RYZUK GEOTECHNICAL
Engineering & Materials Testing
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October 26, 2018
File No: 8-8298-2

WestUrban Developments Ltd.
1-1170 Shoppers Row
Campbell River, BC
V9W 2C8

Attn: Frank Limshue (By E-mail: flimshue@westubran.ca)

Re: Proposed Multi-Storey Residential Development
681 & 685 Admirals Road, Esquimalt, BC

INTRODUCTION

As requested we have completed a geotechnical investigation at 681 & 685 Admirals Road to assess the subsurface conditions as such relate to the potential future development of a multi-storey residential development. Our associated results and recommendations are provided herein. Our work has been carried out in accordance with, and is subject to, the attached Terms of Engagement.

PROPOSED DEVELOPMENT

We understand that the purchase of the properties has not been finalized and that the development plans are in the earliest stages. Generally, the proposed work will involve the construction of a multistory residential building as per the concept design drawings produced by THUJA Architects, dated October 1, 2018. The project would consist of a six-storey wood framed structure with a single level of underground parking and parkade foundation walls offset from property boundary between 0.5 to 1 metre (m). No elevations of the concept structure were provided.

INVESTIGATION PROCEDURE

Initially, we completed a brief review of the expected subsurface conditions using the available surficial geology mapping, online GIS databases, and information from our database on projects completed in the area. Additionally, we undertook a brief site visit to assess existing site conditions and to evaluate access for the subsequent geotechnical investigation program.

Our geotechnical investigation consisted of advancing four test holes (TH18-01 to TH18-04). The drilling was completed on October 5, 2018, using a track mounted auger drill rig, supplied and operated by Drillwell Enterprises Ltd. Test holes were all located within the proposed building footprint as shown on the attached Location Plan (8-8298-2-1). The UTM coordinates of each test hole are also presented on the attached test hole logs.

BC OneCall and other private utility owners were contacted prior to the drilling operation to identify existing underground utilities. Private utility locating services were completed by Western Utilities Services Ltd. and no utilities were located within the work zone of each test hole.

The soil types and conditions were assessed visually in the field and were classified according to the Unified Soil Classification System. Representative disturbed samples were taken at appropriate intervals and at soil stratigraphy changes. Select samples were submitted for laboratory testing which consisted of measuring moisture content. An in-situ shear vane was used at select depths within several test holes to measure the undrained shear strength of the soil. Both laboratory and in-situ testing results are presented on the attached test hole logs.

In addition to the drilling investigation, we recorded three micro-tremor readings (MT18-01 to MT18-03) using our Tromino geophysical survey device, which measures and records ambient vibrations at the ground surface. The results from such devices can be analyzed to infer a predominant resonant (natural) frequency of the soil within the general area of the acquisition point. Local Victoria-based studies have further correlated the natural frequency with the thickness of common Victoria soils, depth to bedrock/hard stratum, as well as shear wave velocity. Tromino surveys were carried out at three locations within and around the proposed building area, as shown on the attached Location Plan (8-8298-2-1).

TOPOGRAPHY

The topography of the site generally consists of a moderate southwestward inclined slope, towards Admirals Road, although it is more steeply inclined towards and beyond the northeastern property boundary. In terms of grade differential, there is a moderate rise of approximately 1.7 m from Admirals Road to the residence locations and parking areas, which appear to have been partially leveled. Both lots have raised backyards relative to the house and parking elevation by approximately 1.4 m. The grade of the backyards is generally level except within the northeast corner and along the eastern fence line of lot 685. Beyond the northeastern property boundary, the grade continues to rise at a slope of 1.5 H: 1.0 V (horizontal : vertical) for a vertical distance of roughly 3.3 m. Elsewhere, the slopes are more gradual.

SUBSURFACE CONDITIONS

General

The subsurface conditions at the site generally consisted of asphalt, concrete, fill and/or topsoil overlying brown silty clay, grey clay, glacial till, and bedrock, respectively. This soil stratigraphy was anticipated and is generally consistent with the typical Victoria clay sequence, with marine deposited clay overtop an intermittently present layer of glacial till over bedrock at depth. The bedrock profile underlying the site was found to be dipping southward, though the bedrock surface between test hole locations is typically highly erratic.

Fill

A thin veneer of fill was encountered in TH18-02 and TH18-03 and was approximately 1.1 and 0.7 m thick, respectively. The fills were described as a stiff, brown to black, silt, with organics, as well as a compact, grey, sand with some cobbles underlying a concrete pad associated with vehicle parking.

Topsoil

Topsoil was encountered at the surface in TH18-01, -02, and -04. The topsoil was approximately 0.1 m to 0.3 m thick and was described as silty, sandy, loose and black in colour.

Sand

A thin layer of sand was encountered in TH18-01, - 03, and -04. The sand was generally 0.2 m to 0.5 m thick and described as compact to dense, brown, with trace silt.

Brown Silty Clay

Brown silty clay was encountered beneath the fill, topsoil, and/or sand within each test hole and extended to depths ranging from 0.7 to 4.2 metres below ground surface (m BGS). This material was described as stiff to very stiff, low to medium plastic, and initially dry but increased in moisture content with depth. Based on laboratory testing the moisture content of the brown silty clay ranged from 19% to 25% with an average of 22%. No in-situ shear vane testing was completed within the brown silty clay due to the stiffness of the soil potentially damaging the shear vane apparatus during insertion into the soil.

Grey Clay

Grey clay was encountered in test holes TH18-02 to -04 below the brown silty clay and extended to depths ranging from 6.9 to 13.7 m BGS. The grey clay was described as firm to stiff, silty, damp to moist and medium plastic. Based on laboratory testing the moisture content of the clay ranged from 18% to 33% with an average of 27%. Shear testing was completed at several depths in each test hole except TH18-01 due to shallow refusal in bedrock. The in-situ shear vanes data collected in the grey clay is summarized in Table 1.

Table 1 - In-Situ Shear Vane Data in Grey Clay

Test Hole No.	Depth (m BGS)	Undrained Shear Strength (kPa)
TH18-02	6.4	97
TH18-03	6.4	92
TH18-03	8.0	44
TH18-04	7.8	77

Glacial Till

Glacial till was encountered in each test hole at depths ranging from 0.8 to 13.7 m BGS. The glacial till was sampled in TH18-01 and TH18-02 and was described as dense to very dense silty sand with trace gravel, moist and grey in colour. No in-situ or laboratory testing was completed on glacial till samples.

Bedrock

Bedrock was encountered within each test hole and outcropping bedrock was observed in the north corner of the site. The bedrock was encountered at depths ranging from 0 to 13.7 m BGS. Given the bedrock depths, it can be concluded that the bedrock is sloped across the site and dips to the south. Bedrock in the Victoria area is typically understood to be hard with an unconfined compressive strength (UCS) of approximately 80 megapascals (MPa) and have an erratic profile. No bedrock samples were collected during our investigation nor was any in-situ testing carried out.

Tromino Survey

Results from the Tromino survey varied throughout the site and were noted to contain somewhat irregular frequencies likely due to the reflection off of the sloping bedrock surface. Generally, the recorded depths to till and/or bedrock were consistent with the inferred glacial till depths from the nearby test hole and outcropping bedrock.

GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

Based on the above and our experience in the area, we expect that the proposed development is feasible from a geotechnical perspective but issues pertaining to the near lot line construction and local grades will need to be addressed. In addition, the potential for differential settlement due to foundation loading where the building transitions from bedrock to the thick, compressible clay will need to be assessed in detail as development plans are advanced.

We expect that the final design will in part be based on the cost implications of geotechnical constraints. On one hand, construction of the parkade with slab level somewhat higher than Admirals Road would avoid encroachment agreements and costly shoring systems, however, excessive settlement of the thick clay would then necessitate the use of more expensive caissons for foundation support. On the other hand, excavation of upwards of 3.5 m for the parkade including blasting would allow conventional spread foundations but encroachment and/or shoring systems would be necessary.

Excavation and Shoring

As noted earlier, the preliminary drawings indicate the underground parking extends to near the property line along the north, east, and south boundaries of the site. This would likely require excavation beyond property boundaries particularly to the northeast to accommodate necessary working room for construction of the foundation wall as well as to ensure a stable slope above. In the event that encroachment beyond property boundaries is not permitted, then a subsurface encroachment agreement would be necessary on the neighboring properties to allow installation of tieback anchors for conventional shoring systems. If no subsurface encroachment is feasible, then such would necessitate an internally braced shoring system at least in localized areas or alternatively pulling the parkade away from property boundaries.

Given the subsurface soils observed at the site, we expect temporary cutslopes, including utility trench excavations, will be stable at the following configurations:

- Topsoil/fill materials – 1.0H : 1.0V
- Stiff to very stiff brown silty clay – 0.5H : 1.0V
- Bedrock – near vertical to vertical

Flattening of the noted cutslope configurations may be required due to seepage and pending onsite inspections carried out by a qualified geotechnical professional. In accordance with WorkSafe BC legislation, all excavations deeper than 1.2 m with cutslopes steeper than 1.0H : 1.0V must be assessed and approved by a qualified geotechnical professional prior to worker entry.

Going forward, we believe that a more intensive bedrock probing program across the site would be beneficial to obtain the underlying bedrock profile and aide in the design process: a greater amount of bedrock present around the perimeter of the site would help reduce the cost of the shoring system. We also expect that further design detail with respect to desired grades would be helpful to better assess the feasibility and cost implications of different options.

Seismic Considerations

Greater Victoria is situated in a region of very high seismicity. Considerable earthquake risk exists, stemming from our proximity to the Cascadia subduction zone and numerous more local faults in southwestern BC and northwestern Washington State.

This site is unique in that the foundation subgrade conditions transition from bedrock in the north corner to relatively thick clay deposit in south corner. In accordance with the 2012 and 2015 BC Building Code, the site has a seismic site classification of 'D' unless all foundations are bearing on bedrock then a seismic site classification of 'A' can be considered. For use in design, the Peak Ground Acceleration (PGA) and Spectral Acceleration Values (S(t)) for Site Class 'D', 'A', and the reference Site Class of 'C' are summarized in Table 2. These values have been provided by the 2010 National Building Code Seismic Hazard Calculation, for a 2% probability of exceedance in 50 years.

Table 2 - Summary of PGA and Spectral Acceleration Values (NBC 2010)

Period (sec)	0.2	0.5	1.0	2.0	PGA (g)
Response (g) Site Class 'C'	1.20	0.80	0.37	0.18	0.60
Response (g) Site Class 'D'	**1.22**	**0.90**	**0.42**	**0.20**	**0.60**
Response (g) Site Class 'A'	**0.96**	**0.46**	**0.21**	**0.10**	**0.60**

Note that at the time of writing this report, the 2015 National Building Code (NBC) had been published. Similarly, the 2015 NBC Seismic Hazard Calculation, which includes new methods of calculating hazard values, has been released. The adoption of the 2015 NBC in the form of the British Columbia Building Code occurs in December 2018. Therefore, the 2015 NBC Seismic Hazard Calculation, including the Peak Ground Velocity (PGA) and Peak Ground Velocity (PGV), provided below should be used during design (Table 3).

Table 3 - Summary of PGA, PGV and Spectral Accelerations Values (NBC 2015)

Period (sec)	0.2	0.5	1.0	2.0	5.0	10.0	PGA (g)	PGV (m/s)
Response (g) Site Class 'C'	1.31	1.17	0.69	0.41	0.13	0.04	0.59	0.84
Response (g) Site Class 'D'	**1.29**	**1.29**	**0.83**	**0.52**	**0.17**	**0.05**	**0.52**	**0.92**
Response (g) Site Class 'A'	**0.90**	**0.67**	**0.39**	**0.24**	**0.08**	**0.03**	**0.53**	**0.52**

Groundwater Considerations

Monitoring wells were not installed as part of our investigation; therefore, long-term groundwater data could not be collected. Upon drilling completion, seepage was not observed in any of the test holes indicating that the short-term groundwater response in the localized area around the respective test hole was minimal.

Based on our experience in the project area, the groundwater conditions are likely to fluctuate seasonally. The long-term groundwater table elevation is typically taken as the transition elevation from brown to grey clay. However, perched groundwater conditions are common in the Esquimalt area due to pockets or seams of more permeable material storing infiltrating surface water. Given topographical conditions of the site area, groundwater seepage will likely flow southwestwards within the upper more permeable soils. This will be most impactful along the northeastern edge of the site during bulk excavation work.

Construction Dewatering

Given the topographical conditions of the site, rainwater runoff and groundwater seepage should be expected to enter the site, primarily along the northeastern edge. Accordingly, a diversion and/or dewatering system should be implemented and maintained during construction. Infiltrating water should not be allowed to pond within open excavations or flow down an exposed soil surface of open excavations causing erosion. The amount of infiltrating water to the site will fluctuate seasonally, but any dewatering system should be designed to handle considerable volumes during the wetter months of the year.

Permanent Drainage

We expect that conventional perimeter foundation drainage tied into a free draining granular backfill material (drain rock) will be sufficient to maintain a locally low groundwater table and prevent the buildup of hydrostatic pressure. Non-woven geotextile filter fabric should be placed between the drain rock and backfill material to prevent the migration of finer particles into the voids in the drain rock.

It may be advantageous to locate the perimeter drains on the interior of the perimeter foundation walls in order to minimize excavation requirements. In order to maintain drainage continuity with exterior drain rock, weep holes through the foundation walls will be required. Weep holes should be 75 to 100 mm in diameter and spaced at 1.5 to 2.0 m intervals. The weep holes should be situated well below the base of slab level yet above the elevation of the perimeter drains. Drain rock or piping should be used to create a continuous drainage path beneath the slab to the interior perimeter drain pipe.

Any structure such as mechanical/elevator pits that extend below the level of the perimeter drainage must either be water proofed and able to accommodate hydrostatic pressure or be provided with stand alone perimeter drainage system.

Slab Considerations

Use of a grade supported floor slab for the proposed development's parkade is considered feasible provided all loose/disturbed material is removed from the building footprint. A minimum 150 mm of 19 mm minus crushed rock is recommended immediately beneath the slab, as well as a conventional subslab moisture barrier to minimize capillary rise of moisture into the slab. All subslab fill material should be compacted to at least 95% of Standard Proctor Maximum Dry Density (SPMDD).

Settlement Considerations

The settlement potential of the very stiff brown silty clay encountered at the site is negligible for most residential construction projects because the material has become well consolidated due to desiccation making it unresponsive to moderate additional loading. However, the underlying firm grey clays are near-normally consolidated making them far more susceptible to settlement when loaded beyond their natural stress regime. Therefore, careful consideration will be required to confirm that the site has been adequately unloaded to not surpass the natural stress regime of the firm grey clays.

Additionally, given that the northern portion of the proposed building will be founded on bedrock and the remainder of the building will be founded on stiff to firm clays, the potential for differential settlement could exist. Further review of this issue will be necessary once the proposed building loads and elevations have been identified.

Building Foundation

Depending upon building elevations, we expect that the footings would be bearing partially on the native stiff to very stiff silty clay and partially on blasted bedrock or engineered fill atop either material. Footings bearing directly on the stiff to very stiff silty clay soil, or engineered fill placed atop such, can be dimensioned using the Serviceability and Ultimate Limit State bearing resistances of 150 and 225 kPa, respectively. However, if footings are placed lower than 4.0 m below the house entrance and parking area grade the bearing resistance of the native soils is anticipated to decrease significantly with depth. For footings bearing directly on bedrock, or engineered fill placed atop such, can be dimensioned using a Serviceability Limit State and Ultimate Limit State bearing resistance of 350 and 500 kPa, respectively.

Any disturbance to native soils by construction activities should be corrected by stripping the disturbed area down to undisturbed native soil before placing formwork for footings. For frost protection, the base of all footings should extend to a depth of at least 450 mm below adjacent finished grades.

Engineered Fill

Any fill material required for support of foundations must be placed upon approved subgrade in suitably thin lifts (no greater than 0.3 m) and compacted to a minimum of 95% of SPMDD. The maximum lift thickness is dependent upon the fill material and compaction equipment used. The fill must have a footprint that extends horizontally beyond the footings a distance equal to the thickness of the engineered fill, in order to provide adequate splay for foundation loads.

Foundation and Retaining Walls

Lateral loads on foundation and retaining walls can be calculated using the following guidelines and equations. Where the grade elevation differs significantly between the two sides of a wall, and the wall is free to rotate in order to develop the active earth pressure state (rotation of 0.1% of the wall height), the wall should be designed to resist a lateral earth pressure (due to granular backfill) similar in magnitude and distribution to that of a fluid having density of 6.3 kN/m^3. Lateral earth pressures due to floor loadings and/or foundation loads from adjacent portions of the building can be calculated assuming a lateral coefficient of 0.35. Where the wall cannot rotate, it should be designed to resist an at rest lateral earth pressure loading, similar in magnitude and distribution to that of a fluid having a density of 8.6 kN/m^3. In this case, lateral earth pressure due to floor loadings and/or foundation loads from adjacent buildings can be calculated assuming a lateral coefficient of 0.45. Equipment larger than a bobcat should not be allowed within 1.5 m of the foundation walls during backfilling. It is recommended that foundation walls be backfilled with clean, well graded granular material, compacted in maximum 300 mm lifts to 95% of SPMDD.

Lateral earth pressures resulting from seismic activity can be calculated according to the following equations:

Non-Rigid Wall : $P_E = 0.375\ k_h\ \gamma\ H^2$

Rigid Wall : $P_E = 0.5\ k_h\ \gamma\ H^2$

where:

- P_E is the resultant force per unit length of wall;
- the coefficients of 0.375 and 0.5 are dimensionless;
- k_h is the design peak horizontal ground acceleration coefficient (from above);
- γ is the moist unit weight of the backfill material, which is approximately 20.4 kN/m^3 for most granular backfill;
- H is the height of the wall.

In the case of the non-rigid wall, the backfill pressure distribution resulting from the earthquake loading can be assumed to be triangular, increasing from zero at the base of the wall to a maximum of $0.75\ k_h\ \gamma\ H$ at the top of the wall, with the resultant force acting at 0.67H above the base of the wall.

In the case of the rigid wall, the backfill pressure distribution resulting from the earthquake loading can be assumed to be parabolic, with the resultant force acting at 0.5H above the base of the wall.

For design purposes, the pressure distribution resulting from earthquake loading on the backfill should be added to either the active or at rest pressure distribution depending on whether or not the noted wall is allowed to rotate or not.

Pavement Considerations

Details of the pavement structure will depend on the anticipated loading conditions and can be provided once that information is available. Special considerations should be given to any areas of heavy truck loading, particularly for garbage enclosures and loading docks, where concrete slabs are often preferred to typical asphalt surfacing. The use of geogrid reinforcement could also be considered as a method to improve subbase performance and reducing the amount of sub excavation otherwise required. Geogrid recommendations should be made by a qualified geotechnical professional once traffic loads have been established.

CLOSURE

In summary, we consider the development as discussed to be feasible from a geotechnical perspective but several challenges regarding the proposed near zero lot line construction will need to be overcome. Careful consideration of excavation and encroachment will be necessary. We recommend completing an additional bedrock probing program to confirm the depth to bedrock throughout the site to support foundation and shoring designs. Further review of building elevations and loads will be necessary to more fully estimate the resulting settlement and thereby provide more detailed recommendations.

We trust the preceding is suitable for your purposes at present. If you have any questions with respect to the above, or require anything further, please contact us.

Sincerely,
Ryzuk Geotechnical

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
R. T. MOSER
45062
26/10/18

Richard Moser, P.Eng.
Project Engineer

26/10/18

Shane Moore, P.Geo.
Senior Geoscientist / Managing Partner

Attachments
- Terms of Engagement
- Location Plan
- Test Hole Logs

TERMS OF ENGAGEMENT

GENERAL

Ryzuk Geotechnical (the Consultant) shall render the Services, as specified in the agreed Scope of Services, to the Client for this Project in accordance with the following terms of engagement. The Services, and any other associated documents, records or data, shall be carried out and/or prepared in accordance with generally accepted engineering practices in the location where the Services were performed. No other warranty, expressed or implied is made. The Consultant may, at its discretion and at any stage, engage sub-consultants to perform all or any part of the Services.

Ryzuk Geotechnical is a wholly owned subsidiary of C. N. Ryzuk & Associates Ltd.

COMPENSATION

All charges will be payable in Canadian Dollars. Invoices will be due and payable by the Client on receipt of the invoice without hold back. Interest on overdue accounts is 24% per annum.

REPRESENTATIVES

Each party shall designate a representative who is authorized to act on behalf of that party and receive notices under this Agreement.

TERMINATION

Either party may terminate this engagement without cause upon thirty (30) days' notice in writing. On termination by either party under this paragraph, the Client shall forthwith pay to the Consultant its Charges for the Services performed, including all expenses and other charges incurred by the Consultant for this Project.

If either party breaches this engagement, the non-defaulting party may terminate this engagement after giving seven (7) days' notice to remedy the breach. On termination by the Consultant under this paragraph, the Client shall forthwith pay to the Consultant its Charges for the Services performed to the date of termination, including all fees and charges for this Project.

ENVIRONMENTAL

The Consultant's field investigation, laboratory testing and engineering recommendations will not address or evaluate pollution of soil or pollution of groundwater. The Consultant will cooperate with the Client's environmental consultant during the field work phase of the investigation.

PROFESSIONAL RESPONSIBILITY

In performing the Services, the Consultant will provide and exercise the standard of care, skill and diligence required by customarily accepted professional practices and procedures normally provided in the performance of the Services contemplated in this engagement at the time when and the location in which the Services were performed.

INSURANCE

Ryzuk Geotechnical is covered by Professional Indemnity Insurance as follows:

1. $ 2,000,000 each and every claim
2. $ 4,000,000 aggregate
3. $ 5,000,000 commercial/general liability coverage

LIMITATION OF LIABILITY

The Consultant shall not be responsible for:

1. the failure of a contractor, retained by the Client, to perform the work required for the Project in accordance with the applicable contract documents;
2. the design of or defects in equipment supplied or provided by the Client for incorporation into the Project;
3. any cross-contamination resulting from subsurface investigations;
4. any Project decisions made by the Client if the decisions were made without the advice of the Consultant or contrary to or inconsistent with the Consultant's advice;
5. any consequential loss, injury or damages suffered by the Client, including but not limited to loss of use, earnings and business interruption;
6. the unauthorized distribution of any confidential document or report prepared by or on behalf of the consultant for the exclusive use of the Client
7. Subsurface structures and utilities

The Consultant will make all reasonable efforts prior to and during subsurface site investigations to minimize the risk of damaging any subsurface utilities/mains. If, in the unlikely event that damage is incurred where utilities were unmarked and/or undetected, the Consultant will not be held responsible for damages to the site or surrounding areas, utilities/mains or drilling equipment or the cost of any repairs.

The total amount of all claims the Client may have against the Consultant or any present or former partner, executive officer, director, stockholder or employee thereof under this engagement, including but not limited to claims for negligence, negligent misrepresentation and breach of contract, shall be strictly limited to the amount of any professional liability insurance the Consultant may have available for such claims.

No claim may be brought against the Consultant in contract or tort more than two (2) years after the date of discovery of such defect.

DOCUMENTS AND REPORTING

All of the documents prepared by the Consultant or on behalf of the Consultant in connection with the Project are instruments of service for the execution of the Project. The Consultant retains the property and copyright in these documents, whether the Project is executed or not. These documents may not be used on any other project without the prior written agreement of the Consultant.

The documents have been prepared specifically for the Project, and are applicable only in the case where there has been no physical alteration to, or deviation from any of the information provided to the Consultant by the Client or agents of the Client. The Client may, in light of such alterations or deviations, request that the Consultant review and revise these documents.

The identification and classification as to the extent, properties or type of soils or other materials at the Project site has been based upon investigation and interpretation consistent with the accepted standard of care in the engineering consulting practice in the location where the Services were performed. Due to the nature of geotechnical engineering, there is an inherent risk that some conditions will not be detected at the Project site, and that actual subsurface conditions may vary considerably from investigation points. The Client must be aware of, and accept this risk, as must any other party making use of any documents prepared by the Consultant regarding the Project.

Any conclusions and recommendations provided within any document prepared by the Consultant for the Client has been based on the investigative information undertaken by the Consultant, and any additional information provided to the Consultant by the Client or agents of the Client. The Consultant accepts no responsibility for any associated deficiency or inaccuracy as the result of a miss-statement or receipt of fraudulent information.

JOBSITE SAFETY AND CONTROL

The Client acknowledges that control of the jobsite lies solely with the Client, his agents or contractors. The presence of the Consultant's personnel on the site does not relieve the Client, his agents or contractors from their responsibilities for site safety. Accordingly, the Client must endeavor to inform the Consultant of all hazardous or otherwise dangerous conditions at the Project site of which the Client is aware.

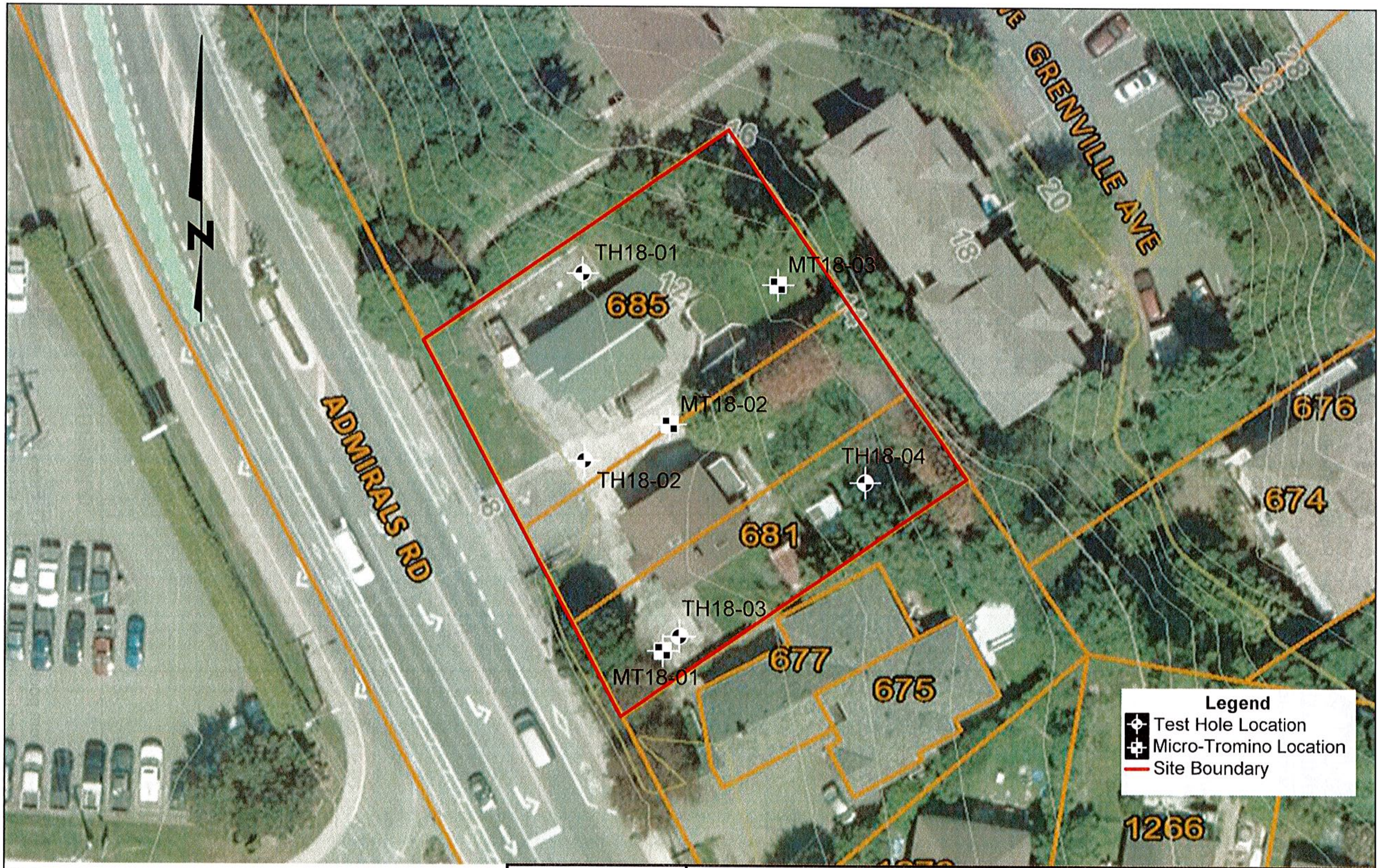
The client must acknowledge that during the course of a geotechnical investigation, it is possible that a previously unknown hazard may be discovered. In this event, the Client recognizes that such a hazard may result in the necessity to undertake procedures which ensure the safety and protection of personnel and/or the environment. The Client shall be responsible for payment of any additional expenses incurred as a result of such discoveries, and recognizes that under certain circumstances, discovery of hazardous conditions or elements requires that regulatory agencies must be informed. The Client shall not bring about any action or dispute against the Consultant as a result of such notification.

FIELD SERVICES

Where applicable, field services recommended for the Project are the minimum necessary, in the sole discretion of the Consultant, to observe whether the work or a contractor retained by the Client is being carried out in general conformity with the intent of the Services. Any reduction from the level of services recommended will result in the Consultant providing qualified certifications for the work.

DISPUTE RESOLUTION

If requested in writing by either the Client or the Consultant, the Client and the Consultant shall attempt to resolve any dispute between them arising out of or in connection with this Agreement by entering into structured non-binding negotiations with the assistance of a mediator on a without prejudice basis. The mediator shall be appointed by agreement of the parties. If a dispute cannot be settled within a period of thirty (30) calendar days with the mediator, the dispute shall be referred to and finally resolved by arbitration under the rules of the arbitrator appointed by agreement of the parties or by reference to a Judge of the British Columbia Court.



Notes:
1. Base plan taken from CRD Natural Areas Atlas using Air Photo 2017 overlay.

WestUrban Developments Ltd.		DRAWN SDJ
LOCATION PLAN Proposed Multi-Storey Residential Development		DATE October, 2018
		APPROVED
681 & 685 Admirals Road	Esquimalt, B.C.	SCALE 1:500
RYZUK GEOTECHNICAL	Engineering & Materials Testing	DRAWING No. 8-8298-2-1

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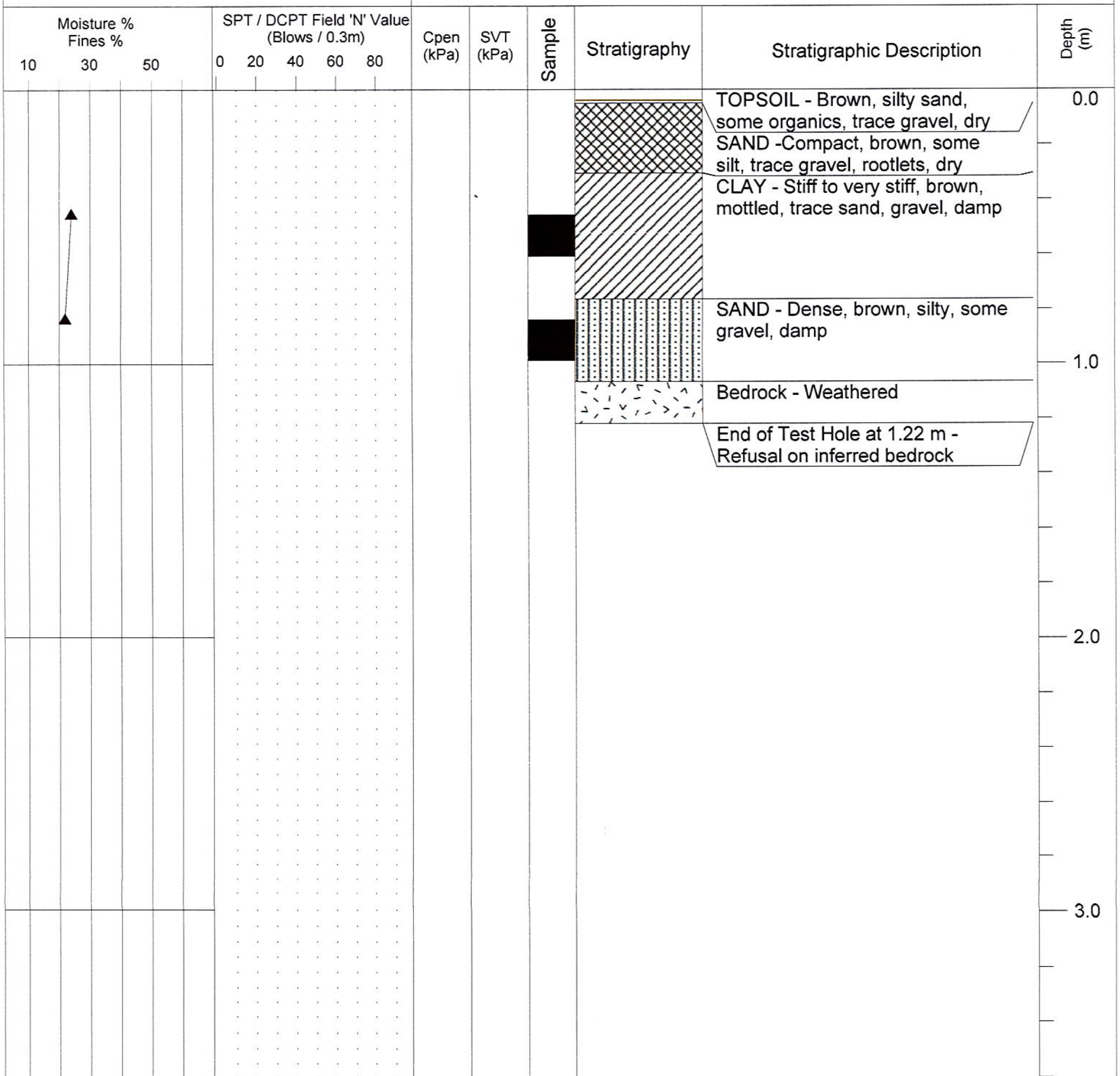
TEST HOLE LOG

TH18-01

Project:	Proposed Multi-Storey Residential Building		
UTM:	469133 m E; 5364597 m N		
Client:	WestUrban Developments Ltd.	Job #:	8-8298-2
Location:	681-685 Admirals Road - Saanich, BC	Method:	Track Mounted Auger
	See Location Plan dwg. 8-8298-2-1	Driller:	Drillwell
Drill Date:	October 5, 2018	Inspector:	SDJ



LEGEND

COMMENTS:

■ SPT Sample	Cpen: Su from pocket penetrometer	○ Fines %	▲ Moisture Content
■ Grab Sample	SVT: Su from in-situ shear vane	▼ Groundwater table	

TEST HOLE LOG

TH18-02

RYZUK GEOTECHNICAL
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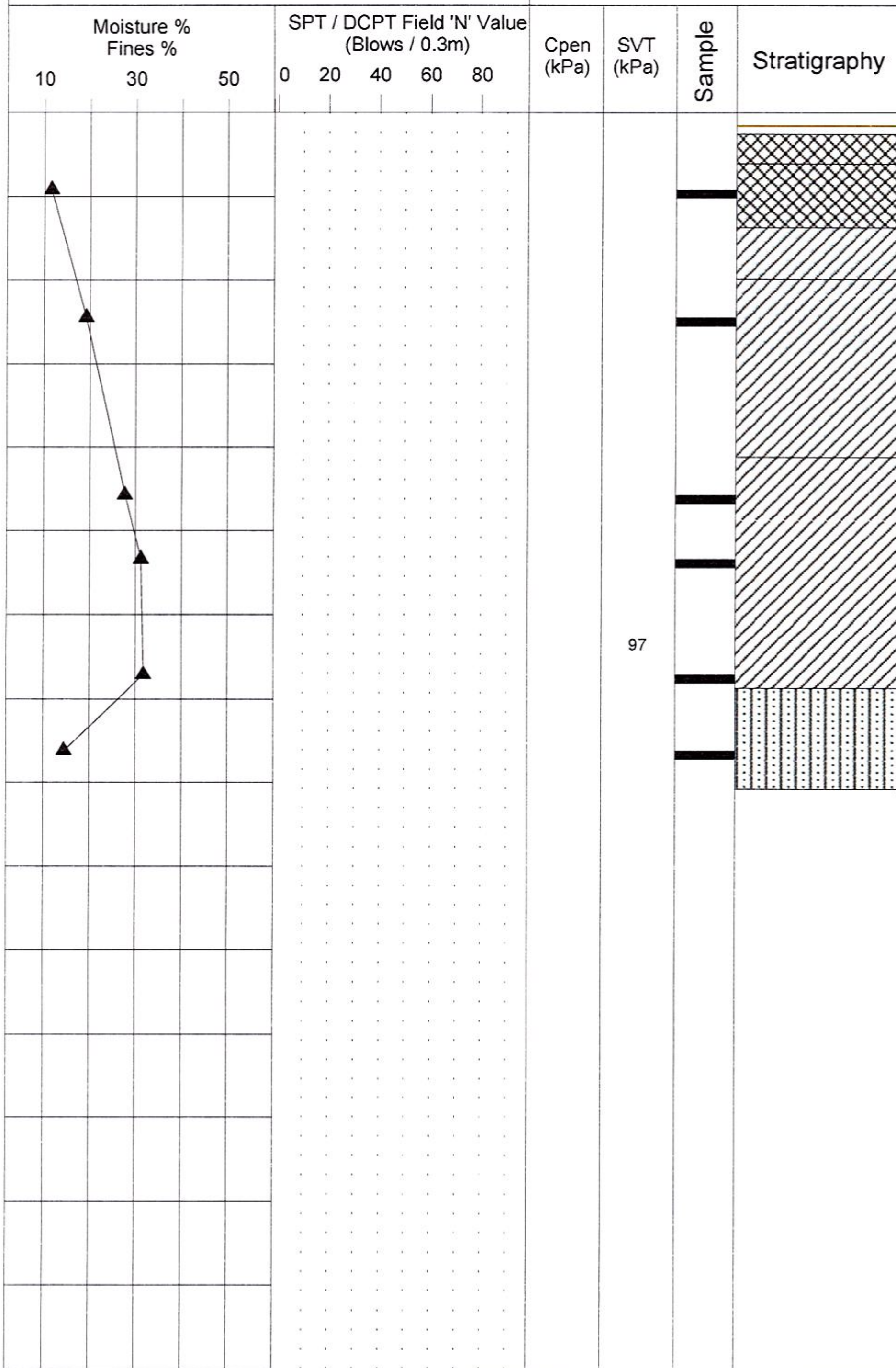
Project: Proposed Multi-Storey Residential Building
UTM: 469129 m E, 5364581 m N
Client: WestUrban Developments Ltd. Job #: 8-8298-2
Location: 681-685 Admirals Road - Saanich, BC Method: Track Mounted Auger
See Location Plan dwg. 8-8298-2-1 Driller: Drillwell Ltd
Drill Date: October 5, 2018 Inspector: SDJ



Stratigraphic Description	Depth (m)
TOPSOIL - Brown, silty, some organics, some sand, rootlets, dry	0.0
SILT (Fill) - Brown, some sand, rootlets, dry	1.0
SILT (Fill) - Black, organics, trace sand, trace gravel, dry	
CLAY - Stiff, brown, mottled, silty, some sand, damp	2.0
CLAY - Stiff to very stiff, brown, mottled, silty, trace sand, trace gravel, damp	3.0
CLAY - Stiff, grey, silty, trace gravel, damp	4.0
	5.0
- Moist at 6.10 m	6.0
SILT and SAND - Very stiff/dense, grey, trace gravel, moist	7.0
	8.0
End of Test Hole at 8.08 m - Refusal on inferred bedrock	
	9.0
	10.0
	11.0
	12.0
	13.0
	14.0
	15.0

LEGEND

COMMENTS:

- SPT Sample
- Grab Sample
- Cpen: Su from pocket penetrometer
- SVT: Su from in-situ shear vane
- ○ Fines %
- ▼ Groundwater table
- ▲ Moisture Content

RYZUK GEOTECHNICAL
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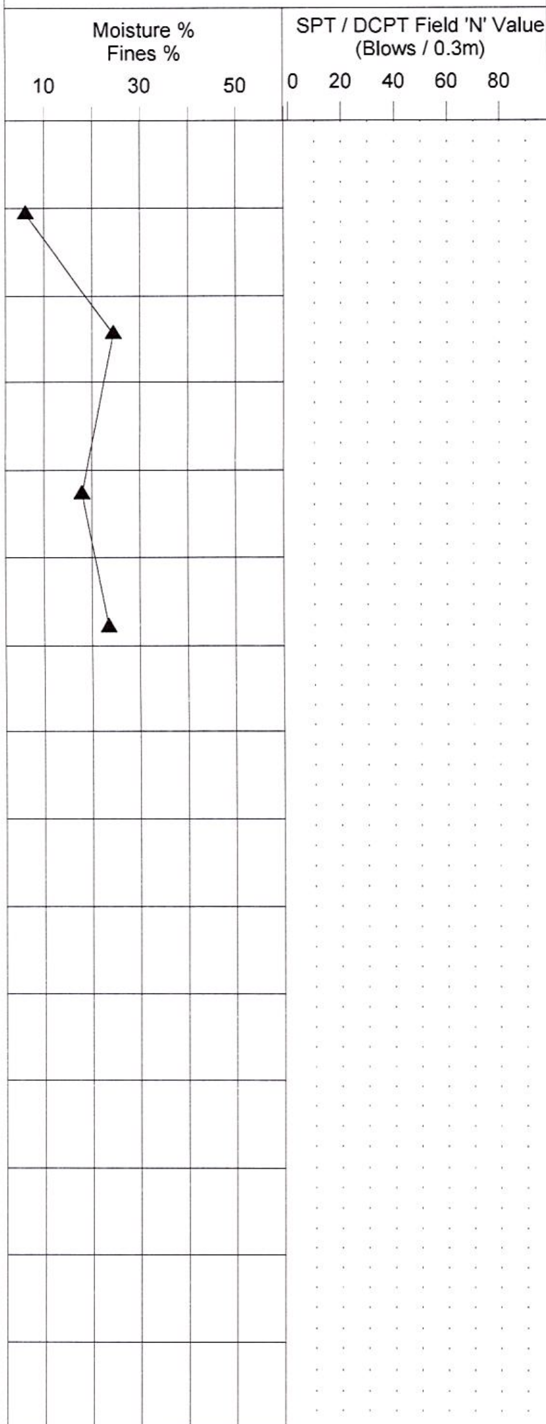
TEST HOLE LOG

TH18-03

Project:	Proposed Multi-Storey Residential Building		
UTM:	469139 m E, 5364560 m N		
Client:	WestUrban Developments Ltd.	Job #:	8-8298-2
Location:	681-685 Admirals Road - Saanich, BC	Method:	Track Mounted Auger
	See Location Plan dwg. 8-8298-2-1	Driller:	Drillwell Ltd
Drill Date:	October 5, 2018	Inspector:	SDJ



Cpen (kPa)	SVT (kPa)	Stratigraphic Description	Depth (m)
		FILL - Concrete	0.0
		SAND (Fill) - Compact, grey, sand, some angular cobbles, trace gravel, damp	1.0
		SILT (Fill) - Black, organics, trace sand, trace gravel, dry	2.0
		SAND - Compact to dense, brown, trace silt, dry - Silty	3.0
		CLAY - Stiff, brown, mottled, silty, sandy, damp - Trace sand	4.0
		CLAY - Stiff to very stiff, brown, silty, trace sand, damp	
		CLAY - Frim to stiff, grey, silty, damp - Moist at 5.64 m - Intermittent pockets of sand and gravel	5.0 – 6.0
	92		7.0
	44		8.0
			9.0 – 13.0
		Inferred Glacial Till - Dense to very dense	14.0
		End of Test Hole at 14.4 m - Refusal on inferred bedrock	15.0

LEGEND

COMMENTS:

■ SPT Sample
■ Grab Sample

Cpen: Su from pocket penetrometer
SVT: Su from in-situ shear vane

○ Fines %
▼ Groundwater table
▲ Moisture Content

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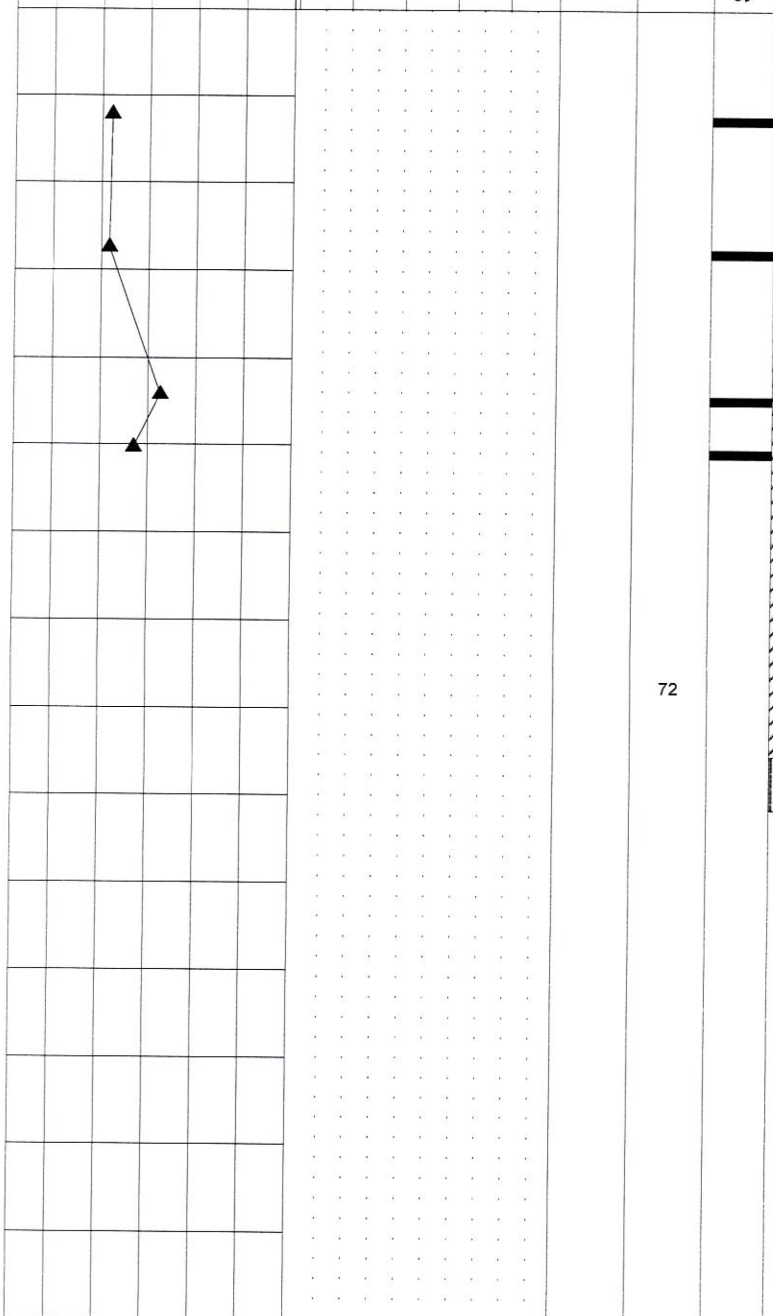
TEST HOLE LOG

TH18-04

Project: Proposed Multi-Storey Residential Building
UTM: 469157 m E, 5364577 m N
Client: WestUrban Developments Ltd.
Job #: 8-8298-2
Location: 681-685 Admirals Road - Saanich, BC
See Location Plan dwg. 8-8298-2-1
Method: Track Mounted Auger
Driller: Drillwell Ltd
Drill Date: October 5, 2018
Inspector: SDJ

Moisture % / Fines % (10, 30, 50)	SPT / DCPT Field 'N' Value (Blows / 0.3m) (0, 20, 40, 60, 80)	Cpen (kPa)	SVT (kPa)	Sample	Stratigraphy	Stratigraphic Description	Depth (m)



- TOPSOIL - Black, organic, silty, trace sand, rootlets, dry (0.0)
- SAND - Compact to dense, brown, trace to some silt, dry (1.0)
- CLAY - Stiff, brown, mottled, silty, some sand, damp
- CLAY - Stiff to very stiff, brown, silty, trace sand, gravel damp (2.0)
- CLAY - Firm to stiff, grey, silty damp (4.0)
  - Moist at 5.03 m
  - Intermittent pockets of sand and gravel
- SVT at ~7.6 m: 72
- Inferred Glacial Till - Dense to very dense (8.5)
- End of hole at 9.14 m - Refusal on inferred bedrock

Depth (m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0

LEGEND

COMMENTS:

- SPT Sample
- Grab Sample
- Cpen: Su from pocket penetrometer
- SVT: Su from in-situ shear vane
- ○ Fines %
- ▼ Groundwater table
- ▲ Moisture Content