 and 10 kN/m^3 for soil above and below the groundwater table; respectively.

Temporary earth pressures for shoring walls are different than for basement walls and will require more detailed assessment when shoring details are known.

Depth to Groundwater

For the structural design of the basement retaining walls, we recommend assuming a depth to groundwater of 3.5 m below existing grade. We have relied on the installation of a perforated perimeter drain at 3.0 m depth to mitigate the buildup of groundwater in the excavation backfill during intense precipitation events (Section 5.8). If a drain pipe is not installed, a groundwater depth of 2.3 m should be assumed.

During the investigations, shallow perched groundwater was encountered in 2017 at locations where bedrock is shallow. The standpipe piezometer at TH18-11 indicates that groundwater pressures within the glacial till may be higher than the long-term groundwater table. However, the excavation and installation of free-draining backfill is anticipated to modify these conditions. We anticipate that the long-term groundwater table within the backfill will equalize at a depth consistent with the desiccation transition between the Upper and Lower Facies of the Victoria Clay (i.e. 3.5 m depth). During construction, groundwater may be shallower due to hydraulic pressures built up in the glacial till.

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Active and At-Rest Earth Pressures

The magnitude of lateral earth pressures acting on basement walls depends on the stiffness of the wall system, the type of backfill materials used, and the width of the excavation that is made to install the walls. It may be possible to reduce the earth pressure loads for narrower shored excavations.

Table 3 provides lateral earth pressure coefficients for various conditions based on the seismic criteria given in the BCBC (2012). The total active earth pressure (a geotechnical load) can be calculated using:

$$P_a = 0.5\ x\ K_a\ x\ \gamma\ x\ H^2$$

Where: P_a = total active pressure in kN/m width
K_a = static active earth pressure coefficient given in Table 2 below
H = depth of foundation below the ground surface (m)
γ = soil unit weight (assume 20 kN/m^3 for dry, granular soil)

TABLE 3
ACTIVE AND AT REST LATERAL EARTH PRESSURE COEFFICIENTS FOR BASEMENT WALLS

COEFFICIENT (unfactored)	**Imported Backfill (compacted to 95% SPMDD)**	**Approved Rockfill (compacted)**
	Sandy Gravel (Φ = 35^o)	Angular 100 mm minus (Φ = 40^o)
Static Active (Ka)	0.22	0.19
Static At-rest (Ko)	0.43	0.36
Seismic Active (Kae)	0.46	0.39
Seismic At-rest (Kae)	0.97	0.79

The parameters given in Table 3 are based on the following assumptions:

- Level ground surface behind the backfilled wall within a distance 'H' of the wall
- No surcharge loads behind the wall within distance 'H' (e.g. structures, stockpiles)
- Wall-backfill interface friction equal to 50% of soil strength
- Temporary cut slopes in soil are 1H:1V or flatter (i.e. no shoring)

Hydrostatic water pressures should be added below the groundwater table (see above, this section) or below the perimeter drains if applicable (Section 5.8).

The static earth pressures should be applied as triangular distributions which increase with depth. Since the seismic pressure distribution is unknown, it is recommended that both a triangular and uniform distribution be considered.

Passive Earth Pressures

The passive earth pressures (a geotechnical resistance) for granular soils can be calculated using a triangular pressure distribution for both the static and seismic load cases.

Table 4 provides passive lateral earth pressure coefficients for various backfilling scenarios. The total passive resistance is calculated as follows:

$$P_p = 0.5 \text{ x } K_p \text{ x } \gamma \text{ x } H^2$$

Where: P_p = total passive resistance in kN/m width
K_p = earth pressure coefficient given in Table 3 below
H = depth of foundation below the ground surface (m)
γ = soil unit weight (assume 20 kN/m^3 for dry, granular soil)

TABLE 4
PASSIVE EARTH PRESSURE COEFFICIENTS
FOR BASEMENT WALLS

COEFFICIENT (unfactored)	**Granular Backfill (compacted to 95% SPMDD)**	**Approved Rockfill (compacted)**
	Sandy Gravel ($\Phi = 35^o$)	Angular 100 mm minus ($\Phi = 40^o$)
Passive (K_p)	3.69	4.60

Wall Movements Required to Develop Active and Passive Pressures

Active or passive earth pressure should only be used if the wall is flexible enough to rotate sufficiently under the applied load. The approximate amount of rotation required for each material type is provided in Table 5 (after CFEM, 2006), where H is the height of the wall and Y is the horizontal displacement of the top of the wall relative to the wall base.

If this movement cannot occur due to structural stiffness or restraint, then the 'at-rest' lateral earth pressures should be used.

TABLE 5
REQUIRED ROTATION FOR ACTIVE AND PASSIVE PRESSURES

Soil Type and Condition	Rotation (Y/H)	
	Active	Passive
Dense sands or gravels (e.g. glacial till or compacted backfill)	0.001	0.020
Loose sands or gravels	0.004	0.060
Very stiff to hard clay (e.g. Upper Facies Clay)	0.010	0.020

Under seismic loading, if sufficient relative movement between the wall and soil can occur, then the seismic active pressure can be used. If sufficient wall movement cannot occur, then the at-rest (rigid) seismic pressure should be used.

Isolated Cantilever Retaining Walls

We understand that relatively small concrete cantilever retaining walls may be required to facilitate site landscaping. However, the location, height and configuration of these walls have not been provided.

Cantilever retaining walls should generally be embedded a minimum of 0.4 m below ground surface in front of the wall to provide sliding resistance and frost protection. A perforated PVC drain pipe wrapped in a non-woven geotextile should be provided behind the wall above the footing, and this drain should be connected to the City storm sewer.

Geotechnical review of the proposed wall geometry and subgrade conditions is required to provide recommended bearing resistances and settlement estimates for engineered walls.

5.8 Perimeter Drainage and Groundwater Lowering

We have relied on the installation of a perforated perimeter drain at 3.0 m depth to mitigate the buildup of groundwater in the backfill during intense precipitation events (Section 5.7). This drain pipe should be connected to a pumped sump in the basement.

Where the Lower Facies Victoria Clay deposit is present, placement of perimeter and sub-slab drains lower than 3.0 m depth will induce long term settlement of areas adjacent to the structure. It is therefore recommended that foundation walls and the raft slab be waterproofed (tanked) and designed to resist hydrostatic water pressures.

Where slab on grade construction is proposed in Zone 6, conventional perimeter drainage is required to mitigate the buildup of water during intense precipitation events. These perimeter drains should be connected to the City storm sewer system.

5.9 Backfill Sources

The sources and gradation of the proposed backfill materials should be provided to Thurber for review.

Reuse of Excavated Materials

Granular soils excavated from the building and parking areas can potentially be re-used as general site backfill provided the material is clean, free of organics and debris, and is not excessively wet. Excavated clay soils are moisture sensitive and should not be used as backfill within the building footprint or within the pavement structure.

Blasted rock could potentially be reused to construct working pads and to backfill retaining walls provided a suitable stockpile location is available during construction. To be suitable for reuse as wall backfill, blasted rock should be screened or crushed to a well graded material with maximum 100 mm diameter rock fragments and less than 5% passing the 0.075 mm sieve. The contractor should provide quality control of the grain size distribution to confirm these requirements have been met. From a project sustainability perspective, there may be potential benefits to the reuse of blast rock on site rather than importing backfill materials from further away.

Topsoil will be stripped from work areas prior to excavation. From a project sustainability perspective, consideration should be given to how to best manage this topsoil (a slowly renewing resource). The City may have immediate use for this topsoil at nearby sites, or perhaps the topsoil could be stored during construction and used during final landscaping of the site. The City should consult with an agricultural soils or plant health specialist for guidance on how to best handle and store topsoil to retain its value as a plant growth medium.

5.10 Pavement Design

Design Parameters

Geotechnical recommendations for parking area pavement structure are provided in Table 6 below. These recommendations assume that the pavement subgrade has been prepared in accordance with our recommendations below.

The light vehicle pavement structure is applicable to parking lots with limited frequency of heavy vehicles such as garbage trucks or delivery vans. The heavy-vehicle pavement should be used where a more frequent volume of truck traffic is anticipated. These pavement recommendations are based on our experience and are intended to provide a balance between performance and cost. These recommendations should be reviewed if specific performance criteria for the pavement are applicable.

TABLE 6
RECOMMENDED PAVEMENT STRUCTURES FOR PARKING AREAS

	Minimum Thickness (mm)	
	Light Vehicles	Heavy Vehicles
Asphalt	60	80
Crushed Base Course	150	200
Select Granular Sub-base	200	250

All granular materials should meet the MMCD specifications for gradation and compaction requirements. Asphalt concrete should also meet MMCD specifications for materials and placement. The fill should be placed in lifts no thicker than 300 mm and compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD).

Subgrade Preparation

The conditions at pavement subgrade are assumed to be favourable, including very stiff clay (Victoria Clay Upper Facies), very dense glacial till, or bedrock. Geotechnical field review is required if other subgrade conditions are encountered.

All existing asphalt and fill materials, as well as, any disturbed or softened native clay must be removed from beneath the parking lot areas. The exposed subgrade surface should be proof rolled to identify any weak or soft areas prior to placing engineered fill. Soft areas should be sub-excavated an additional 300 mm and backfilled with 75 mm minus, sandy gravel fill, with less than 5% passing the 0.075 mm sieve up to base course elevation (or approved reused material from the site per Section 5.7).

If wet conditions are encountered at pavement subgrade, the depth of excavation should increase by a minimum of 300 mm which should be backfilled with 19 mm diameter clear crushed gravel (i.e. containing no sand or fines) fully wrapped with a non-woven geotextile fabric (Nilex 4545 or equivalent) to inhibit the migration of fines.

Where practical, clay and bedrock subgrades should be sloped at a minimum 2% towards ditches or drains to facilitate drainage.

Off-Site Roadworks

We understand that roadway improvements may be required at the Quadra and Pembroke and/or the Quadra and Princess intersections. The scope of these improvements has not been confirmed. Depending on the scope of the improvements, supplementary investigation may be required to confirm the pavement subgrade conditions and existing pavement structure.

5.11 Temporary Excavations

Excavation General Arrangement

The proposed excavation for the raft slab will extend to depths ranging from about 4 m to 7 m below the present ground surface. We understand the City prefers to preserve many of the existing trees at the site. Required setbacks from specific trees are not yet available.

The bedrock at the site is hard and will require blasting for economical removal. We anticipate that temporary blasted rock slopes may be sloped at 0.25H:1V. Spot rock bolting may be required to provide temporary stability to local rock features if unfavourable rock jointing is encountered, but we do not anticipate the need for extensive pattern bolting or other stabilization of the rock slopes provided good quality blasting and pre-shearing is carried out. Geotechnical comments on controlled blasting will be provided with the excavation and shoring design.

Overburden soils above blasted rock slopes may be sloped, if practical within the required setbacks from trees. Temporary excavations in the stiff to very stiff, Upper Facies clay deposit and in the glacial till can typically be sloped at 0.75H:1V to a depth of about 4 m. Flatter slopes may be required if seepage is encountered. The Lower Facies clay typically requires flatter slopes of 1.5H:1V for stability. These slopes will usually remain stable during the construction period provided they are protected from weathering with polyethylene sheeting. Flatter slopes may be necessary if other soil conditions are encountered such as uncompacted fills near surface.

Shoring

To maintain the existing trees, we anticipate that shoring will be required along portions of the east and north perimeters of the site where bedrock is relatively deep. Shoring may also be required along sections of the west and south perimeters depending on the required setbacks from trees and roadways and the local variations in the depth to bedrock.

We recommend a reinforced shotcrete shoring wall tied back with inclined anchors extending into bedrock. This is a commonly used shoring system in the Victoria area for deep excavations. Vertical piles (e.g. steel H-piles) will likely be required to provide bearing support for the shoring wall where it is founded in the relatively weak Lower Facies clay. These piles are typically installed and grouted into holes drilled into bedrock prior to excavation and embedded into the shotcrete facing as it is constructed.

The design of inclined tie back anchors beyond the site perimeter will consider the presence of buried infrastructure on Quadra Street and Pembroke Street. We anticipate that the anchors can be located below the shallow feeder roots of the existing trees. However, deeper tree anchor roots could potentially be encountered in inclined drill holes. As an alternative to inclined tie back anchors, the shotcrete wall could be supported laterally by larger diameter vertical piles installed into bedrock. However, we anticipate that a cantilever pile supported shoring wall would be more expensive to construct. Internal bracing could also be considered but this would likely result in challenges for construction staging within the excavation.

From a project sustainability perspective, consideration should be given to the tradeoffs between preservation of the existing trees and the environmental impacts embodied in the steel and concrete used in shoring systems.

Temporary Groundwater Control

The groundwater conditions encountered during the investigations were described in Section 4.4. During construction, groundwater will seep into the open excavation through the base and side-slopes. We anticipate that the greatest rates of seepage will be encountered where glacial till is encountered. Relatively slow seepage is anticipated from the Victoria Clay deposit (primarily from sand seams within the clay). Seepage is also anticipated at shallow depths (less than 2 m below surface) where groundwater is perched above shallow bedrock or the Victoria Clay. This shallow seepage should be expected to increase following intense or prolonged precipitation events.

The volume of groundwater entering excavations in the Victoria area can typically be controlled using a system of pumps and sumps because of the low hydraulic conductivity of the Victoria Clay deposit. However, in our experience with excavations in this area of the City, groundwater seepage rates are occasionally difficult to manage in excavations which encounter the glacial till which has a significantly higher hydraulic conductivity.

Glacial till is anticipated to be encountered below the groundwater level. The contractor should provide a contingency plan to install temporary dewatering wells if seepage rates become unmanageable. However, dewatering the glacial till has the potential to initiate consolidation of the overlying Lower Facies Victoria Clay and induce settlement of the ground surface adjacent to the excavation.

6. RECOMMENDED FURTHER WORK

6.1 Supplementary Investigation

The geotechnical investigations carried out to date have provided sufficient characterization of subsurface conditions for design to proceed.

<u>Pavement</u>

Consideration should be given to a 1-day supplementary geotechnical investigation during detailed design to investigate pavement subgrade conditions once the location of parking lots and the scope of intersection upgrades have been determined.

6.2 Foundation Detailed Design

We anticipate that geotechnical input during detailed design will include:

- Geotechnical review of the foundation design. Any changes to the proposed design, building layout or loading may require modifications to the recommendations provided herein. This could include review of foundation loads and layouts, lateral pile analyses for piles, and uplift resistances for individual piles and uplift rock anchors, as well as, for group effects. Laboratory consolidation testing and settlement analyses may also be required if smaller structures are considered with shallow foundations.
- Discussions and meetings with City representatives, the structural engineer, and other design team members.
- Completion of BCBC Schedule B forms.

6.3 Excavation and Shoring Design

Once the work area constraints have been confirmed, Thurber will prepare excavation and shoring design drawings. Geotechnical specifications for rock blasting will also be provided.

THURBER

DRAWINGS

Client: City of Victoria
File No.: 22952

NOTES:

1. Test Holes were located by hand-held GPS unit; locations are approximate only.
2. Depth to inferred bedrock using Hydraulic Rock drilling is based on the action of the drill and driller's comments. Refer to geotechnical report for limitations of this drill method.
3. 2017 aerial image from CRD Geospatial Data download site.
4. Base plan from drawing entitled "ACAD-2113-03490-0-V-0-TS-R0 TOCLIENT-Model_DWNSV.dwg" provided by HCMA Architeture, August 14, 2018.

0 5 10 15 20 25 30m

SCALE 1:500

THURBER ENGINEERING LTD.

CITY OF VICTORIA

TEST HOLE LOCATION PLAN

CPWC REPLACEMENT PROJECT
GEOTECHNICAL INVESTIGATION

VICTORIA, B.C.

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
CHS	RRS	SMB	OCTOBER 26, 2018	1:500	22952	- 1	-

Plotted: October 26, 2018

NOTES:

1. Test Holes were located by hand-held GPS unit; locations are approximate only.
2. 2017 aerial image from CRD Geospatial Data download site.
3. Base plan from drawing entitled "ACAD-2113-03490-0-V-0-TS-R0 TOCLIENT-Model_DWNSV.dwg" provided by HCMA Architeture, August 14, 2018.

0 5 10 15 20 25m

SCALE 1:400

THURBER ENGINEERING LTD.

CITY OF VICTORIA

GEOTECHNICAL DESIGN ZONES

CPWC REPLACEMENT PROJECT
GEOTECHNICAL INVESTIGATION

VICTORIA, B.C.

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
CHS	RRS	SMB	OCTOBER 26, 2018	1:400	22952	2	-

22952-CAD001.dwg

THURBER

APPENDIX A

2018 Investigation Results

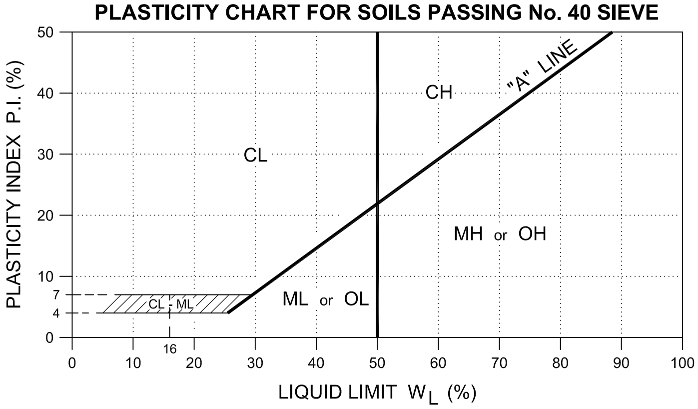
Client: City of Victoria
File No.: 22952

UNIFIED CLASSIFICATION SYSTEM FOR SOILS (ASTM D2487)

THURBER

MAJOR DIVISION			SYMBOLS GROUP	SYMBOLS GRAPH	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
COARSE-GRAINED SOILS (MORE THAN 50% BY WEIGHT RETAINED ON No. 200 SIEVE)	GRAVELS MORE THAN 50% COARSE FRACTION RETAINED ON No. 4 SIEVE	CLEAN GRAVELS (< 5% FINES)	GW		WELL GRADED GRAVEL and WELL GRADED GRAVEL with SAND.	$C_U = \frac{D_{60}}{D_{10}} \geq 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			GP		POORLY GRADED GRAVEL and POORLY GRADED GRAVEL with SAND.	NOT MEETING ABOVE REQUIREMENTS
		GRAVELS WITH FINES (> 12% FINES)	GM		SILTY GRAVEL, GRAVEL - SAND - SILT MIXTURES.	FINES CLASSIFY AS ML or MH (3)
			GC		CLAYEY GRAVEL, GRAVEL - SAND - CLAY MIXTURES.	FINES CLASSIFY AS CL or CH (3)
	SANDS MORE THAN 50% COARSE FRACTION PASSES No. 4 SIEVE	CLEAN SANDS (< 5% FINES)	SW		WELL GRADED SAND and WELL GRADED SAND with GRAVEL	$C_U = \frac{D_{60}}{D_{10}} \geq 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			SP		POORLY GRADED SAND and POORLY GRADED SAND with GRAVEL.	NOT MEETING ABOVE REQUIREMENTS
		SANDS WITH FINES (> 12% FINES)	SM		SILTY SAND, SAND - SILT MIXTURES.	FINES CLASSIFY AS ML or MH (3)
			SC		CLAYEY SAND, SAND - CLAY MIXTURES.	FINES CLASSIFY AS CL or CH (3)
FINE-GRAINED SOILS (MORE THAN 50% BY WEIGHT PASSES No. 200 SIEVE)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	W_L < 50%	ML		INORGANIC SILTS, SILTS with SAND and SILTS with GRAVEL and SANDY or GRAVELLY SILTS.	P.I. < 4 or PLOTS BELOW THE "A" LINE
		W_L > 50%	MH		INORGANIC SILTS, SILTS with SAND & SILTS with GRAVEL & SANDY or GRAVELLY SILTS, FINE SANDY or SILTY SOILS.	P.I. PLOTS BELOW THE "A" LINE
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT	W_L < 50%	CL		INORGANIC CLAYS of LOW PLASTICITY, GRAVELLY, SANDY, or SILTY CLAYS, LEAN CLAYS.	P.I. > 7 and PLOTS ON OR ABOVE THE "A" LINE
		W_L near 50%	CL-CH		BORDERLINE INORGANIC CLAYS and SILTY CLAYS with LIQUID LIMITS NEAR 50%.	(only used for visual identification)
		W_L > 50%	CH		INORGANIC CLAYS of HIGH PLASTICITY, FAT CLAYS.	P.I. PLOTS ON OR ABOVE THE "A" LINE
	ORGANIC SILTS and CLAYS	W_L < 50%	OL		ORGANIC SILTS and ORGANIC SILTY CLAYS of LOW PLASTICITY.	$\frac{W_L \text{ (oven dried)}}{W_L \text{ (not dried)}} < 0.75$
		W_L > 50%	OH		ORGANIC CLAYS OF HIGH PLASTICITY.	$\frac{W_L \text{ (oven dried)}}{W_L \text{ (not dried)}} < 0.75$
HIGHLY ORGANIC SOILS			PT		PEAT and other HIGHLY ORGANIC SOILS.	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE.

PLASTICITY CHART FOR SOILS PASSING No. 40 SIEVE



NOTES:

1. ALL SIEVE SIZES ARE U.S. STANDARD, A.S.T.M. E11-04.
2. COARSE GRAINED SOILS WITH 5 TO 12% FINES REQUIRE DUAL SYMBOLS (GW-GM, GW-GC, GP-GM, GP-GC, SW-SM, SW-SC, SP-SM, SP-SC).
3. IF FINES CLASSIFY CL-ML USE DUAL SYMBOL (GC-GM or SC-SM).
4. WHERE TESTING IS NOT CARRIED OUT, THE IDENTIFICATIONS ARE DETERMINED BY VISUAL-MANUAL PROCEDURES DESCRIBED IN ASTM D2488-06.

SYMBOLS AND TERMS
USED ON TEST LOGS

1. PARTICLE SIZE CLASSIFICATION OF MINERAL SOILS

DESCRIPTION		APPARENT PARTICLE SIZE
BOULDERS		> 200 mm
COBBLES		75 mm to 200 mm
GRAVEL	coarse	19 mm to 75 mm
	fine	4.75 mm to 19 mm
SAND	coarse	2 mm to 4.75 mm
	medium	0.475 mm to 2 mm
	fine	0.075 mm to 0.475 mm
SILT		Non-plastic particles, not visible to the naked eye
CLAY		Plastic particles, not visible to the naked eye

NOTE: Metric Conversion is approximate only

2. TERMS DESCRIBING CONSISTENCY (Cohesive Soils Only)

DESCRIPTION	APPROXIMATE UNDRAINED SHEAR STRENGTH
Very Soft	Less than 10 kPa (250 psf)
Soft	10 to 25 kPa (250 - 500 psf)
Firm	25 to 50 kPa (500 - 1000 psf)
Stiff	50 to 100 kPa (1000 - 2000 psf)
Very Stiff	100 to 200 kPa (2000 - 4000 psf)
Hard	Greater than 200 kPa (4000 psf)

NOTE: Metric Conversion is approximate only

3. TERMS DESCRIBING DENSITY (Cohesionless Soils Only)

DESCRIPTION	STANDARD PENETRATION TEST Number of blows per foot (300 mm) *
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

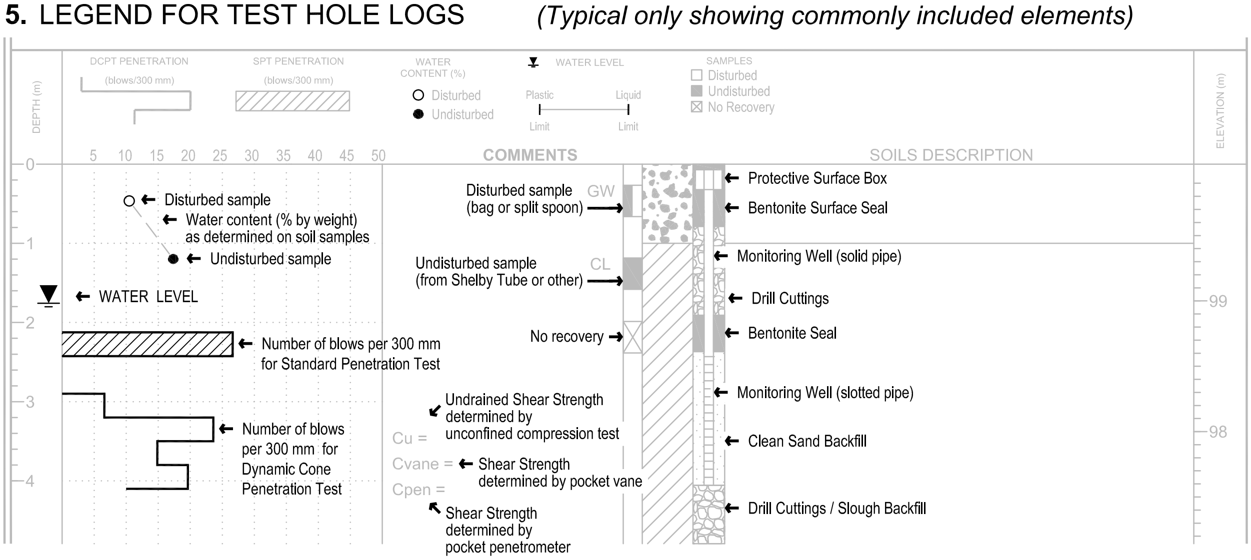
** Directly applicable to sands and, with interpretation, to gravels*

4. PROPORTION OF MINOR COMPONENTS BY WEIGHT

DESCRIPTION	PECENT BY WEIGHT
and	35 to 50 %
y / ey	20 to 35 %
some	10 to 20 %
trace	less than 10 %

EXAMPLE: Silty SAND, trace of gravel = Sand with 20 to 35% silt and up to 10% gravel, by dry weight. (Percentages of secondary materials are estimates based on visual and tactile assessment of samples).

5. LEGEND FOR TEST HOLE LOGS *(Typical only showing commonly included elements)*



Client: TEL Standard Detail
File No.: n/a
E-File: UCSFS-20140729.dwg

Revised: July 29, 2014

LOG OF TEST HOLE

TEST HOLE NO. **TH18-9**

THURBER

LOCATION:	See Drawing 22952-1 N 5364420 E 473596 (Approx.)	**CLIENT:**	City of Victoria
TOP OF HOLE ELEV:		**PROJECT:**	CPWC Replacement Project Geotechnical Investigation
METHOD:	Solid Stem Auger	**DATE:**	1-Aug-2018
DRILLING CO.:	Drillwell Enterprises Ltd.	**FILE NO.:**	22952
INSPECTOR:	JLU		

DCPT PENETRATION (blows/300 mm) · SPT PENETRATION (blows/300 mm) · WATER CONTENT (%): ○ Disturbed ● Undisturbed · WATER LEVEL: Plastic Limit, Liquid Limit · SAMPLES: ◧ Disturbed ■ Undisturbed ☒ No Recovery · GRAIN SIZE (%): ▲ Passing #200 sieve △ Passing #4 sieve · SOIL HEADSPACE READING (ppm): GASTECH reading, PID reading

DEPTH (m) 0–10; scale 10 20 30 40 50 60 70 80 90 100

Depth (m)	COMMENTS	SOILS DESCRIPTION
0		Moist, dark brown, organic SILT (TOPSOIL)
0.2–1.2	DCPT driven from surface to Refusal at 0.6 m depth. SP-SM	Dense to very dense, moist, brown SAND; trace to some silt; trace gravel to 20 mm diameter
1.2		Auger Refusal on probable Bedrock at 1.2 m depth. No water encountered during drilling. Upon completion of drilling: Hole backfilled with cuttings to surface.

LOG OF TEST HOLE

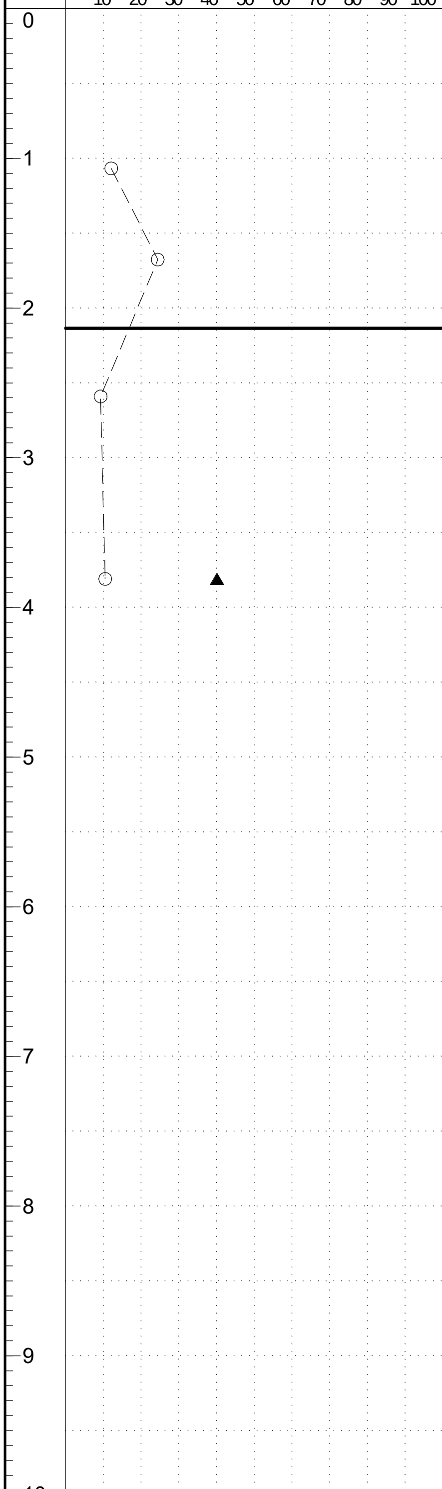
TEST HOLE NO. **TH18-10**

THURBER

LOCATION:	See Drawing 22952-1 N 5364461 E 473557 (Approx.)	**CLIENT:**	City of Victoria
TOP OF HOLE ELEV:		**PROJECT:**	CPWC Replacement Project Geotechnical Investigation
METHOD:	Solid Stem Auger	**DATE:**	1-Aug-2018
DRILLING CO.:	Drillwell Enterprises Ltd.	**FILE NO.:**	22952
INSPECTOR:	JLU		

DEPTH (m) | DCPT PENETRATION (blows/300 mm) | SPT PENETRATION (blows/300 mm) | WATER CONTENT (%) ○ Disturbed ● Undisturbed | WATER LEVEL | Plastic Limit – Liquid Limit | SAMPLES: Disturbed, Undisturbed, ☒ No Recovery | GRAIN SIZE (%): ▲ Passing #200 sieve, △ Passing #4 sieve | SOIL HEADSPACE READING (ppm): GASTECH reading, PID reading | DEPTH (m)

10 20 30 40 50 60 70 80 90 100



COMMENTS

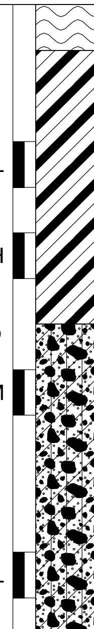
CL

CH

DCPT driven from surface to Refusal at 2.1 m depth.

SP-SM

SM/ML



SOILS DESCRIPTION

Depth (m)	Description
0	Moist, dark brown, organic SILT (TOPSOIL)
	Stiff to very stiff to hard, moist, brown, silty CLAY; medium plasticity; trace gravel to 25 mm diameter
	Very dense, moist, brown SAND becoming SAND and SILT (till-like); trace gravel to 45 mm diameter; trace clay
	Auger Refusal at 4.2 m depth. No water encountered during drilling. Upon completion of drilling: Hole backfilled with cuttings to 1.2 m depth. Bentonite seal installed 0.3 to 1.2 m depth. Backfilled with cuttings to surface.

0 1 2 3 4 5 6 7 8 9 10

LOG OF TEST HOLE

TEST HOLE NO. **TH18-11**

THURBER

LOCATION: See Drawing 22952-1
N 5364481 E 473592 (Approx.)

TOP OF HOLE ELEV:

METHOD: Solid Stem Auger

DRILLING CO.: Drillwell Enterprises Ltd.

INSPECTOR: JLU

CLIENT: City of Victoria

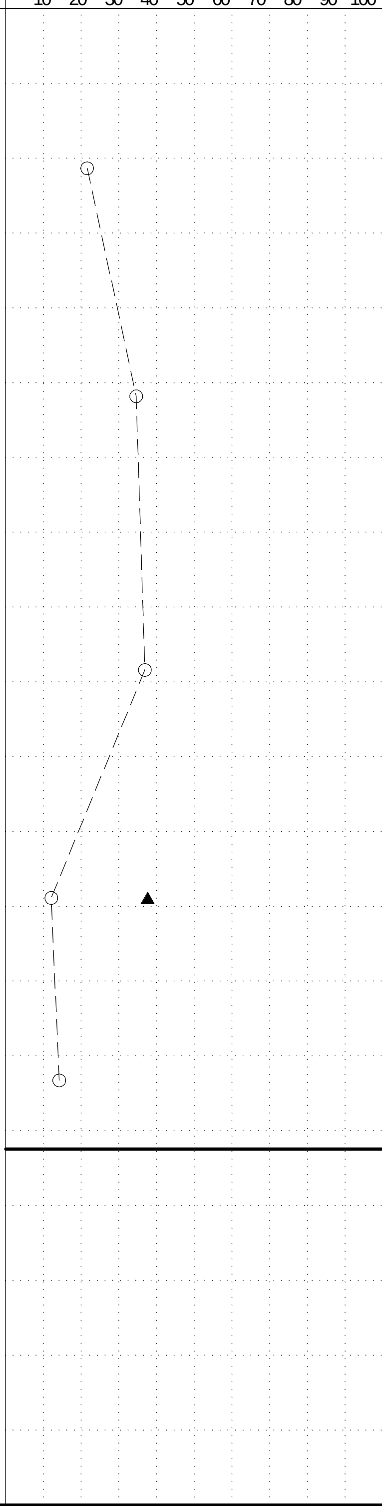
PROJECT: CPWC Replacement Project
Geotechnical Investigation

DATE: 1-Aug-2018

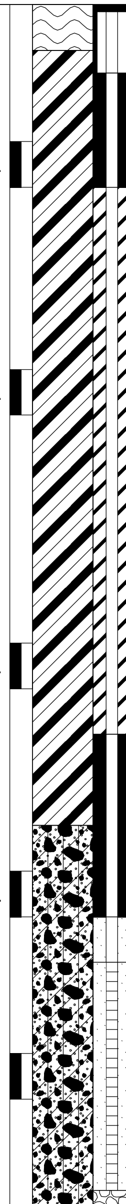
FILE NO.: 22952

DCPT PENETRATION (blows/300 mm) | SPT PENETRATION (blows/300 mm) | WATER CONTENT (%): ○ Disturbed, ● Undisturbed; Plastic Limit – Liquid Limit | WATER LEVEL | SAMPLES: Disturbed, Undisturbed, ☒ No Recovery | GRAIN SIZE (%): ▲ Passing #200 sieve, △ Passing #4 sieve | SOIL HEADSPACE READING (ppm): GASTECH reading, PID reading

DEPTH (m) 0–10; scale 10 20 30 40 50 60 70 80 90 100



Depth (m)	COMMENTS	Sample	SOILS DESCRIPTION
0			Moist, dark brown, organic SILT (TOPSOIL)
0.3			Very stiff to hard, moist, brown, silty CLAY; low to medium plasticity; trace gravel to 20 mm diameter; trace sand
1		CL	
2.3	Water Level (14-Aug-2018) Water Level (7-Aug-2018)		
2.6		CL	
4.3		CL	- becomes stiff at 4.3 m depth
4.8	Water Level (1-Aug-2018)		
5.5			Dense to very dense, moist to wet, grey SAND and SILT (till-like); trace gravel to 30 mm diameter; trace clay
6		SM/ML	
7.2		SM	
7.6	DCPT driven from surface to Refusal at 7.6 m depth.		
8.1			Auger Refusal at 8.1 m depth. No water encountered during drilling. Upon completion of drilling: 50 mm diameter monitoring well installed.



Sheet 1 of 1

LOG OF TEST HOLE

TEST HOLE NO. **TH18-12**

THURBER

LOCATION: See Drawing 22952-1
N 5364467 E 473604 (Approx.)

TOP OF HOLE ELEV:

METHOD: Solid Stem Auger

DRILLING CO.: Drillwell Enterprises Ltd.

INSPECTOR: JLU

CLIENT: City of Victoria

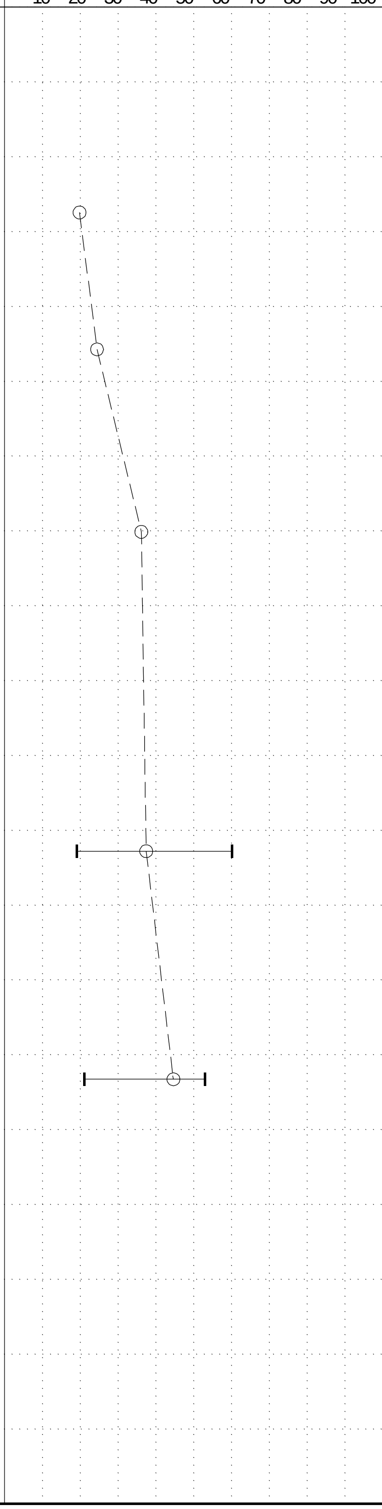
PROJECT: CPWC Replacement Project
Geotechnical Investigation

DATE: 1-Aug-2018

FILE NO.: 22952

DEPTH (m) | DCPT PENETRATION (blows/300 mm) | SPT PENETRATION (blows/300 mm) | WATER CONTENT (%) ○ Disturbed ● Undisturbed | WATER LEVEL | Plastic Limit – Liquid Limit | SAMPLES ◧ Disturbed ■ Undisturbed ☒ No Recovery | GRAIN SIZE (%) ▲ Passing #200 sieve △ Passing #4 sieve | SOIL HEADSPACE READING (ppm) GASTECH reading, PID reading

10 20 30 40 50 60 70 80 90 100



Depth (m)	COMMENTS	Sample	SOILS DESCRIPTION
0 – 0.6			Moist, dark brown, organic SILT (TOPSOIL)
0.6 – 4.6		CH (1.4 m), CL (2.3 m), CL (3.5 m)	Stiff to very stiff, moist, brown, silty CLAY; medium to high plasticity; trace sand - becomes grey-brown at 3.4 m depth
4.6 – 7.6	Vane (p) = 75 kPa Vane (r) = 25 kPa Vane (p) = 45 kPa Vane (r) = 10 kPa	CH (5.6 m), CH (7.2 m)	Firm to stiff, moist to wet, grey, silty CLAY; high plasticity
7.6	DCPT driven from surface to Refusal at 12.2 m depth.		End of Hole at 7.6 m depth. No water encountered during drilling. Upon completion of drilling: Bentonite seal installed at bottom of hole. Hole backfilled with cuttings to 1.2 m depth. Bentonite seal installed 0.3 to 1.2 m depth. Backfilled with cuttings to surface.

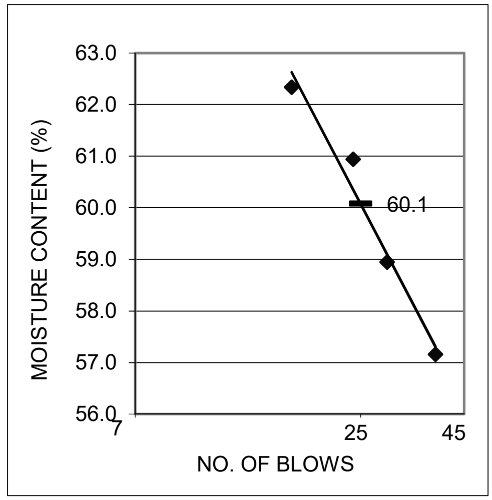
THURBER

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client:	City of Victoria			
Project:	Crystal Pool			
Project No:	22952		**Date Tested:**	8/Aug/18
Test Hole:	TH18-12	Depth: 5.5 to 5.8 m	**Tested By:**	CLV
Sample No:	Sa. 4		**Checked By:**	AGW

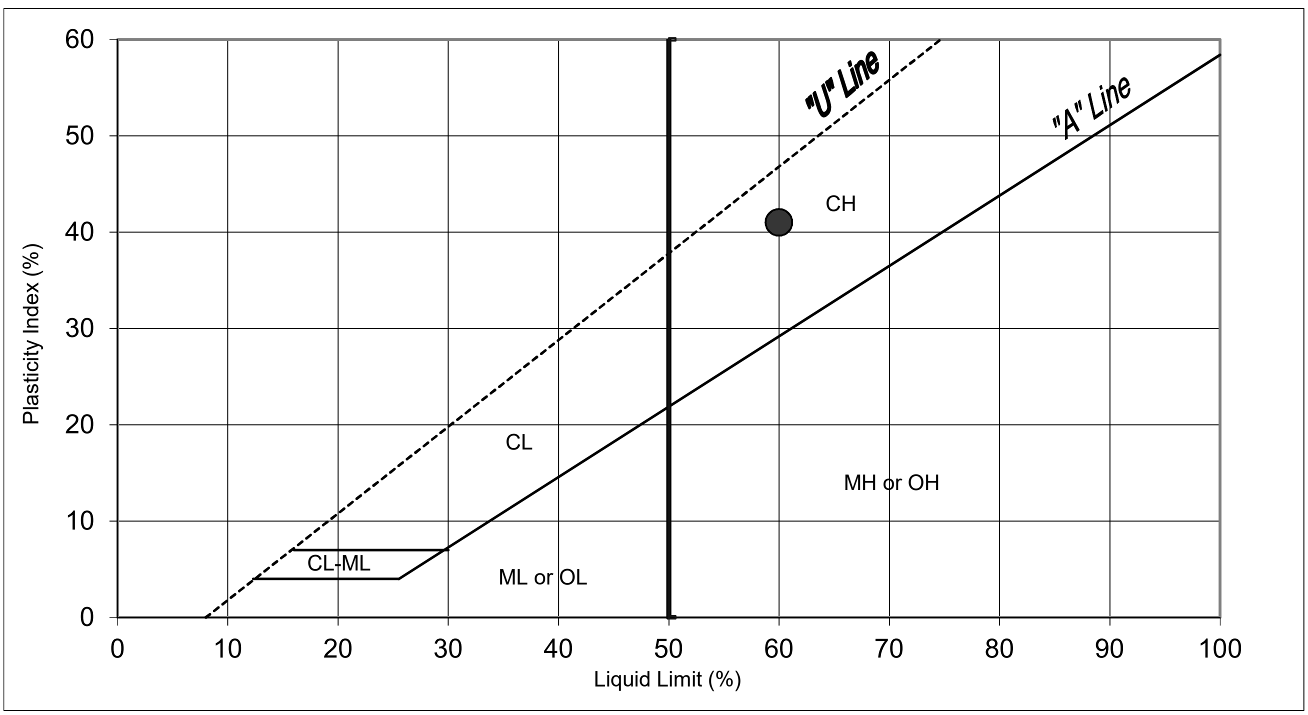
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	38	29	24	17
Container No.	220	219	207	226
Wet Soil + Container	31.49	30.72	29.18	28.29
Dry Soil + Container	24.98	24.66	23.61	22.96
Wt. Of Container	13.59	14.38	14.47	14.41
Moisture Content	57.2	58.9	60.9	62.3



PLASTIC LIMIT

	1	2		**AVERAGE**
Container No.	650	659		
Wet Soil + Container	32.22	32.87		
Dry Soil + Container	30.53	31.06		
Wt. Of Container	21.66	21.87		
Moisture Content	19.1	19.7		**19.4**



REMARKS

Liquid Limit:	**60**
Plastic Limit:	**19**
Plasticity Index:	**41**
USC Classification:	**CH**

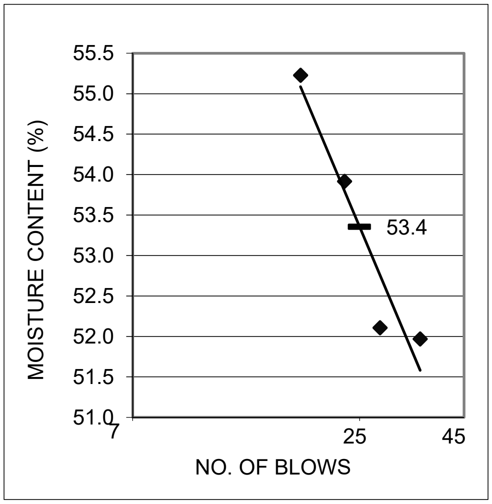
THURBER

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client:	City of Victoria			
Project:	Crystal Pool			
Project No:	22952		**Date Tested:**	8/Aug/18
Test Hole:	TH18-12	Depth: 7.0 to 7.3 m	**Tested By:**	CLV
Sample No:	Sa. 5		**Checked By:**	AGW

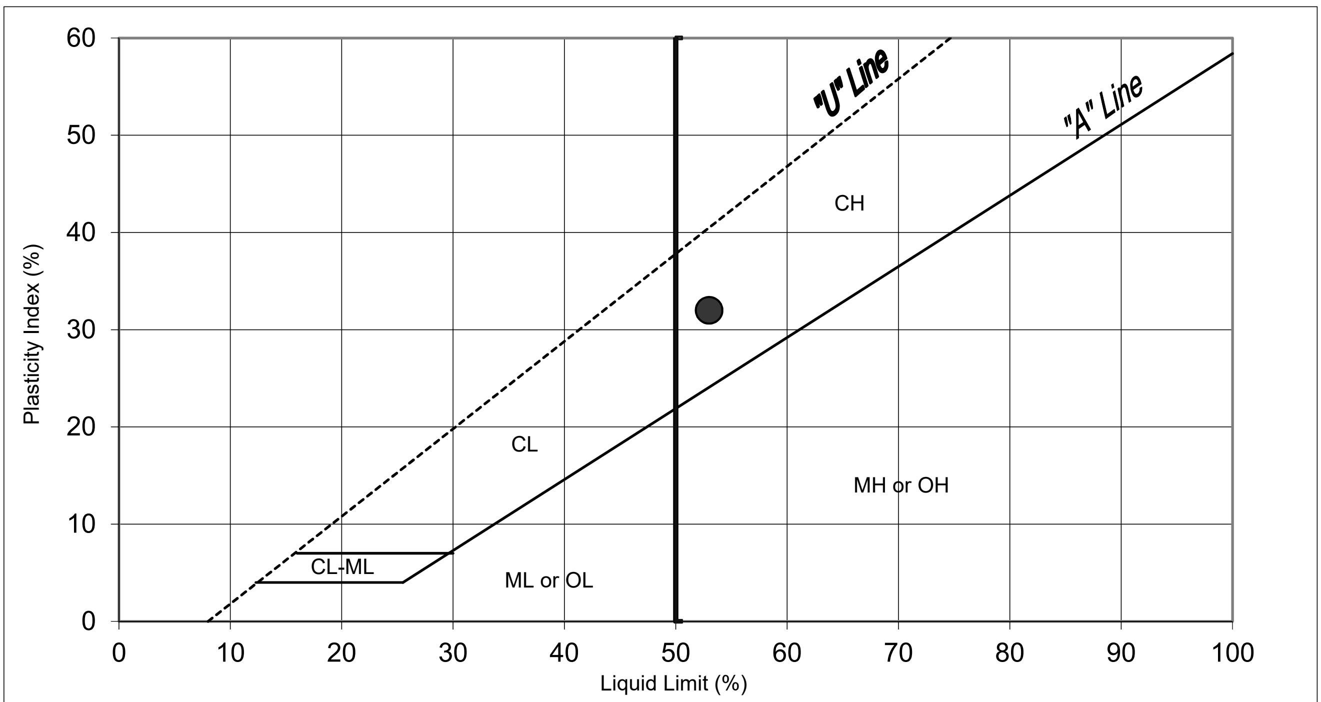
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	35	28	23	18
Container No.	222	233	212	247
Wet Soil + Container	33.69	32.49	30.8	31.14
Dry Soil + Container	27.22	26.06	24.88	24.96
Wt. Of Container	14.77	13.72	13.9	13.77
Moisture Content	52.0	52.1	53.9	55.2



PLASTIC LIMIT

	1	2		**AVERAGE**
Container No.	602	604		
Wet Soil + Container	32.26	30.9		
Dry Soil + Container	30.33	29.14		
Wt. Of Container	20.88	20.97		
Moisture Content	20.4	21.5		**21.0**



REMARKS

Liquid Limit:	**53**
Plastic Limit:	**21**
Plasticity Index:	**32**
USC Classification:	**CH**

THURBER

APPENDIX B

2017 Investigation Results

Client: City of Victoria
File No.: 22952

PROJECT No.: 1671469 / 3000 / 3003

RECORD OF BOREHOLE: BH17-04

DATUM: WGS 84 UTM Zone 10N

CLIENT: City of Victoria
PROJECT: Preliminary Geotechnical Investigation
LOCATION: 2275 Quadra St.
N: ~5364431 E: ~473566
Note: Northing and Easting Coordinates have been determined by GPS in the field and are approximate only.

DRILLING DATE: 12-13 June 2017
DRILLING CONTRACTOR: McRae's Environmental Services Ltd., Drillwell Enterprises Ltd.

DEPTH SCALE METRES	DRILLING RIG	DRILLING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	CORE No.	CORE RECOVERY %	WATER CONTENT PERCENT (Wp – W – Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION
			Ground Surface									
0	Hydrovac	Vacuumed (Casing:254 mm;)	TOPSOIL - **(ML)** sandy SILT, fine to medium sand; black, with organics (rootlets); non-cohesive, moist, compact.	0.00								Filter Sand
	Hydrovac	Vacuumed (Casing:254 mm;)	**(CL/CI)** sandy SILTY CLAY, some rounded gravel; grey mottled with oxidized brown, with organics (rootlets); cohesive, w>PL, stiff.	0.25	1	CS				Wp ~17, W ~20, Wl ~30		Bentonite Chips
1	Geoprobe 8140LS	Sonic (Casing:102 mm;)	BEDROCK, fresh to slightly weathered, dark grey with some iron staining on joint surfaces, medium grained, non-porous to faintly porous, strong, DIORITE with calcite veins (up to ~15 mm thick) (Wark Gneiss Complex).	0.68	2	CS						Bentonite Chips
			End of Borehole.	1.09								
2												
3												

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
WATER CONTENT PERCENT: 10 20 30 40
NP - Non-Plastic

DEPTH SCALE
1 : 15

Golder Associates

SOIL CLASSIFICATION SYSTEM: GACS
LOGGED: AB/KDB
CHECKED: AB

PROJECT No.: 1671469 / 3000 / 3003

RECORD OF BOREHOLE: BH17-05

SHEET 1 OF 1

CLIENT: City of Victoria
PROJECT: Preliminary Geotechnical Investigation
LOCATION: 2275 Quadra St.
N: ~5364444 E: ~473593
Note: Northing and Easting Coordinates have been determined by GPS in the field and are approximate only.

DATUM: WGS 84 UTM Zone 10N
DRILLING DATE: 12-13 June 2017
DRILLING CONTRACTOR: McRae's Environmental Services Ltd., Drillwell Enterprises Ltd.
SAMPLER HAMMER, 140lbs.; DROP, 30in.

DEPTH SCALE METRES	DRILLING RIG	DRILLING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION
0	Hydrovac	Vacuumed (Casing:254 mm;)	Ground Surface TOPSOIL - **(ML)** sandy SILT, fine to medium sand; black, with organics (rootlets); non-cohesive, moist, compact.	0.00				Filter Sand
			(CL/CI) sandy SILTY CLAY, trace rounded gravel; grey mottled with oxidized brown, with organics (rootlets); cohesive, w>PL, stiff.	0.25	1	CS		Bentonite Chips
	Geoprobe 8140LS	Sonic (Casing:102 mm;)			2	CS		
2			**(CI)** SILTY CLAY, some sand; brown mottled with grey; cohesive, w>PL, stiff.	2.13	3	CS		
3					4	SS	5	
					5	CS		
4			**(SC/GC)** CLAYEY SAND and GRAVEL, rounded gravel; grey and brown; cohesive, w<PL, very stiff to hard.	4.11	6	SS	30	Sonic Core
5					7	CS		
			(SC) gravelly CLAYEY SAND, rounded gravel; bluish grey; cohesive, w<PL, very soft to soft.	5.64	8	CS		
6					9	SS	2	
			BEDROCK, fresh, dark grey, medium grained, non-porous to faintly porous, strong, DIORITE with calcite veins (up to ~2 mm thick) and calcite coatings on joint and fracture surfaces (Wark Gneiss Complex).	6.40	10	CS		
7			End of Borehole.	7.01				
8								
9								
10								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
WATER CONTENT PERCENT: Wp W Wl — 10 20 30 40; NP - Non-Plastic
0 0 0 0 ⊕
ADDITIONAL LAB. TESTING
STRATA PLOT
CORE No.
CORE RECOVERY %

DEPTH SCALE
1 : 50

Golder Associates

SOIL CLASSIFICATION SYSTEM: GACS
LOGGED: AB/KDB
CHECKED: AB

PROJECT No.: 1671469 / 3000 / 3003

RECORD OF BOREHOLE: BH17-06

SHEET 1 OF 1

DATUM: WGS 84 UTM Zone 10N

CLIENT: City of Victoria
PROJECT: Preliminary Geotechnical Investigation
LOCATION: 2275 Quadra St.
N: ~5364422 E: ~473516
Note: Northing and Easting Coordinates have been determined by GPS in the field and are approximate only.

DRILLING DATE: 12-13 June 2017
DRILLING CONTRACTOR: McRae's Environmental Services Ltd., Drillwell Enterprises Ltd.

DEPTH SCALE METRES	DRILLING RIG	DRILLING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	CORE No.	CORE RECOVERY %	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); WATER CONTENT PERCENT Wp W Wl (10 20 30 40); NP - Non-Plastic	ADDITIONAL LAB. TESTING	PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION
			Ground Surface										
0	Hydrovac	Vacuumed (Casing:254 mm;)	TOPSOIL - **(ML)** sandy SILT, fine to medium sand; black, with organics (rootlets); non-cohesive, moist, compact.		0.00								Filter Sand
			(CL) sandy SILTY CLAY, some rounded gravel; grey mottled with oxidized brown, with organics (rootlets); cohesive, w>PL, stiff.		0.25	1	CS						Bentonite Chips
1	Geoprobe 8140LS	Sonic (Casing:102 mm;)	BEDROCK, fresh to slightly weathered, dark grey with some iron staining on joint and fracture surfaces, medium grained, non-porous to faintly porous, strong, DIORITE with calcite veins (up to ~3 mm thick) (Wark Gneiss Complex).		1.18	2	CS						
			End of Borehole.		1.52								
2													
3													

DEPTH SCALE
1 : 15

Golder Associates

SOIL CLASSIFICATION SYSTEM: GACS
LOGGED: AB/KDB
CHECKED: AB

PROJECT No.: 1671469 / 3000 / 3003

RECORD OF BOREHOLE: BH17-07

SHEET 1 OF 1

DATUM: WGS 84 UTM Zone 10N

CLIENT: City of Victoria
PROJECT: Preliminary Geotechnical Investigation
LOCATION: 2275 Quadra St.
N: ~5364470 E: ~473522
Note: Northing and Easting Coordinates have been determined by GPS in the field and are approximate only.

DRILLING DATE: 12-13 June 2017
DRILLING CONTRACTOR: McRae's Environmental Services Ltd., Drillwell Enterprises Ltd.

DRILLING RIG	DRILLING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	CORE No.	CORE RECOVERY %	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION
		Ground Surface									
Hydrovac	Vacuumed (Casing:254 mm;)	TOPSOIL - **(ML)** sandy SILT, fine to medium sand; black, with organics (rootlets); non-cohesive, moist, compact.	0.00								Filter Sand
		(CL-ML) gravelly sandy SILTY CLAY to CLAYEY SILT, rounded gravel; grey mottled with oxidized brown, with organics (rootlets); cohesive, w>PL, stiff.	0.25	1	CS						Bentonite Chips
Geoprobe 8140LS	Sonic (Casing:102 mm;)	**(GP-GC/SP-SC)** GRAVEL and SAND, some plastic fines, rounded gravel; grey and some brown; cohesive, w<PL, very stiff.	1.65	2	CS						Sonic Core
		BEDROCK, highly weathered, dark grey with iron staining on joint and fracture surfaces, medium grained, non-porous to faintly porous, strong, DIORITE (Wark Gneiss Complex).	1.83								
		BEDROCK, slightly weathered, dark grey with iron staining on joint and fracture surfaces, medium grained, non-porous to faintly porous, strong, DIORITE with calcite veins (up to ~5 mm thick) (Wark Gneiss Complex).	2.29	3	CS						
		End of Borehole.	2.59								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
WATER CONTENT PERCENT: Wp W Wl; 10 20 30 40; NP - Non-Plastic

DEPTH SCALE
1 : 15

Golder Associates

SOIL CLASSIFICATION SYSTEM: GACS
LOGGED: AB/KDB
CHECKED: AB

Golder Associates

WATER CONTENT DETERMINATION

ASTM D 2216

Client: City of Victoria
Project: Preliminary Geotechnical Investigation
Location: 2275 Quadra St.
Project No.: 1671469 **Phase:** 3000 **Task:** 3003

Project No.: 1671469 **Phase:** 3000 **Task:** 3003
Lab Schedule No.:

Sample Location	Sample No.	Specimen No.	Depth Interval		Water Content (%)
			Depth (m)	Bottom (m)	
BH17-04	**1**		0.51	0.61	**22.0**
BH17-05	**1**		0.61	0.71	**23.1**
BH17-05	**4**		2.74	3.35	**32.6**
BH17-05	**6**		4.27	4.88	**13.6**
BH17-05	**9**		5.79	6.40	**11.5**
BH17-06	**1**		0.41	0.51	**22.6**
BH17-07	**1**		0.51	0.61	**15.9**

National IM Server:GINT_GAL_NATIONALIM Unique Project ID: Output Form:_LAB_WATER CONTENT (REPORT) 2015 dmackie 22/6/17

Checked	Date
DGM	6/22/2017

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2nd floor, 3795 Carey Road Victoria, British Columbia, Canada V8Z 6T8
Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS

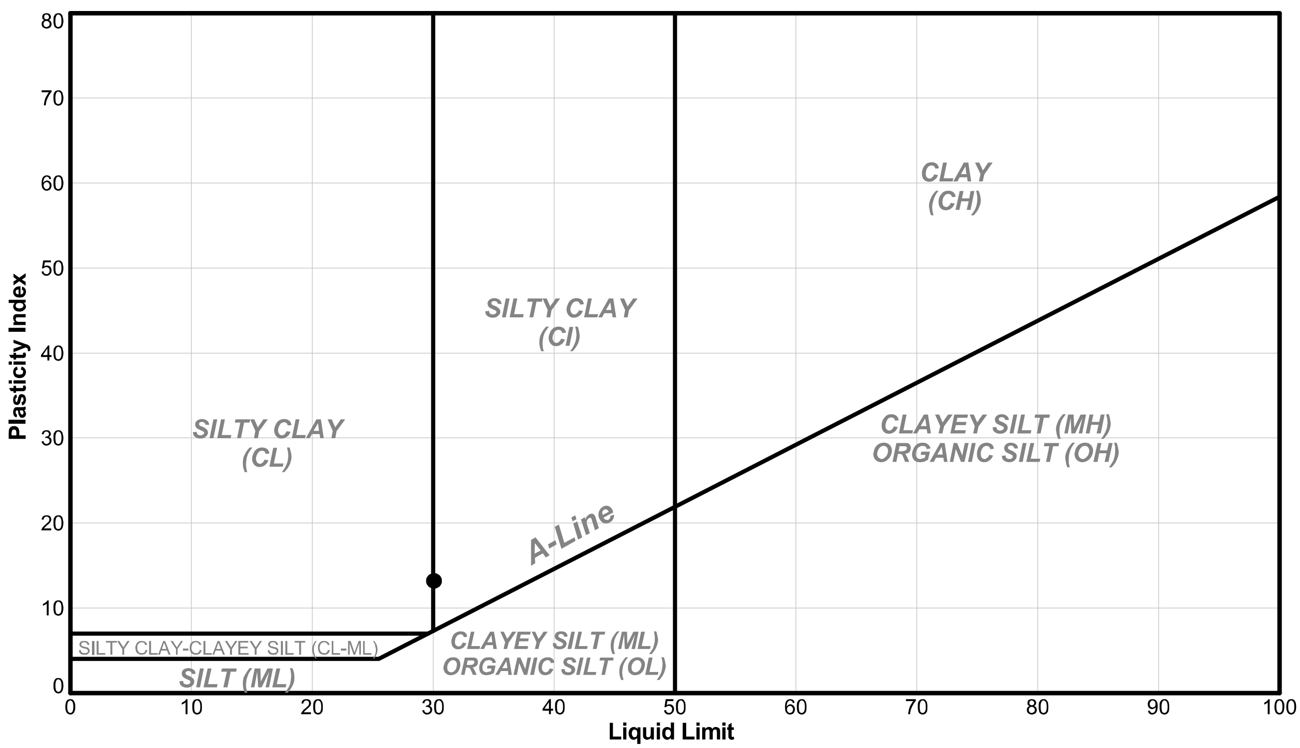
ASTM D 4318-10

Client:	City of Victoria	**Borehole ID:**	**BH17-04**
Project:	Preliminary Geotechnical Investigation	**Sample No.:**	**1**
Location:	2275 Quadra St.	**Depth Interval (m):**	0.51 to 0.61
Project No.:	1671469 **Phase:** 3000 **Task:** 3003	**Lab Schedule No.:**	

Other Remarks: **N/A**

Test Method: A-Multi Point | **Preparation Method: Air Dried**

PLASTICITY CHART



Sym.	Sample Location	Sample / Specimen Number	Depth (m)	Bottom (m)	Percent Passing #40 Sieve (%)	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content (%)	Liquidity Index
●	BH17-04	1	0.51	0.61	ND	30	17	13.0	22.0	0.4

NP - NON-PLASTIC RESULT ND - NOT DETERMINED

KM	**6/19/2017**	**DGM**	**6/22/2017**
Tech	Date	Checked	Date

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LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS

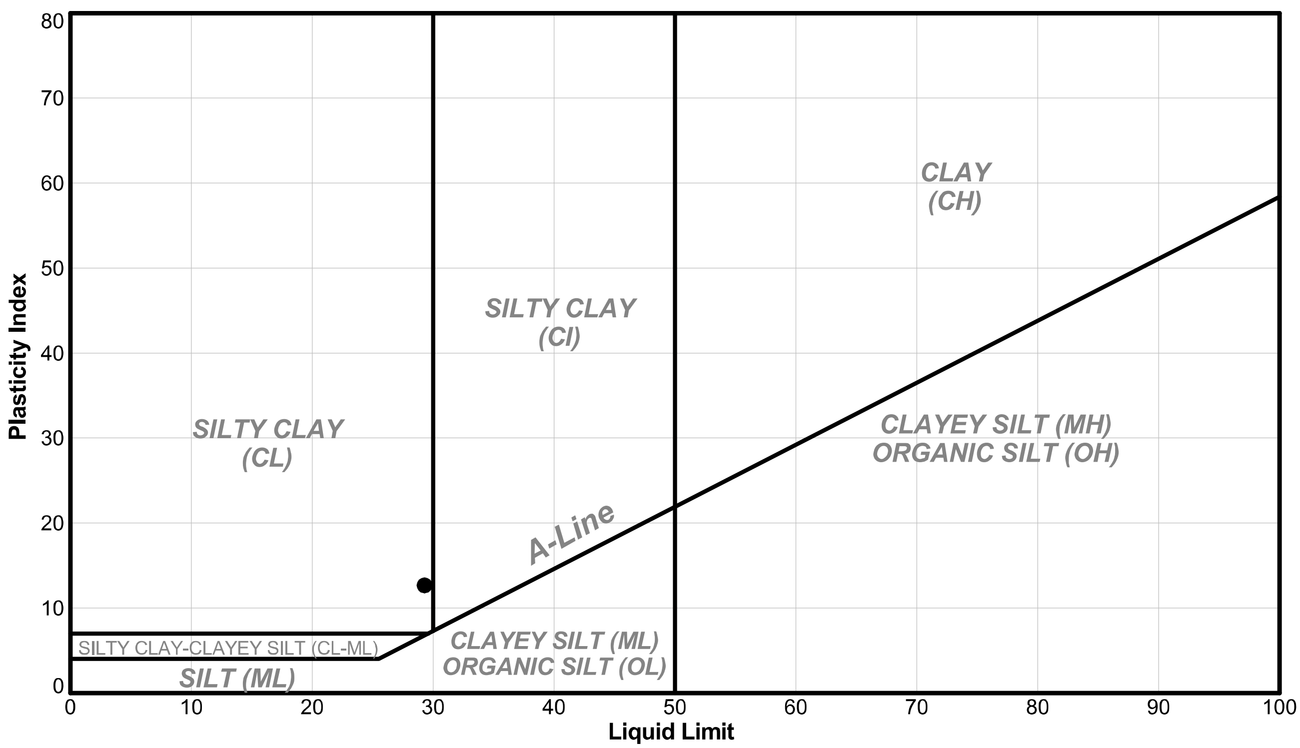
ASTM D 4318-10

Client:	City of Victoria	**Borehole ID:**	**BH17-05**
Project:	Preliminary Geotechnical Investigation	**Sample No.:**	**1**
Location:	2275 Quadra St.	**Depth Interval (m):**	0.61 to 0.71
Project No.:	1671469 **Phase:** 3000 **Task:** 3003	**Lab Schedule No.:**	

Other Remarks: **N/A**

Test Method: A-Multi Point | **Preparation Method: Air Dried**

PLASTICITY CHART



Sym.	Sample Location	Sample / Specimen Number	Depth (m)	Bottom (m)	Percent Passing #40 Sieve (%)	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content (%)	Liquidity Index
●	BH17-05	1	0.61	0.71	ND	29	17	12.0	23.1	0.5

NP - NON-PLASTIC RESULT ND - NOT DETERMINED

KM	**6/19/2017**	**DGM**	**6/22/2017**
Tech	Date	Checked	Date

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Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS

ASTM D 4318-10

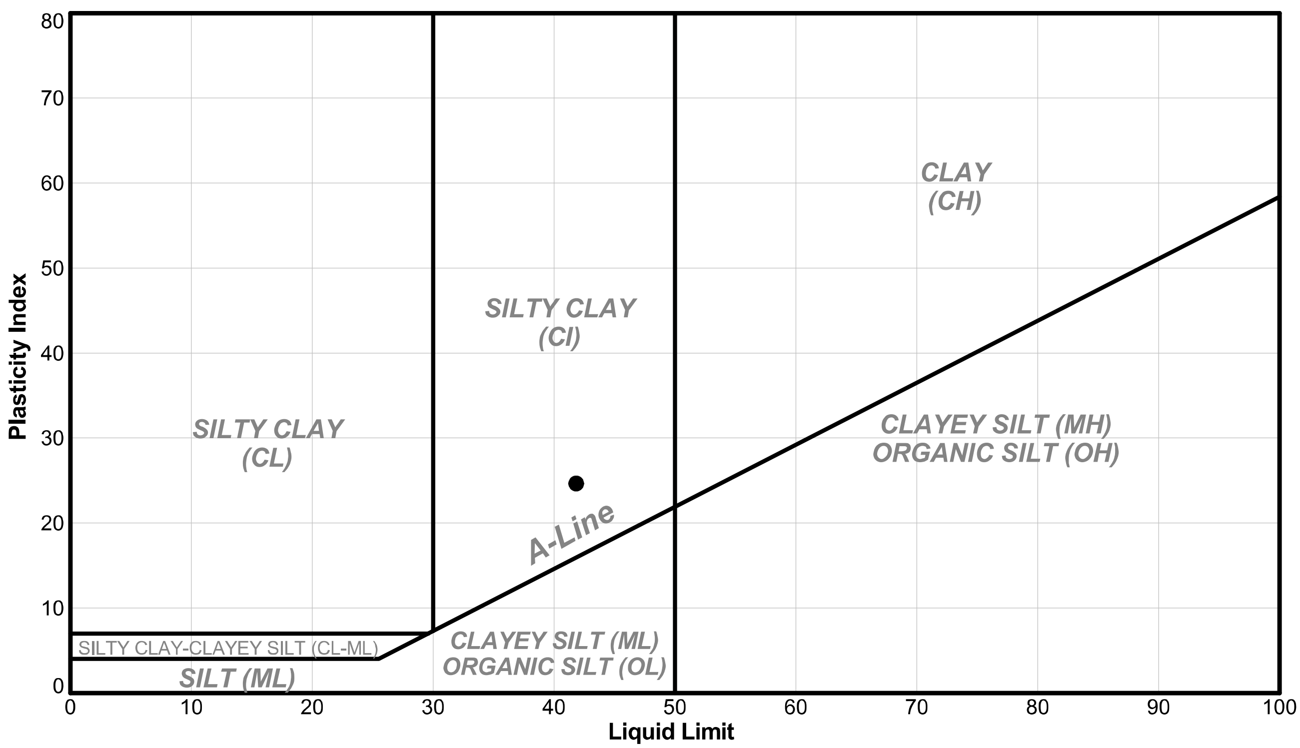
Client: City of Victoria
Project: Preliminary Geotechnical Investigation
Location: 2275 Quadra St.
Project No.: 1671469 **Phase:** 3000 **Task:** 3003

Borehole ID: BH17-05
Sample No.: 4
Depth Interval (m): 2.74 to 3.35
Lab Schedule No.:

Other Remarks: N/A

Test Method: A-Multi Point | **Preparation Method:** Air Dried

PLASTICITY CHART



Sym.	Sample Location	Sample / Specimen Number	Depth (m)	Bottom (m)	Percent Passing #40 Sieve (%)	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content (%)	Liquidity Index
●	BH17-05	4	2.74	3.35	ND	42	17	25.0	32.6	0.6

NP - NON-PLASTIC RESULT   ND - NOT DETERMINED

National IM Server:GINT_GAL_NATIONALIM Unique Project ID: Output Form:_LAB ATTERBERG CASAGRANDE (SINGLE) 2015 dmackie 22/6/17

KM	6/19/2017	DGM	6/22/2017
Tech	Date	Checked	Date

LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS

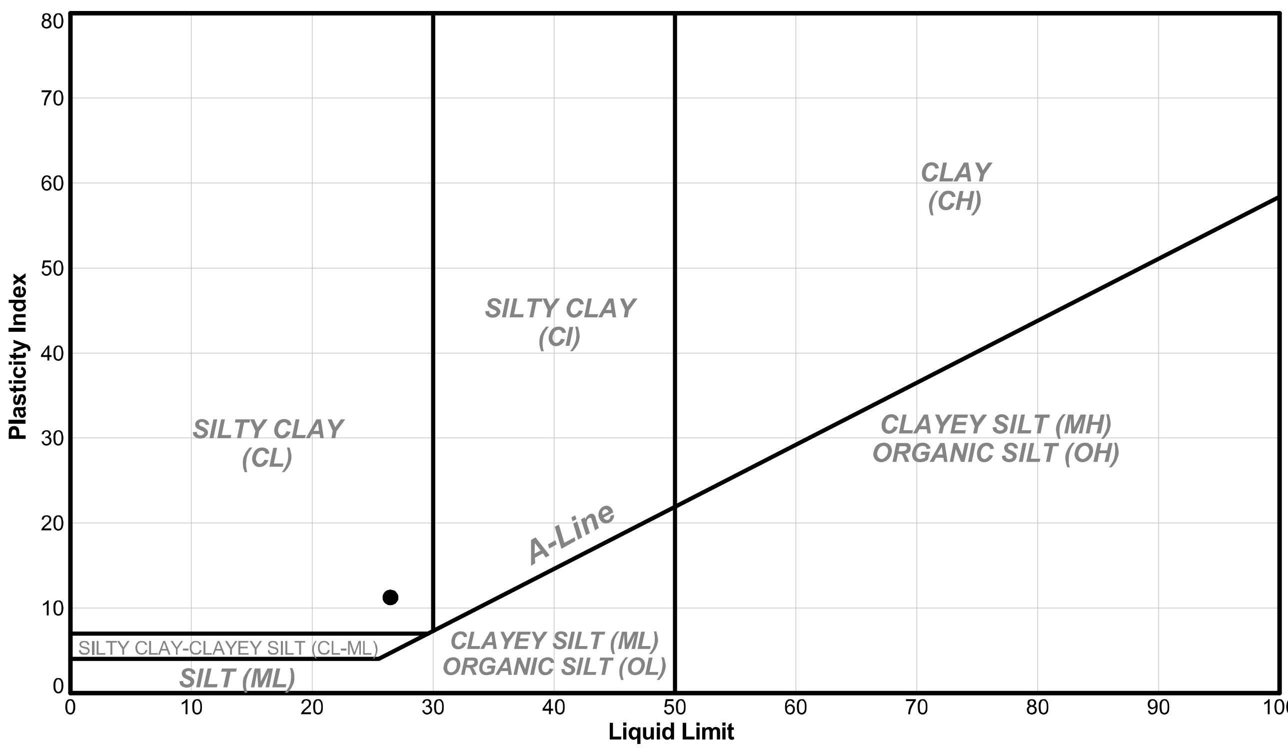
ASTM D 4318-10

Client:	City of Victoria	**Borehole ID:**	**BH17-06**
Project:	Preliminary Geotechnical Investigation	**Sample No.:**	**1**
Location:	2275 Quadra St.	**Depth Interval (m):**	0.41 to 0.51
Project No.:	1671469 **Phase:** 3000 **Task:** 3003	**Lab Schedule No.:**	

Other Remarks: **N/A**

Test Method: A-Multi Point | **Preparation Method: Air Dried**

PLASTICITY CHART



Sym.	Sample Location	Sample / Specimen Number	Depth (m)	Bottom (m)	Percent Passing #40 Sieve (%)	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content (%)	Liquidity Index
●	BH17-06	1	0.41	0.51	ND	26	15	11.0	22.6	0.7

NP - NON-PLASTIC RESULT ND - NOT DETERMINED

KM	**6/19/2017**	**DGM**	**6/22/2017**
Tech	Date	Checked	Date

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Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS

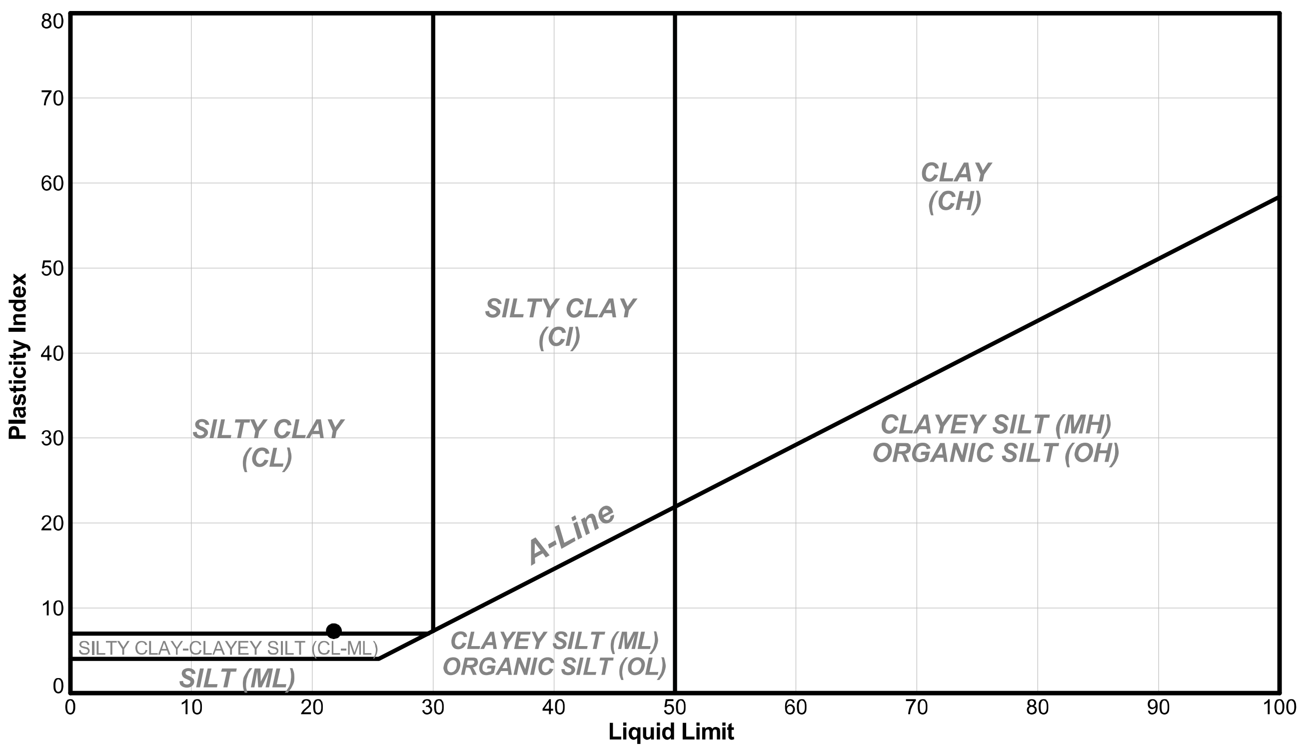
ASTM D 4318-10

Client:	City of Victoria	**Borehole ID:**	**BH17-07**
Project:	Preliminary Geotechnical Investigation	**Sample No.:**	**1**
Location:	2275 Quadra St.	**Depth Interval (m):**	0.51 to 0.61
Project No.:	1671469 **Phase:** 3000 **Task:** 3003	**Lab Schedule No.:**	

Other Remarks: **N/A**

Test Method: A-Multi Point | **Preparation Method: Air Dried**

PLASTICITY CHART



Sym.	Sample Location	Sample / Specimen Number	Depth (m)	Bottom (m)	Percent Passing #40 Sieve (%)	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content (%)	Liquidity Index
●	BH17-07	1	0.51	0.61	ND	22	14	8.0	15.9	0.2

NP - NON-PLASTIC RESULT ND - NOT DETERMINED

KM	**6/19/2017**	**DGM**	**6/22/2017**
Tech	Date	Checked	Date

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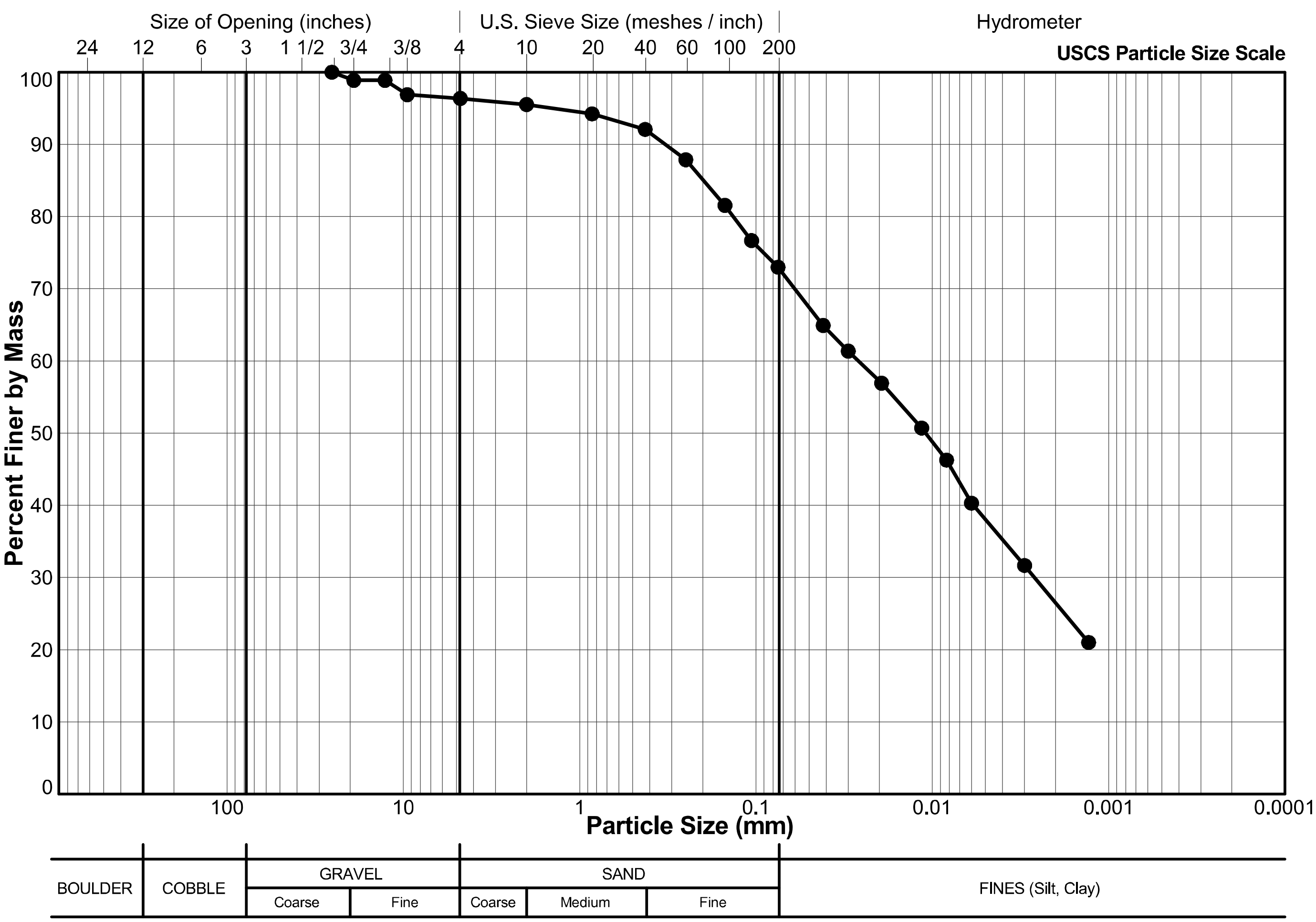
Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

SUMMARY OF PARTICLE SIZE DISTRIBUTION

ASTM D 422

Client: City of Victoria
Project: Preliminary Geotechnical Investigation
Location: 2275 Quadra St.
Project No.: 1671469 **Phase:** 3000 **Task:** 3003

Sample Location: BH17-05
Sample No.: 2
Depth Interval (m): 1.52 to 1.83
Lab Schedule No.:



Legend

Sieve Size (USS)	(mm)	Particle Size (mm)	Percent Passing
1"	25.4		100.0
3/4"	19.1		98.9
1/2"	12.7		98.9
3/8"	9.5		96.9
#4 US MESH	4.75		96.4
#10 US MESH	2		95.5
#20 US MESH	0.85		94.2
#40 US MESH	0.425		92.1
#60 US MESH	0.25		87.9
#100 US MESH	0.15		81.6
#140 US MESH	0.106		76.7
#200 US MESH	0.075		73.0
		0.0416	64.9
		0.0300	61.4
		0.0194	56.9
		0.0115	50.7
		0.0083	46.3
		0.0060	40.3
		0.0030	31.7
		0.0013	21.0

KM	6/16/2017	DGM	6/22/2017
Tech	Date	Checked	Date

National IM Server:GINT_GAL_NATIONALIM Unique Project ID:1659 Output Form: LAB_PARTICLE SIZE (W/ GRADATIONS) 2015_dmackie_22/6/17

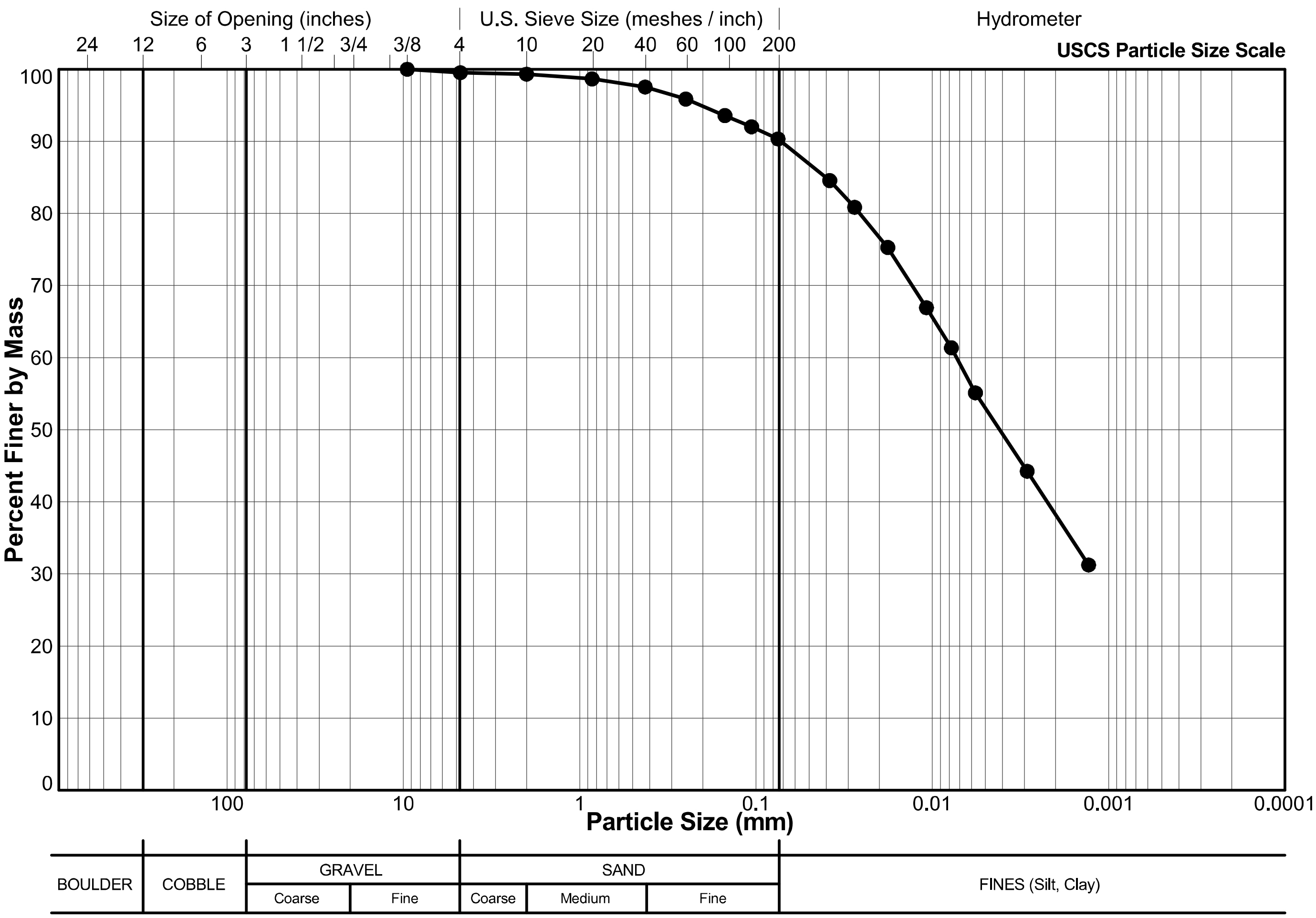
Golder Associates Ltd.
2nd floor, 3795 Carey Road Victoria, British Columbia, Canada V8Z 6T8
Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

SUMMARY OF PARTICLE SIZE DISTRIBUTION

ASTM D 422

Client: City of Victoria
Project: Preliminary Geotechnical Investigation
Location: 2275 Quadra St.
Project No.: 1671469 **Phase:** 3000 **Task:** 3003

Sample Location: BH17-05
Sample No.: 3
Depth Interval (m): 2.59 to 2.74
Lab Schedule No.:



Legend

Sieve Size (USS)	Sieve Size (mm)	Particle Size (mm)	Percent Passing
3/8"	9.5		100.0
#4 US MESH	4.75		99.5
#10 US MESH	2		99.3
#20 US MESH	0.85		98.7
#40 US MESH	0.425		97.5
#60 US MESH	0.25		95.9
#100 US MESH	0.15		93.6
#140 US MESH	0.106		92.0
#200 US MESH	0.075		90.3
		0.0382	84.6
		0.0276	80.9
		0.0179	75.3
		0.0108	66.9
		0.0078	61.4
		0.0057	55.1
		0.0029	44.2
		0.0013	31.2

KM	6/16/2017	DGM	6/22/2017
Tech	Date	Checked	Date

National IM Server:GINT_GAL_NATIONALIM Unique Project ID:1659 Output Form: LAB_PARTICLE SIZE (W/ GRADATIONS) 2015_dmackie_22/6/17

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Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

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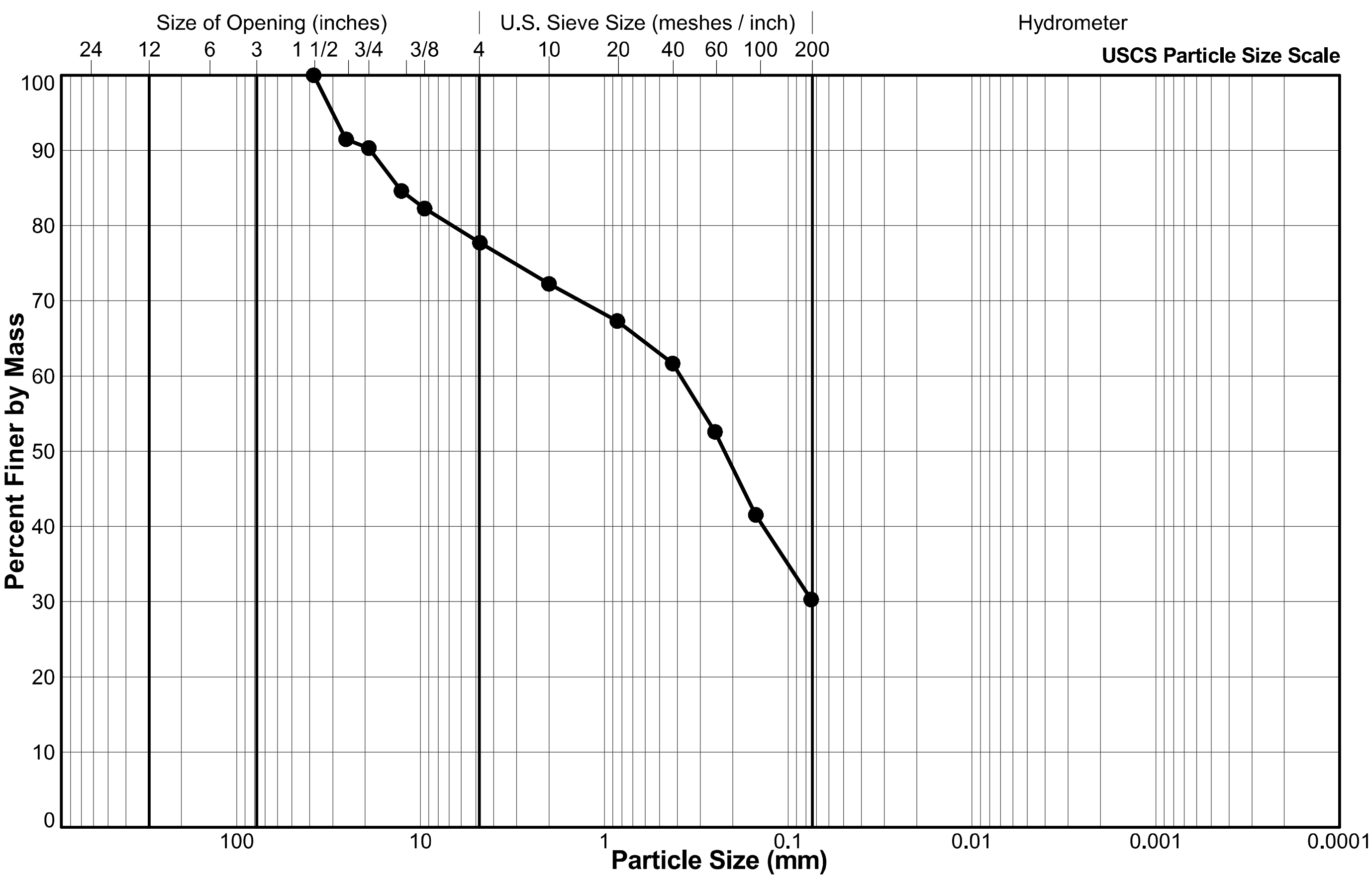
SUMMARY OF PARTICLE SIZE DISTRIBUTION

ASTM C136

Client: City of Victoria
Project: Preliminary Geotechnical Investigation
Location: 2275 Quadra St.
Project No.: 1671469 **Phase:** 3000 **Task:** 3003

Sample Location: BH17-05
Sample No.: 8
Depth Interval (m): 5.64 to 5.79
Lab Schedule No.:



BOULDER	COBBLE	GRAVEL		SAND			FINES (Silt, Clay)
		Coarse	Fine	Coarse	Medium	Fine	

Legend

Sieve Size (USS)	Sieve Size (mm)	Particle Size (mm)	Percent Passing
1 1/2"	38.1		100.0
1"	25.4		91.5
3/4"	19.1		90.3
1/2"	12.7		84.6
3/8"	9.5		82.3
#4 US MESH	4.75		77.7
#10 US MESH	2		72.3
#20 US MESH	0.85		67.3
#40 US MESH	0.425		61.7
#60 US MESH	0.25		52.6
#100 US MESH	0.15		41.5
#200 US MESH	0.075		30.3

KM	6/16/2017	DGM	6/22/2017
Tech	Date	Checked	Date

National IM Server:GINT_GAL_NATIONALIM Unique Project ID:1659 Output Form: LAB_PARTICLE SIZE (W/ GRADATIONS) 2015_dmackie_22/6/17

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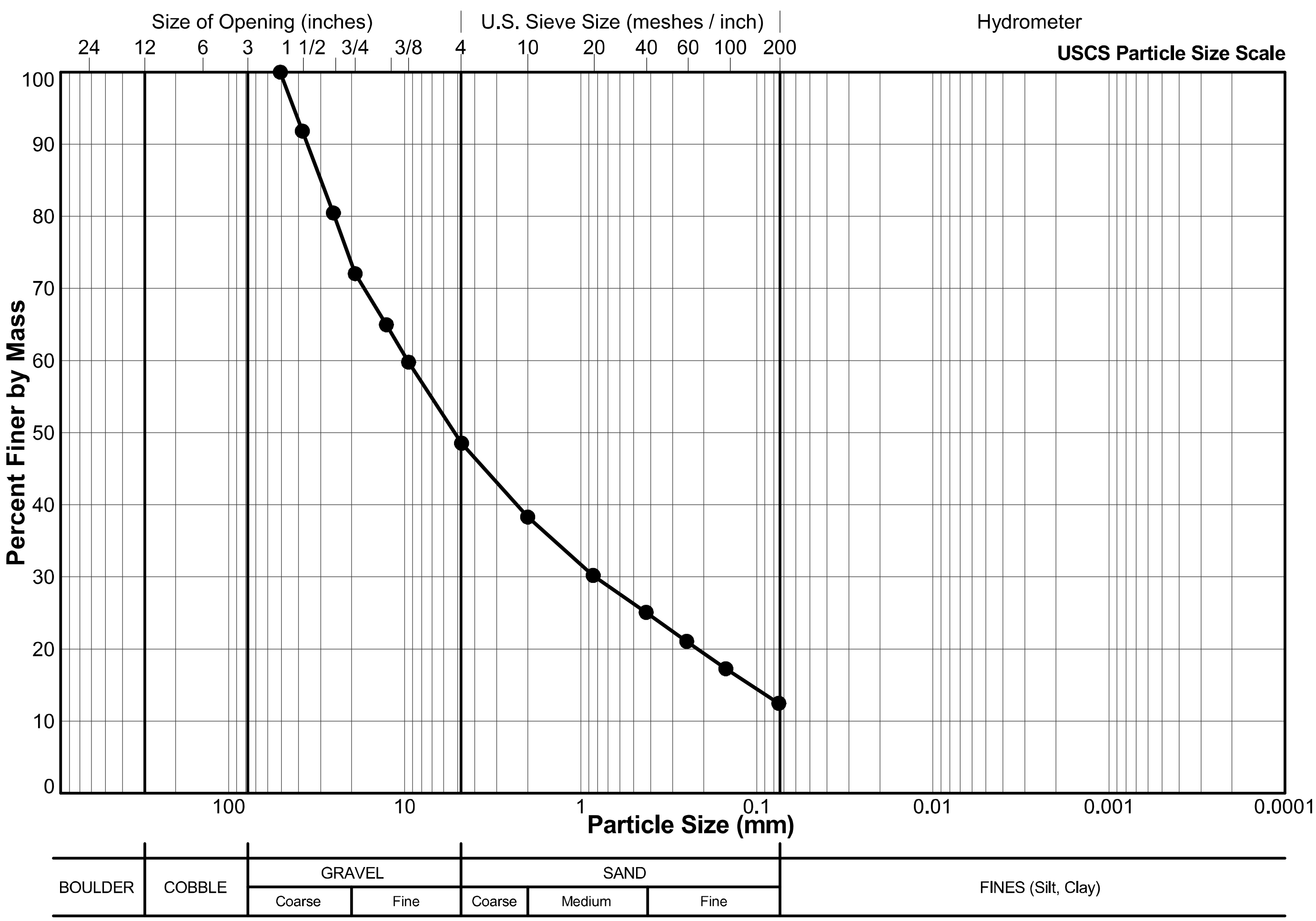
2nd floor, 3795 Carey Road Victoria, British Columbia, Canada V8Z 6T8
Tel: +1 (250) 881-7372 Fax: +1 (250) 881 7470 www.golder.com

SUMMARY OF PARTICLE SIZE DISTRIBUTION

ASTM C136

Client: City of Victoria
Project: Preliminary Geotechnical Investigation
Location: 2275 Quadra St.
Project No.: 1671469 **Phase:** 3000 **Task:** 3003

Sample Location: BH17-07
Sample No.: 2
Depth Interval (m): 1.65 to 1.83
Lab Schedule No.:



Legend

Sieve Size (USS)	(mm)	Particle Size (mm)	Percent Passing
2"	50.8		100.0
1 1/2"	38.1		91.8
1"	25.4		80.5
3/4"	19.1		72.0
1/2"	12.7		65.0
3/8"	9.5		59.8
#4 US MESH	4.75		48.5
#10 US MESH	2		38.3
#20 US MESH	0.85		30.2
#40 US MESH	0.425		25.1
#60 US MESH	0.25		21.1
#100 US MESH	0.15		17.3
#200 US MESH	0.075		12.5

KM	6/16/2017	DGM	6/22/2017
Tech	Date	Checked	Date

National IM Server:GINT_GAL_NATIONALIM Unique Project ID:1659 Output Form: LAB_PARTICLE SIZE (W/ GRADATIONS) 2015_dmackie_22/6/17

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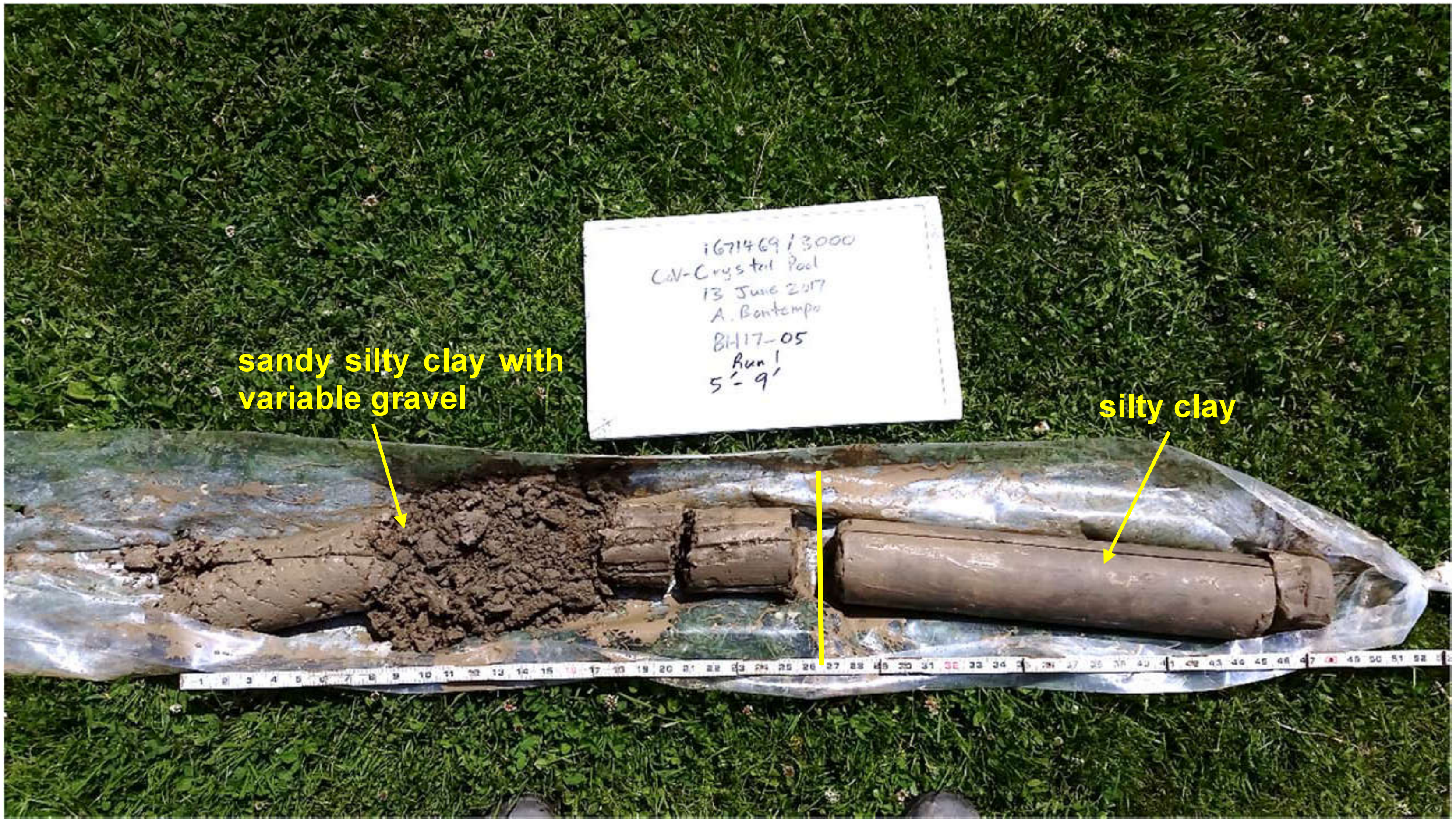
Photograph 1: Mobilizing drill rig to BH17-05, located near the left side of the photograph.

Photograph 2: Topsoil overlying sandy silty clay with variable gravel in the hydrovacuumed upper approximately 1.5 m of BH17-05.



Photograph 3: Sandy silty clay with variable gravel overlying silty clay in a sonic core run from 1.52 to 2.74 m bgs in BH17-05. The contact between the two units was encountered at approximately 2.1 m bgs and is indicated by the thick yellow line in the photograph.

Photograph 4: Clayey sand and gravel in a sonic core run from 1.65 to 1.83 m bgs in BH17-07.

Photograph 5: Clayey sand and gravel overlying gravelly clayey sand in a sonic core run from 4.27 to 5.79 m bgs in BH17-05. The contact between the two units was encountered at approximately 5.6 m bgs and is indicated by the thick yellow line in the photograph.

Photograph 6: Gravelly clayey sand and gravel overlying bedrock in a sonic core run from 5.79 to 6.71 m bgs in BH17-05. The contact between the two units was encountered at approximately 6.4 m bgs and is indicated by the thick yellow line in the photograph.

Photograph 7: Fresh to slightly weathered bedrock sonic core sample collected between approximately 0.7 and 1.1 m bgs in BH17-04. Calcite veins are visible as continuous greyish white strips running along the outside of the sample.

Photograph 8: Slightly weathered bedrock in a sonic core run from 2.13 to 2.59 m bgs in BH17-07. Iron staining is indicated by orange discolouration on joint and fracture surfaces of the rock.

\\golder.gds\gal\burnaby\Final\2016\3 Proj\1671469 CityofVictoria_PIESA_2275QuadraSt\1671469-004-R-Rev0\Appendix B - Photo Summary\Appendix B - Photo Summary.docx

APPENDIX C

NRCAN Seismic Hazard Calculator Output

Client: City of Victoria
File No.: 22952

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

July 27, 2018

Site: 48.4325 N, 123.3575 W User File Reference: Crystal Pool Wellness Centre - Replacement Project

Requested by: , Thurber Engineering Ltd.

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.707	1.079	**1.296**	1.296	**1.149**	**0.670**	**0.393**	**0.123**	**0.043**	**0.577**	**0.826**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.165	0.369	0.505
Sa(0.1)	0.253	0.569	0.779
Sa(0.2)	0.308	0.690	0.936
Sa(0.3)	0.304	0.688	0.936
Sa(0.5)	0.249	0.593	0.820
Sa(1.0)	0.118	0.311	0.453
Sa(2.0)	0.061	0.170	0.258
Sa(5.0)	0.012	0.038	0.071
Sa(10.0)	0.0040	0.013	0.024
PGA	0.134	0.305	0.417
PGV	0.149	0.393	0.566

References

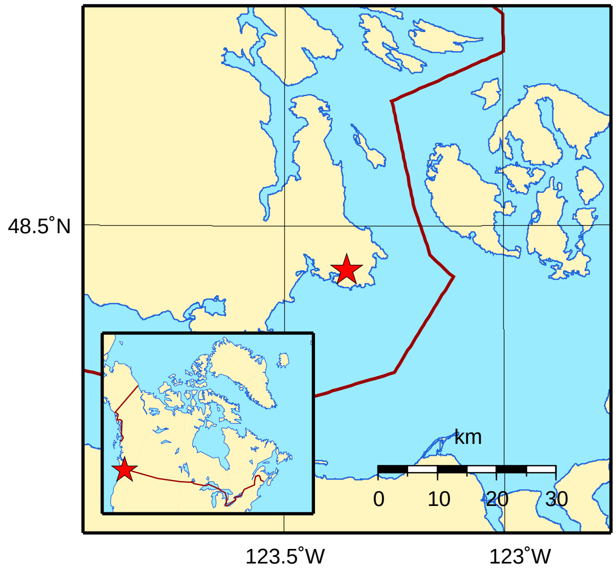
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



APPENDIX D

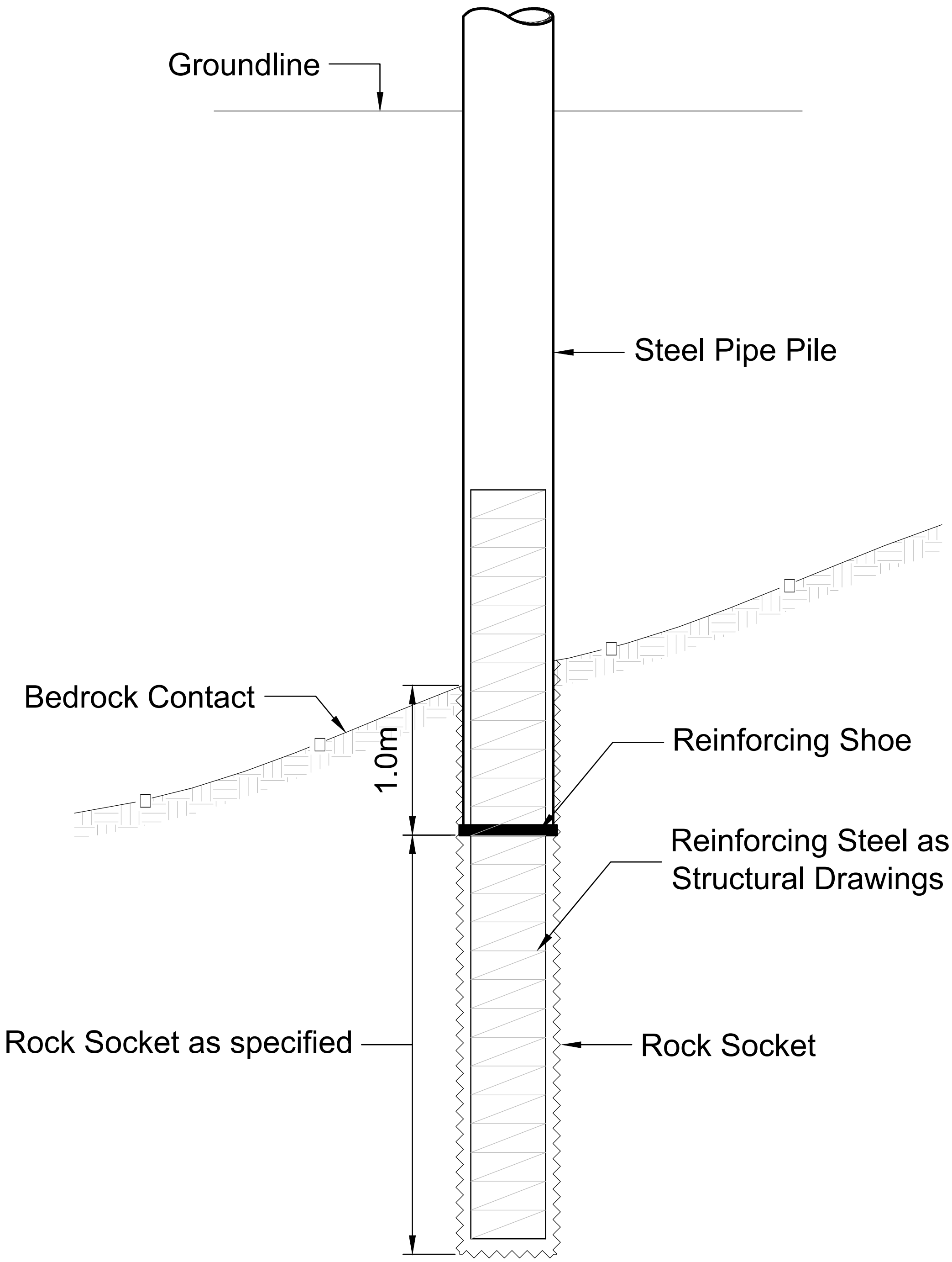
Rock Socketed Caissons Typical Detail

Client: City of Victoria
File No.: 22952

THURBER

ROCK SOCKET DETAIL

(Not to Scale)



Client: TEL Standard Details
File No.: n/a
E-File: Rock Socket Detail_20160405.dwg

Date: April 5, 2016