Attachment 11

GEOPACIFIC
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PCI Developments Corporation
#300 – 1030 West Georgia Street
Vancouver, BC
V6E 2Y3

May 19, 2023
File: 16004
R0

Attention: Brad Howard

Re: Geotechnical Investigation Report – Proposed Mixed-Use Development
3020 Spring Street, Port Moody

1.0 INTRODUCTION

We understand that a new development is proposed for the site referenced above. Architectural drawings for the development prepared by Perkins and Will, dated May 10, 2023, indicate the development will consist of two towers 40 and 37 stories tall over 2 to 3 stories of shared mixed-use podium all over 3 to 4 levels of below grade parking. We expect reinforced concrete construction throughout and very heavy structural loading.

This report presents the results of our geotechnical investigation and provides recommendations for the design and construction of the proposed development. The report has been prepared exclusively for PCI Developments Corporation, for their use and the use of others on their design and construction team. We also expect this report will be relied upon by the City of Port Moody during their permit process. No other use of this report is permitted without written consent of GeoPacific.

2.0 SITE DESCRIPTION

The proposed Spring Street site is an assemblage of lots including 3006, 3010, and 3020 Spring Street. The site is bounded by the Evergreen Line Rapid Transit right-of-way to the north, private property to the east and west, and Spring Street to the south. The site has a frontage of approximately 100 m along Spring Street. Furthermore, the site is relatively flat with elevations of approximately 9 to 11 m geodetic according to the City of Port Moody GIS and provided architectural drawings. The site is currently improved with industrial warehouse buildings, surrounded by asphalt paved parking and drive aisles.

The location of the site relative to surrounding properties and roads is shown on our site plan, Drawing No. 16004-02, following the text of this report.

3.0 FIELD INVESTIGATION

GeoPacific Consultants Ltd. completed a geotechnical site investigation on April 4th, 2018, using the subcontracted services of On Track Drilling of Langley, BC. A total of four test holes were completed to depths of up to 15.2 m below existing grades. To provide subsurface profiling, test holes were supplemented with electronic cone penetration test (CPT) soundings to depths of up to 12.0 m, and one seismic cone penetration test (SCPT) sounding. to determine relative density/consistency of subsurface soils, a Dynamic Cone Penetration Test (DCPT) sounding was completed to refusal to 15.2 m at TH18-04.

The test hole and CPT logs are presented in Appendices A and B, respectively. Geotechnical parameters calculated from the CPT soundings, such as undrained shear strength and standard penetration N1(60) values, are presented in Appendix C of this report. Liquefaction analysis and shear wave velocity profile of the subsurface soils are presented in Appendices D and E, respectively.

The CPT is an in-situ testing device which is pushed into the ground by employing a hydraulic ram on the drill rig. The cone penetrometer records measurements of tip resistance, sleeve resistance, dynamic pore water pressure, temperature, and inclination in 5 centimeter increments. The data obtained may be correlated to engineering parameters such as shear strength, relative density, soil behavior type, and consolidation coefficients. The SCPT is an in-situ test, used in conjunction with the CPT, to obtain shear wave velocity profiles which assist in the seismic design of building foundations. The investigation was organized and supervised by a member of our technical staff who logged the soils and collected samples for laboratory testing. All test holes were backfilled in accordance with provincial requirements immediately upon completion. The approximate locations of the test holes and CPT soundings are shown on Drawings No. 15322-E-01.

4.0 SUBSURFACE CONDITIONS

4.1 Soil Conditions

In general, the soil profile noted from the surface downwards at our test holes consists of FILL, overlying SAND, then SAND AND GRAVEL with variable silt content, overlying CLAYEY SILT, overlying a basal layer of Till-Like Silty SAND. A general description of the soils encountered is as follows.

SAND AND GRAVEL (FILL)

The fill consists of loose to compact fine to medium grained SAND to silty SAND and GRAVEL with trace cobbles. The fill was brown to grey, moist, and loose to compact. The fill ranged in depth from 1.2m to 1.5m below current site grades.

SAND

The fill is typically underlain by compact to dense SAND with trace to some silt, trace gravel, and cobbles. The sand is fine to medium grained, brown to grey and moist to wet. The sand extends to depths of 4.6 to 5.5 m below existing grades.

SAND AND GRAVEL (COLLUVIUM)

The sand is underlain by loose to compact SAND and GRAVEL colluvial deposit with variable silt and cobbles. The sand and gravel is fine to medium grained with trace coarse grained sand layers. The gravel clast ranges from 5 to 20 mm and were observed to be subrounded. The sand and gravel is grey to light grey and wet. The deposit extends to approximately 8.5 to 10.1 m below existing grades.

CLAYEY SILT

The colluvial deposit is underlain by soft to firm CLAYEY SILT with trace sand at all test hole location with the exception of TH18-02 where the colluvium layer appeared to be directly underlain by sill-like silty sand. The clayey silt is grey and moist to wet with water content ranging from 15.4 to 28.1% based on laboratory testing completed. The clayey silt extends to depths of 10.7 to 13.1 m below existing grades. Based on our CPT interpreted indices, the clayey silt has an undrained shear strength of approximately 30 to 50 kPa in the upper portion, increasing to 75 to 100 kPa with depth.

Silty SAND (Glacial Till)

The silt and/or colluvium is underlain by a basal layer of dense to very dense glacial till substratum composed of silty sand with trace to some gravel and occasional cobbles. The top of this layer is inferred to be between 8.5 and 13.1 m depth. Our experience in the area indicates that boulders are commonly present within the glacial deposits present on-site. Additional sonic drilling and density/consistency testing is recommended to confirm the depth to glacial till across the site.

For a more detailed description of the subsurface soil conditions, refer to the appendices.

4.2 Groundwater Conditions

The static groundwater table was estimated at the time of our investigation to be at a depth of 1.8 to 2.1m below current site grades based on observations at our test hole locations. Occasional perched water was noted in the surficial fill material and should be expected to occur during wetter periods. Groundwater levels are expected to vary seasonally with generally higher level following sustained precipitation, and to a lesser extent by tidal fluctuations.

For a more detailed description of the subsurface soil conditions refer to the Test Hole Logs and CPT Sounding Logs in Appendices A and B.

5.0 DISCUSSION

As noted in Section 1.0, architectural drawings for the development prepared by Perkins and Will, dated May 10, 2023, indicates the development will consist of two towers 40 to 37 stories tall over 2 to 3 stories of shared mixed-use podium all and 3 to 4 levels of below grade parking. We expect reinforced concrete construction throughout and very heavy structural loading. We assume that finished exterior grades will be maintained at or near existing grades within the building areas.

Based on the provided architectural drawings the proposed below grade parking will extend to or near the property lines on all sides except for the north side which is set back approximately 15 to 19 m away from the property line and the existing evergreen line. As such, we expect anchored vertical shoring will likely be required to facilitate the proposed excavation. Due to the variability in permeability of underlying soils and the observed depth to groundwater, a groundwater cut off wall will likely be required to control groundwater both in the temporary excavation and shoring phases and for the permanent condition to facilitate normal drained cavity construction.

The majority of the development appears to include 4 levels of below grade construction with a small portion along the north edge of the parkade in the middle of site indicated to be only three levels of below grade based on the latest architectural drawings. We anticipate the excavation depth for the underground parking will range from about 10 to 14 meters below existing grades. The subsurface conditions at underside of foundation are expected to consist of dense till-like soils for 4 levels of below grade but will be relatively variable for 3 levels of below grade and consist of sand and gravel colluvium to soft to firm clayey silt. Due to the expected very heavy loading of the proposed towers, where encountered the sand and gravel colluvium and/or soft to firm clayey silt will need to be over-excavated to expose till-like soils and replaced with lean-mix concrete or alternatively, piled foundations will be required to transfer structure loads to the glacial till below. Where the structure is supported directly on glacial till or where the unsuitable soils are over-excavated and replaced with lean-mix, we expect the proposed development can be adequately supported on conventional foundations. Additional sonic drilling and density/consistency testing is recommended to confirm the depth to glacial till across the site and particularly where three levels of parkade are proposed.

We confirm, from a geotechnical point of view, that the proposed development is feasible provided the recommendations outlined in this report are incorporated into the overall design.

6.0 RECOMMENDATIONS

6.1 Site Preparation

Existing structures, pavements, underground services, all organic materials, vegetation, topsoil, fills, and loose or otherwise disturbed soils must be removed from the construction area. Depending on the depth to foundation we expect the subgrade will consist of either sand and gravel colluvium, soft to firm clayey silt, or dense glacial till. Where encountered, the sand and gravel colluvium and/or soft to firm clayey silt will need to be over-excavated to expose glacial till and replaced with lean mix concrete or alternatively, short piled foundations will be required to transfer structural loads to the glacial till below. Lean-mix concrete must have a minimum unconfined compressive strength (UCS) of 5MPa.

Due to the high bearing pressures recommended in Section 6.3.2 below, subgrades for convention foundations must be blinded with lean-mix concrete with minimum 5 MPa UCS, blinding with conventional clear crush gravel below footings is not acceptable.

The loose to compact sand and gravel or firm to stiff clayey silt layers are expected to susceptible to disturbance at the excavated surface due to groundwater seepage, precipitation, construction activity, and vehicular traffic. We expect for these conditions all slab subgrades should be blinded with a minimum thickness of 100 mm clear crushed gravel, increasing to 600 mm in areas where vehicular traffic must traverse the site. Subgrades should be graded to inhibit ponding of water. Any water softened/disturbed soils must be excavated to expose undisturbed subgrade.

In the event over-excavation is required due to poor quality soils near the excavated surface, reinstatement of subgrade below slabs should be completed with compacted "engineered fill". In the context of this report, engineered fill is defined as clean sand or sand and gravel, compacted in 300 mm loose lifts to a minimum of 98% Standard Proctor dry density (ASTM D698), at a moisture content that is within 2% of its optimum for compaction.

The subgrade must be reviewed prior to the placement of any engineered fill or clear crushed gravel.

6.2 Groundwater Cut Off

A perimeter cut off wall is recommended to minimize the need for temporary site de-watering (the cut off wall would need to be extended into the undisturbed glacial till layer to be effective). The cut off wall can be constructed using a secant pile system. Techniques such as sheet pile walls as well as Geo-Jet are not recommended due to the quality of the underlying soils which contain considerable cobbles and gravel in the upper stratum.

Due to recent changes with City of Port Moody encroachment policies, no portion of the perimeter cut-off wall is permitted to extend over the property line; therefore, we expect the building should be setback from property lines a minimum of 0.75 m to permit the installation of a secant pile groundwater cut off system.

6.3 Building Foundations

6.3.1 Conventional Foundations

We expect conventional pad and strip foundations may be used to support the proposed structure where glacial till is present at/near the foundation elevation. We recommend that conventional foundations are designed using a Serviceability Limit State (SLS) bearing pressure of 500 kPa based on support on dense to very dense glacial till. Factored Ultimate Limit State (ULS) bearing pressures can be taken as 1.5 × SLS bearing pressure provided.

We expect soils can be over-excavated up to 3 m below conventional foundations to expose dense glacial till below, then replaced with lean mix concrete, with a minimum UCS of 5 MPa, extending from underside of conventional foundation to the glacial till interface. The lean mix concrete is intended to transfer the conventional foundation loads directly to the dense glacial till below.

Irrespective of allowable bearing pressures, footings should not be less than 600 mm by 600 mm and strip footings should not be less than 450 mm in width. Footings should also be buried a minimum of 450 mm below the surface for frost protection. We estimate for foundations designed as per recommended, settlements will not exceed 25 mm total and 20 mm in 10 metres differential.

Adjacent conventional pad and strip foundations constructed at differing elevations should be offset from each other by a minimum distance of twice the difference in elevation, 2H:1V. Similarly, excavations adjacent to footings should be completed outside a 2H:1V slope from outside edge of bottom of footings, including excavations for utility trenches.

GeoPacific must be contacted to review all footing subgrade prior to blinding and footing construction.

6.3.2 Pile Foundations

Given the expected magnitude of superstructure loading for mid to high rise developments, we expect pile supported foundations will be required to transfer the foundation loading to the underlying dense to very dense soils where glacial till is deeper than about 3 to 4 m below the proposed underside of footing (potentially where 3 levels of below grade construction are proposed). A number of pile types can be considered; however, based on our experience in the area, drilled concrete piles or closed ended steel pipe piles should be considered as suitable options.

Driven steel pipe piles can be expected to develop their full structural capacity when driven to refusal in the dense to very dense soil which underlies the site at inferred depths of 9 to 14 m below existing site grades.

We expect up to 4.0 m of penetration into the dense to very dense soils for a drilled concrete pile or fully utilized closed ended steel pipe pile. Nominal 16-inch and 24-inch diameter steel pipe piles can support working loads of between 1,400 kN and 2,200 kN depending on yield strength and wall thickness, similar loading capacity is expected for drilled concrete piles depending on pile size. We do not expect a noticeable group effect, provided that the centre-to-centre pile spacing is maintained at a minimum of 3×D, where D is the pile diameter (a reduction in capacity is required for piles located within 3D of one another). GeoPacific can provide further details and refinement of pile design in the detailed design stage as required.

We recommend a test pile program to be implemented to evaluate the capacity of the piles and ascertain pile dimensions for the site. Pile Dynamic Analysis (PDA) tests will be required to confirm in-situ pile capacities for driven piles.

Pile foundations require full time supervision during construction, as per British Columbia Building Code, BCBC 2018.

6.4 Slab-on-grade Floors

In order to provide suitable support for slab-on-grade floors we recommend that any fill placed under the slab should be granular and essentially “clean” with not more than 5% passing the #200 sieve.

In addition, this granular fill must be compacted to a minimum of 95% Modified Proctor (ASTM D1557) maximum dry density with water content within 2% of optimum for compaction.

Floor slabs should be directly underlain by a minimum of 150 mm of 19 mm clear crushed gravel, hydraulically connected to perimeter drainage. A moisture barrier should underlie the slab directly above the free draining granular material.

Slab-on-grade fill compaction must be reviewed by GeoPacific.

6.5 Site and Foundation Drainage Systems

We recommend that a sub-drainage system be included in the mechanical design for the proposed building to prevent the development of water pressures on the foundation walls and the basement slabs. Provided a groundwater cut off approach is used in the design we expect typical perimeter drainage should be feasible to drain parkade walls and slab on grade. Flows should be light to moderate, at less than 50 litres/minute per site. Flows should be confirmed at the time of construction.

6.6 Seismic Design of Foundations

For structures to be constructed at the above referenced site, the Site Classification, as defined in Section 4.1.8.4 of the 2018 BCBC, should be assumed to be “Site Class C” in accordance with Table 4.1.8.4., based on conventional foundation supported directly on dense to very dense glacial till. Re-evaluation of the seismic site classification and seismic hazard will be required if the building is subject to the upcoming BCBC 2023.

Based on our CPT results, the native soils underlying the proposed site may be prone to liquefaction or ground softening at the design earthquake event; however, based on the contemplated excavation depths we expect the proposed foundations will be constructed below the liquefaction susceptible layers.

6.7 Temporary Excavations

Due to depth of excavation, proximity of public and private structures, soil conditions, and presence of a high static groundwater table, a vertical secant pile wall should be used to permit vertical excavation and act as a cut-off wall. The secant pile wall will be internally reinforced and tied back with ground anchors to resist the earth pressure and water pressure forces generated on the shoring wall.

Hollow core anchors should be anticipated, and special seals and/or water stopping materials will be required at anchor locations to ensure retained water and sediment do not migrate into the site. Sumps should be located in the excavation to assist in collection and removal of groundwater.

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All excavations and trenches must conform to the latest Occupational Health and Safety Regulation supplied by Work Safe BC. Any excavation in excess of 1.2 metres in depth requiring worker entry must be reviewed by a professional geotechnical engineer. Temporary excavations in the fill soils and native soils can be cut at a slope angle of 1H:1V. All slopes should be covered with poly sheeting.

GeoPacific may provide a shoring and excavation design upon request.

6.8 Earth Pressures on Foundation Walls

Earth pressures against the foundation walls are dependent on factors such as, available lateral restraint along the wall, surcharge loads, backfill materials, compaction of the backfill (if applicable) and drainage conditions. We assume that the backfill between foundation walls and shoring would be a free-draining granular material such as birds eye gravel. The foundation wall is expected to be partially yielding and fully restrained between the parking floor slabs. We recommend that the foundation walls to be designed to resist the following lateral pressures:

Static:	Triangular soil pressure distribution of 6.0H kPa (where H is equal to the total wall height below grade). Below 2 metres depth, the pressure should increase to 13D (kPa) triangular soil and water pressure, where D is the total wall height in metres below 2 metres. Depending on the stiffness of the perimeter cut-off wall utilized, static lateral pressure may be further distributed to the permanent structure suspended slabs which would reduce lateral pressure on structural foundation wall.
Seismic:	Inverted triangular seismic surcharge of 3.0H kPa (where H is equal to the total backfill height in metres). The seismic lateral loading may change depending on the code in effect at the time of detailed design.

Any additional surcharge loads located near the foundation walls should be added to the earth pressures given and the design of the wall shall be revised, accordingly. A hydrogeological investigation and analysis report may be required to determine long term groundwater elevations during the wetter winter months and permeability of underlying soils.

7.0 DESIGN REVIEWS AND CONSTRUCTION INSPECTIONS

The preceding sections make recommendations for the design and construction of the proposed residential development. We have recommended the review of certain aspects of the design and construction in this report. In summary, geotechnical reviews are required for the aspects of work listed on the following page.

1. Stripping/Excavation	Review of stripping, temporary cut slopes, and soil conditions
2. Shoring	Review of shoring and cut off wall installation
3. Anchoring	Review of anchor installation, testing, and decommissioning
4. General Compaction	Review of compaction of engineered fill and clear crushed gravel
5. Foundation	Review of foundation subgrades
7. Pile Foundations	Review of pile driving and testing, as required
8. Slab on-grade	Review of subgrade, under-slab fill materials, and compaction.

It is important that these reviews are carried out to ensure that our intentions have been adequately communicated. It is also important that the contractors working on the site review this document prior to commencing their work and notify GeoPacific at least 48 hrs in advance of the required field reviews.

Attachment 11

8.0 CLOSURE

This report has been prepared exclusively for our client, for the purpose of providing geotechnical recommendations for the design and construction of the development described herein. The report remains the property of GeoPacific Consultants Ltd. and unauthorized use of, or duplication of, this report is prohibited.

We are pleased to be of assistance to you on this project and we trust that our recommendations are both helpful and sufficient for your current purposes. If you would like further details or would like clarification of any of the above, please do not hesitate to call.

For:
GeoPacific Consultants Ltd.

Reviewed by:

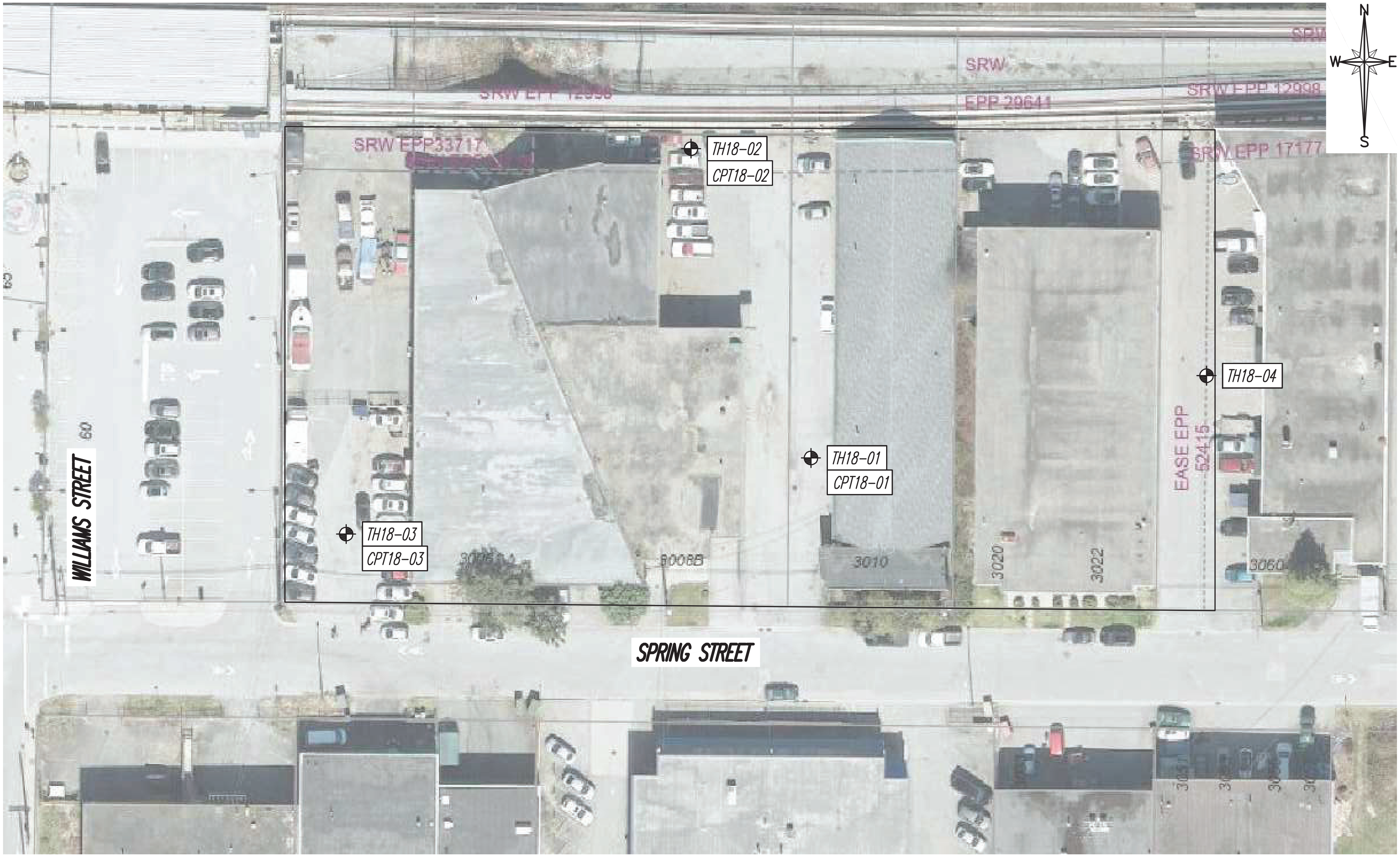
2023-05-25
PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
K. J. L. BODNAR
37915
Permit to Practice 1000782

Wyatt Johnson, B.Eng., P.Eng.
Project Engineer

Kevin Bodnar, M.Eng., P.Eng.,
Principal

Liam Jones B.Eng., EIT
Geotechnical Engineer-In-Training

Attachment 11



SITE PLAN

LEGEND:

⌖ TH21-XX – TEST HOLE (TH) LOCATION
▼ CPT22-XX – CONE PENETRATION TEST (CPT) LOCATION
▼ SCPT22-XX – SEISMIC CONE PENETRATION TEST (SCPT) LOCATION

*TEST LOCATIONS ARE APPROXIMATE

REFERENCE:

GEOPACIFIC
CONSULTANTS

DATE: 2023-05-19
DRAWN BY: LJ
APPROVED BY:
REVIEWED BY:
SCALE: NTS

PROPOSED MIXED-USE DEVELOPMENT
3020 SPRING STREET
TEST HOLE LOCATION PLAN

FILE NO.: 16004
DWG. NO.: 16004-01

REVISIONS:
A.
B.
C.

ORIGINAL PAPER SIZE 8.5"x11"

Attachment 11

APPENDIX A - TEST HOLE LOGS

Test Hole Log: TH18-01 (CPT18-01)

File: 16004

Project: MIXED USE DEVELOPMENT

Client: PCI DEVELOPMENTS

Site Location: 3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY, BC

GEOPACIFIC
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1779 West 75th Avenue, Vancouver, BC, V6P 6P2
Tel: 604-439-0922 Fax:604-439-9189

INFERRED PROFILE

SOIL DESCRIPTION	Depth (m)/Elev (m)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
Ground Surface	0.0				
Asphalt [76.2mm]					
Sand and Gravel [Fill] loose to compact silty SAND, trace to some gravel fill, fine to medium grained sand, 10-20mm subrounded gravel clast, grey, slightly moist to moist	1.2	16.0			
Sand compact SAND, trace to some silt, trace gravel, trace cobbles, tan to grey, moist to wet	4.0	35.5		▼	1.8m estimated water table depth
Sand and Gravel compact to dense, silty SAND, trace to some GRAVEL, trace cobbles, till-like, grey, moist	9.8	10.4 11.3			
Silt soft to firm SILT, trace sand, trace sand lenses at depth, grey, moist to wet	12.2	28.1			
Sand and Gravel [Till] dense to very dense silty SAND, trace to some GRAVEL till, fine to medium grained sand, grey, moist	13.7	21.9			
End of Borehole					

Logged: SH
Method: Solid stem auger
Date: 2018-Apr-04

Datum: Ground elevation
Figure Number: A.01
Page: 1 of 1

Test Hole Log: TH18-02 (CPT18-02)

File: 16004

Project: MIXED USE DEVELOPMENT

Client: PCI DEVELOPMENTS

Site Location: 3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY, BC

GEOPACIFIC CONSULTANTS
1779 West 75th Avenue, Vancouver, BC, V6P 6P2
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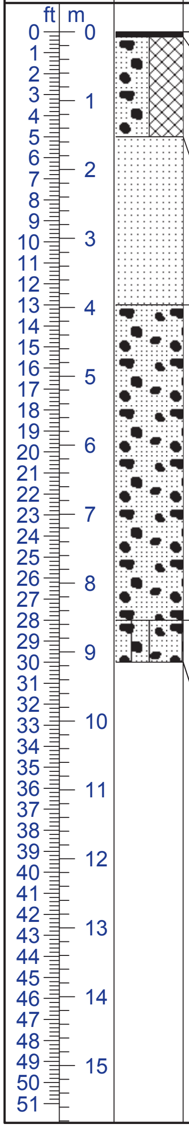
INFERRED PROFILE



SOIL DESCRIPTION	Depth (m)/Elev (m)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
Ground Surface	0.0				
Asphalt [76.2mm]					
Sand and Gravel [Fill] loose to compact silty SAND, trace to some gravel fill, fine to medium grained sand, 10-20mm subrounded gravel clast, brown to grey, slightly moist to moist	1.5	12.9			
Sand compact SAND, trace to some silt, trace gravel, trace cobbles, tan to grey, moist to wet	4.0	20.3		▼	1.8m estimated water table depth
Sand and Gravel compact to dense, silty SAND, trace to some GRAVEL, trace cobbles, till-like, grey, moist	8.5	12.7 30.4			
Sand and Gravel [Till] dense to very dense silty SAND, trace to some GRAVEL till, fine to medium grained sand, grey, moist	9.1	20.5			
End of Borehole					

Logged: SH
Method: Solid stem auger
Date: 2018-Apr-04

Datum: Ground elevation
Figure Number: A.02
Page: 1 of 1

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Test Hole Log: TH18-03 (CPT18-03)

File: 16004

Project: MIXED USE DEVELOPMENT

Client: PCI DEVELOPMENTS

Site Location: 3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY, BC

GEOPACIFIC
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INFERRED PROFILE							
Depth (ft / m)	Symbol	SOIL DESCRIPTION	Depth (m)/Elev (m)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
		Ground Surface					
		Asphalt [76.2mm]	0.0				
		Sand and Gravel [Fill] loose to compact silty SAND, trace to some gravel fill, fine to medium grained sand, 10-20mm subrounded gravel clast, grey, slightly moist to moist		25.1			
		Sand compact SAND, trace to some silt, trace gravel, trace cobbles, tan to grey, moist to wet	1.5	24.5		▼	1.8m estimated water table depth
		Sand and Gravel compact to dense, silty SAND, trace to some GRAVEL, trace cobbles, till-like, grey, moist	4.6	10.7			
		Silt soft to firm SILT, trace sand, trace sand, grey, moist to wet	10.1	15.4			
		Sand and Gravel [Till] dense to very dense silty SAND, trace to some GRAVEL till, fine to medium grained sand, grey, moist	10.7				
		End of Borehole	12.2				

Logged: SH
Method: Solid stem auger
Date: 2018-Apr-04

Datum: Ground elevation
Figure Number: A.03
Page: 1 of 1

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Test Hole Log: TH18-04

File: 16004

Project: MIXED USE DEVELOPMENT

Client: PCI DEVELOPMENTS

Site Location: 3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY, BC

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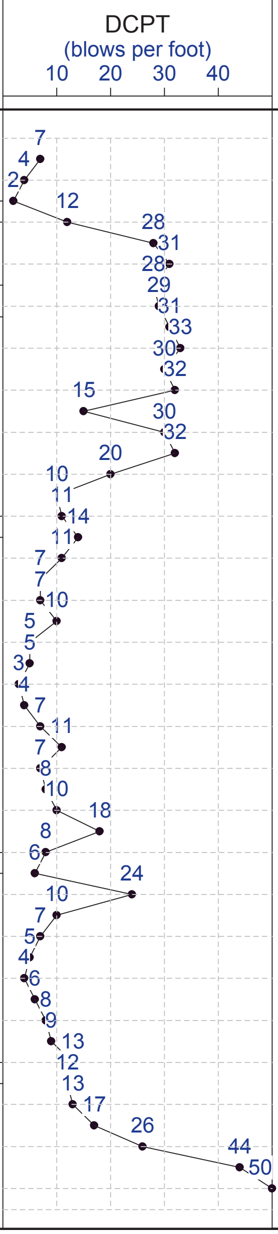
INFERRED PROFILE

SOIL DESCRIPTION	Depth (m)/Elev (m)	Moisture Content (%)	Groundwater / Well	Remarks
Ground Surface				
Asphalt [76.2mm]				
Sand and Gravel [Fill] loose to compact silty SAND, trace to some gravel fill, fine to medium grained sand, 10-20mm subrounded gravel clast, grey, slightly moist to moist	1.5	31.7		
Sand compact to dense SAND, trace to some silt, trace gravel, trace cobbles, tan to grey, moist to wet	5.5	13.9	▼	2.1m observed water table depth
Sand and Gravel loose to compact, silty SAND, trace to some GRAVEL, trace cobbles, till-like, grey, moist	9.8	10.5		
Silt soft to firm SILT, trace sand, trace sand, grey, moist to wet	13.1	27.3		
Sand and Gravel [Till] compact to dense becoming very dense at depth, silty SAND, trace to some GRAVEL till, fine to medium grained sand, grey, moist	15.2	25.4		DCPT refusal at 15.2m
End of Borehole				

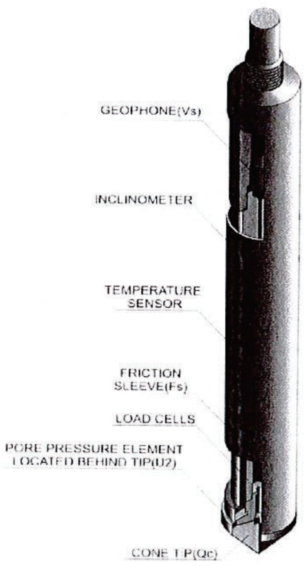


Logged: SH
Method: Solid stem auger
Date: 2018-Apr-04

Datum: Ground elevation
Figure Number: A.04
Page: 1 of 1

APPENDIX B - ELECTRONIC CONE PENETRATION RESULTS

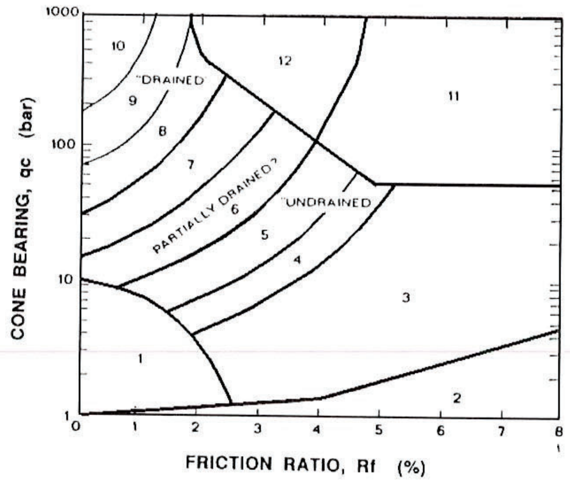
Electronic Cone Penetrometer



The system used is owned and operated by GeoPacific and employs a 35.7 mm diameter cone that records tip resistance, sleeve friction, dynamic pore pressure, inclination and temperature at 5 cm intervals on a digital computer system. The system is a Hogentogler electronic cone system and the cone used was a 10 ton cone with pore pressure element located behind the tip and in front of the sleeve as shown on the adjacent figure.

In addition to the capabilities described above, the cone can be stopped at specified depths and dissipation tests carried out. These dissipation tests can be used to determine the groundwater pressures at the specified depth. This is very useful for identifying artesian pressures within specific layers below the ground surface.

Interpretation of the cone penetration test results are carried out by computer using the interpretation chart presented below by Robertson[1]. Raw data collected by the field computer includes tip resistance, sleeve friction and pore pressure. The tip resistance is corrected for water pressure and the friction ratio is calculated as the ratio of the sleeve friction on the side of the cone to the corrected tip resistance expressed as a percent. These two parameters are used to determine the soil behaviour type as shown in the chart below. The interpreted soil type may be different from other classification systems such as the Unified Soil Classification that is based upon grain size and plasticity.



ZONE	SOIL BEHAVIOUR TYPE
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

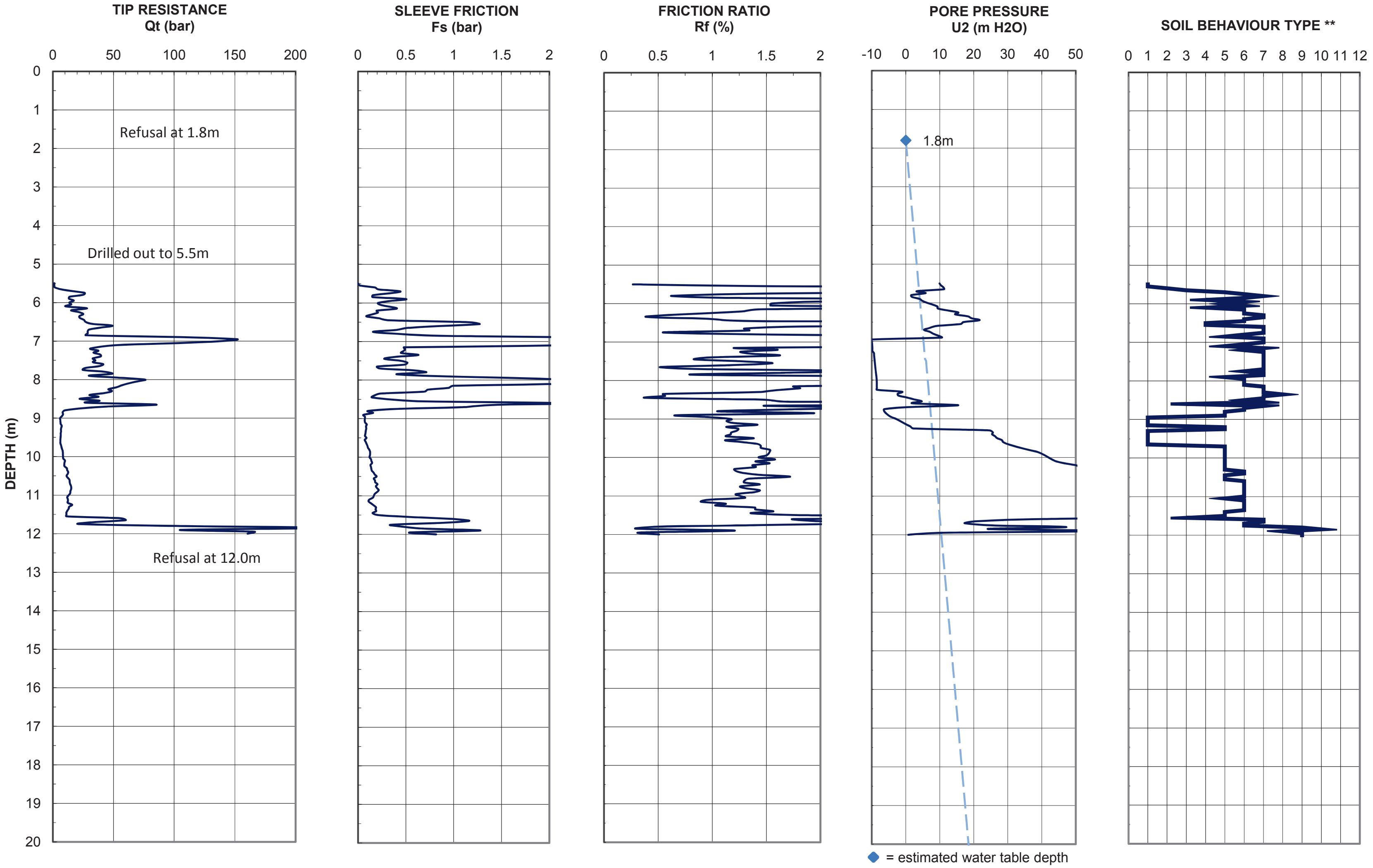
(*) overconsolidated or cemented

---

[1] Robertson, P.K., 1990, "Soil Classification using the cone penetration test", 1990 Canadian Geotechnical Colloquium, Canadian Geotechnical Journal, Vol. 27, No. 1, 1990

Attachment 11

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	**2018-Apr-4**	**PCI DEVELOPMENTS CORP**	**GeoPacific Project #: 16004**
	Sounding: CPT18-01	**3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY**	**Figure: B.01**

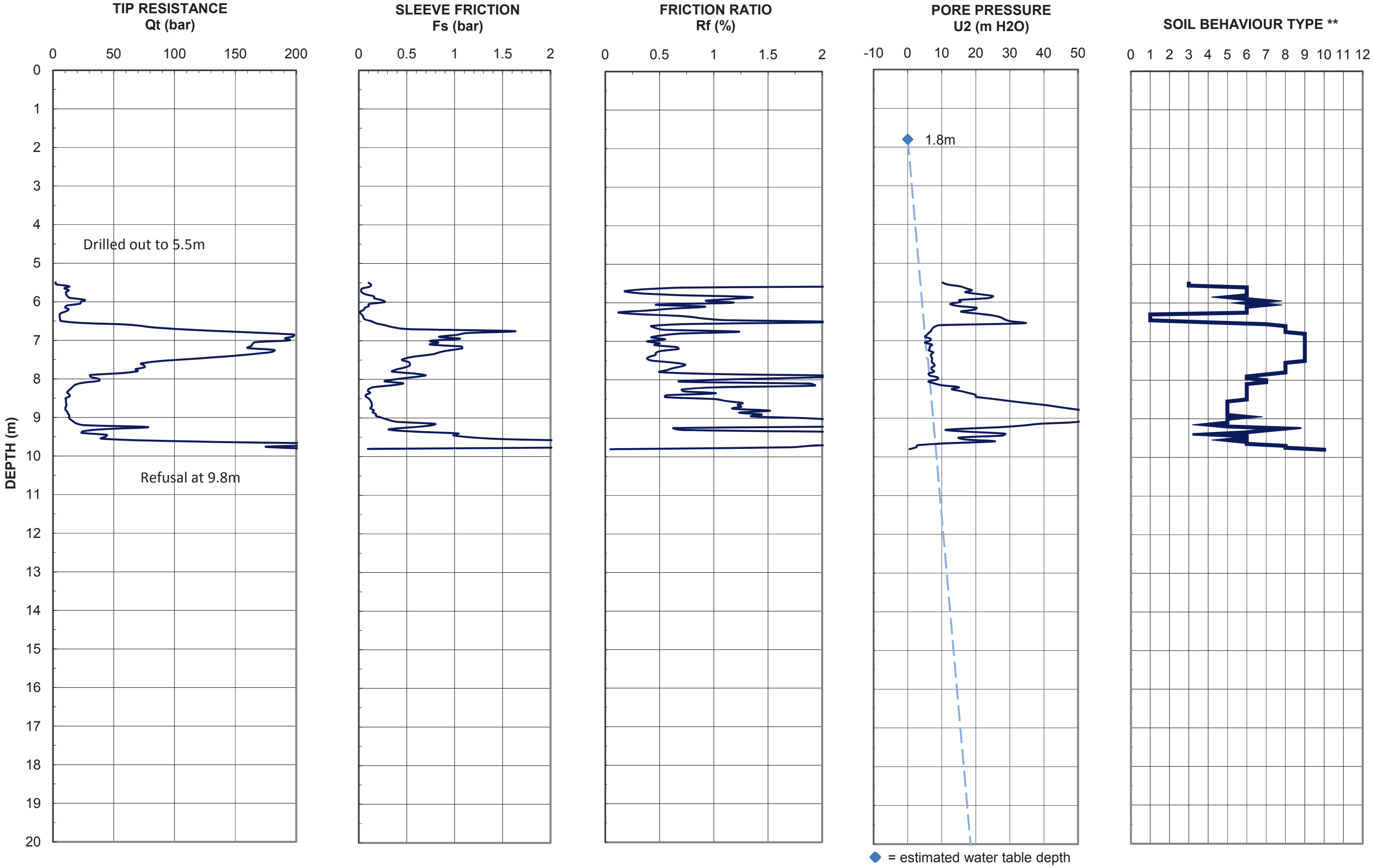


** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

Attachment 11

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
	Sounding: CPT18-02	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: B.02



** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

Attachment 11

GEOPACIFIC
VANCOUVER KAMLOOPS CALGARY

2018-Apr-4

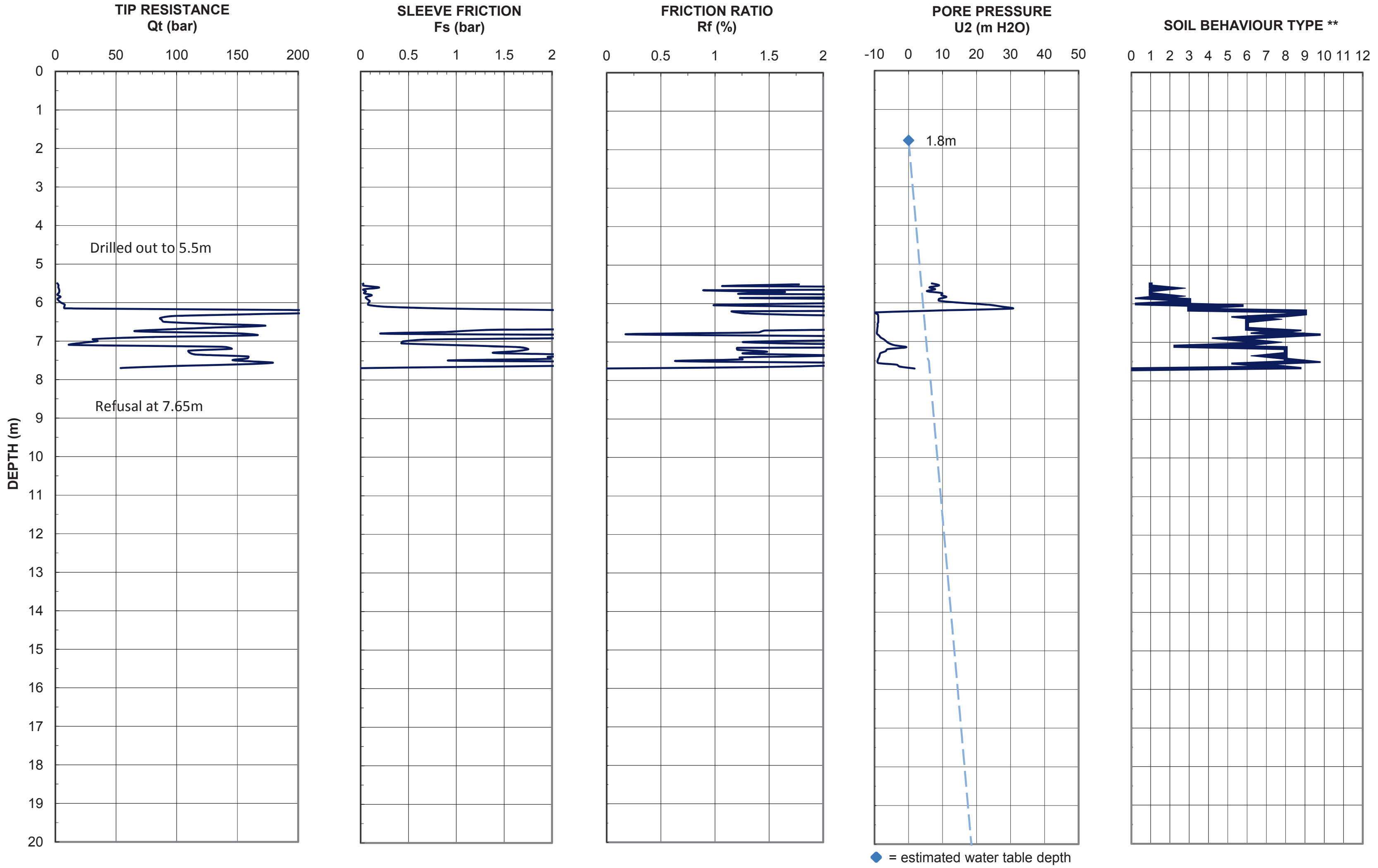
Sounding: CPT18-03

PCI DEVELOPMENTS CORP

3006, 3010, 3060 SPRING STREET AND
3001 MURRAY STREET, PORT MOODY

GeoPacific Project #: 16004

Figure: B.03



** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

Attachment 11

APPENDIX C - OVER CONSOLIDATION RATIO ANALYSIS

The over consolidation ratio (OCR) is defined as the ratio between the maximum past vertical pressure on the soil versus the current in-situ vertical pressure. The maximum past vertical pressure is typically caused by the presence of excess overburden which is removed by either natural or man-made reasons. Soil ageing and other chemical precipitation affects can also cause a soil to behave as if it has a higher maximum past pressure, which is sometimes described as pseudo-overconsolidation.

Research by Schmertmann (1974) showed the following equation reasonably approximates the OCR of medium plastic to clayey soils:

$$OCR = \left(\frac{\left(\frac{Su / p'oc}{Su / p'nc} \right)^{5/3} + 0.82}{1.82} \right)$$

Su/p'oc = The undrained shear strength to effective stress ratio of the over consolidated soil

Su/p'nc = The undrained shear strength to effective stress ratio of a normally consolidated soil (OCR = 1). Typically = ~0.2

Soils which are subject to loads less than the maximum past pressure of the soil are typically subject to relatively small elastic settlements. Loads which exceed the maximum past pressure on the soil typically cause consolidation which is the gradual settlement of the ground as a result of expulsion of water from the pores of the soil. The rate of settlement and the time to complete consolidation is a function of the permeability of the soil.

The Schmertman equation has been employed to estimate the OCR of the soils with depth employing the CPT data provided in Appendix B and C.

APPENDIX C - INTERPRETED PARAMETERS

The following charts plot the Standard Penetration Test (SPT) values and the undrained strength of fine grained soils based upon generally accepted correlations. The methods of correlation are presented below.

STANDARD PENETRATION TEST CORRELATION

The Standard Penetration Test $N_{1(60)}$ value is related to the cone tip resistance through a Qc/N ratio that depends upon the mean grain size of the soil particles. The soil type is determined from the interpretation described in Appendix B and the data of Table C.1 below is used to calculate the value of $N_{(60)}$.

Table C.1. Tablulated $Qc/N_{1(60)}$ Ratios for Interpreted Soil Types

Soil Type	Qc/N Ratio
Organic soil - Peat	1.0
Sensitive Fine Grained	2.0
Clay	1.0
Silty Clay to Clay	1.5
Clayey Silt to Silty Clay	2.0
Silt	2.5
Silty Sand to Sandy Silt	3.0
Clean Sand to Silty Sand	4.0
Clean Sand	5.0
Gravelly Sand to Sand	6.0
Very Stiff Fine Grained	1.0
Sand to Clayey Sand	2.0

The $Qc/N_{1(60)}$ ratio is based upon the published work of Robertson (1985)[2]. The values of N are corrected for overburden pressure in accordance with the correction suggested by Liao and Whitman using a factor of 0.5. Where the correction is of the form:

$$N_1 = \sigma^{0.5} * N$$

All calculations are carried out by computer using the software program CPTint.exe developed by UBC Civil Engineering Department. The results of the interpretation are presented on the following Figures.

UNDRAINED SHEAR STRENGTH CORRELATION

It is generally accepted that there is a correlation between undrained shear strength of clay and the tip resistance as determined from the cone penetration testing. Generally the correlation is of the form:

$$S_u = \frac{(q_c - \sigma_v)}{N_k}$$

where q_c = cone tip resistance, σ = in situ total stress, N_k = cone constant

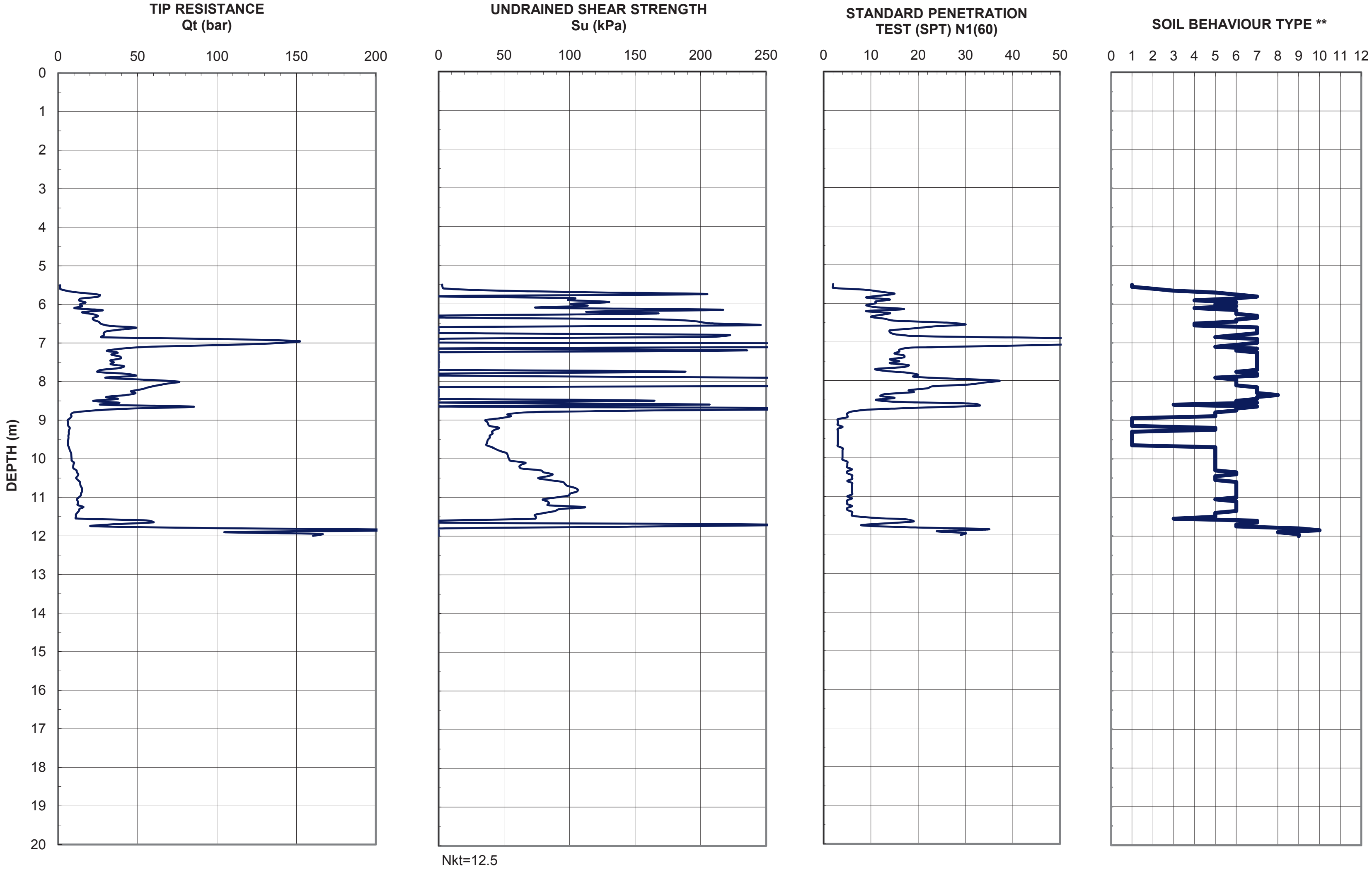
The undrained shear strength of the clay has been calculated using the cone tip resistance and an N_k factor of 12.5. All calculations have been carried out automatically using the program CPTint.exe. The results are presented on the Figures following.

---

[2] Robertson, P.K., 1985, "In-Situ Testing and Its Application to Foundation Engineering", 1985 Canadian Geotechnical Colloquium, Canadian Geotechnical Journal, Vol. 23, No. 23, 1986

Attachment 11

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
	Sounding: CPT18-01	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: C.01



** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

Attachment 11

GEOPACIFIC
VANCOUVER KAMLOOPS CALGARY

2018-Apr-4

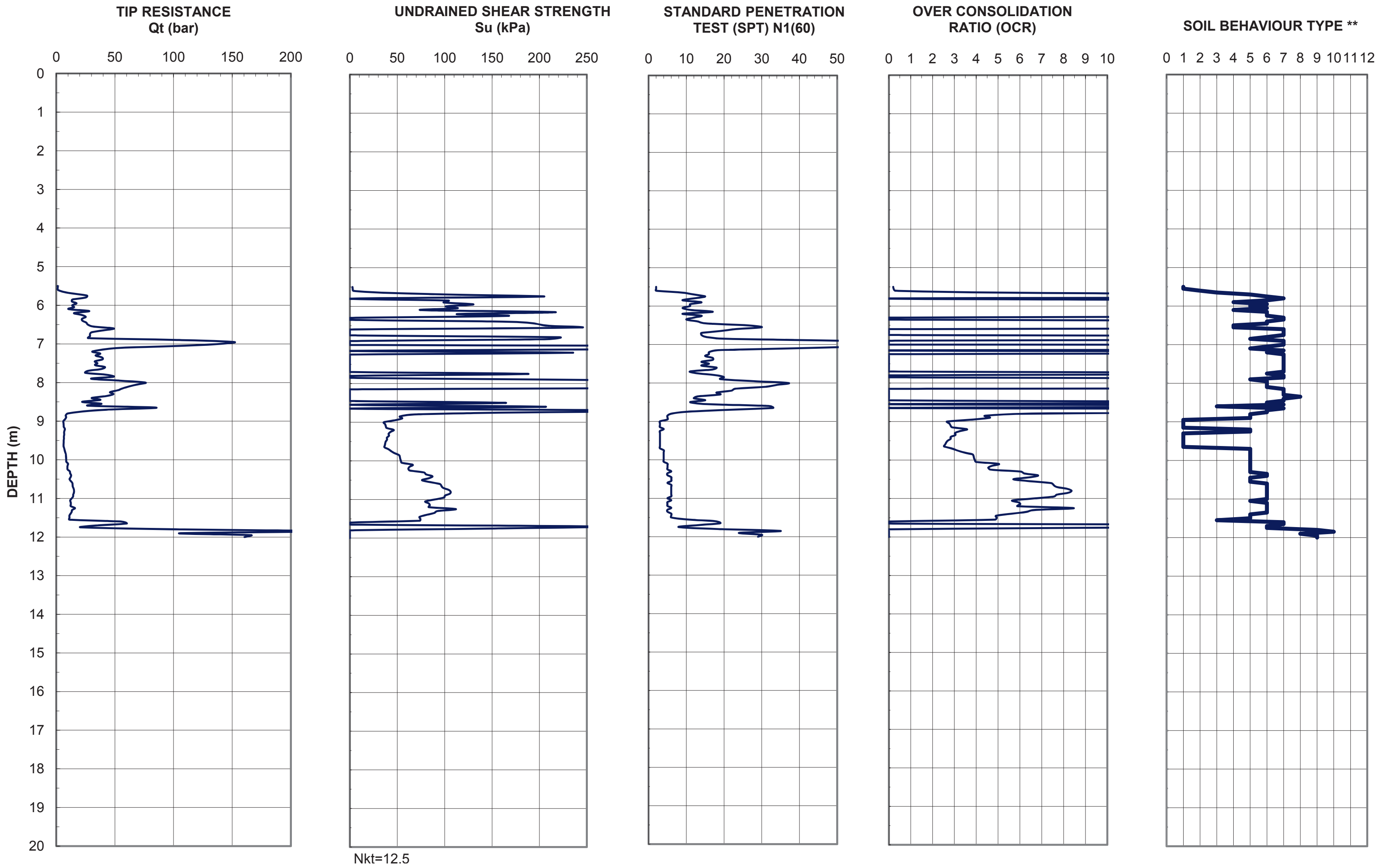
Sounding: CPT18-01

PCI DEVELOPMENTS CORP

3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY

GeoPacific Project #: 16004

Figure: C.01



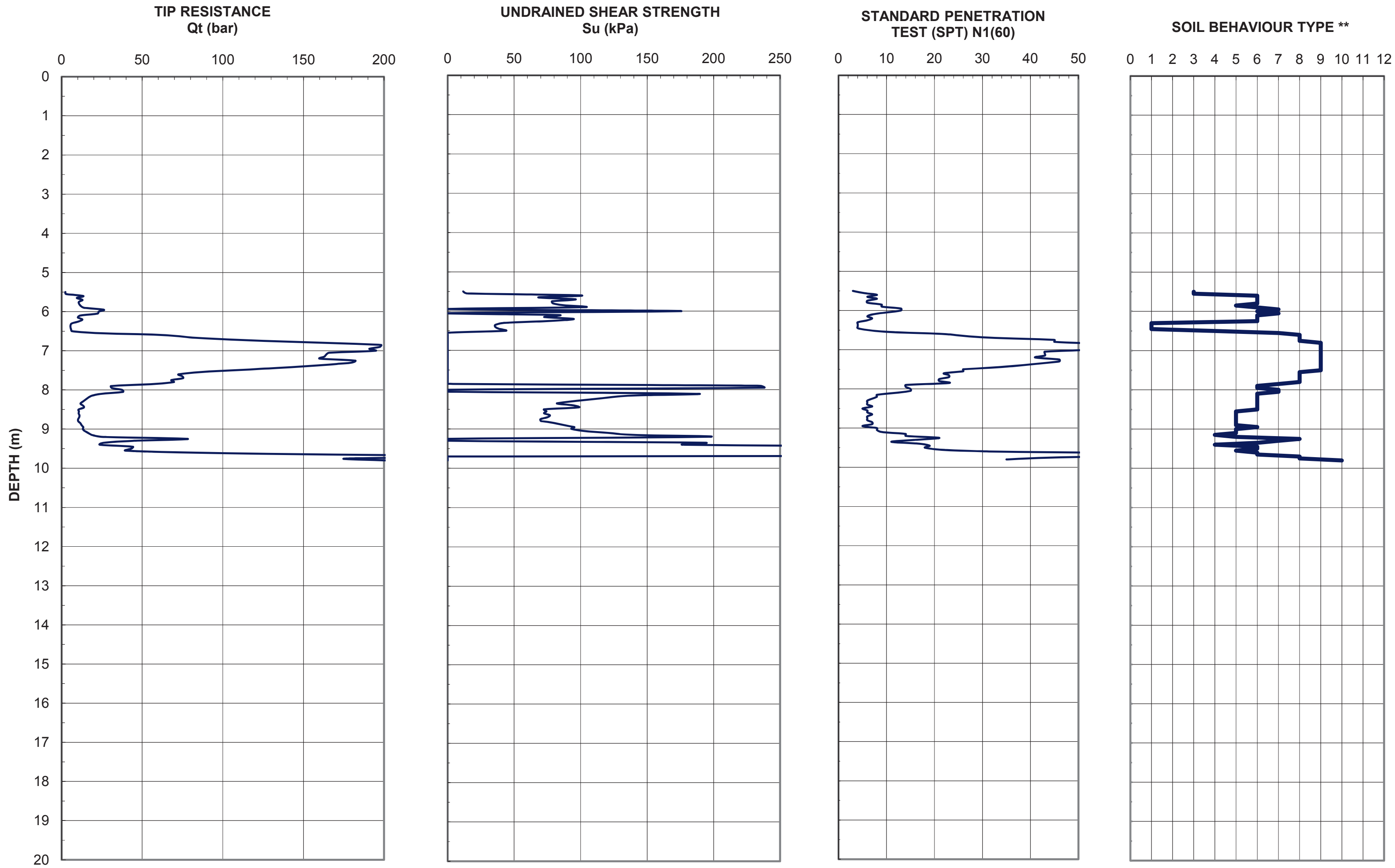
** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

Attachment 11

GEOPACIFIC
VANCOUVER KAMLOOPS CALGARY

GEOPACIFIC	2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
	Sounding: CPT18-02	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: C.02



Nkt=12.5

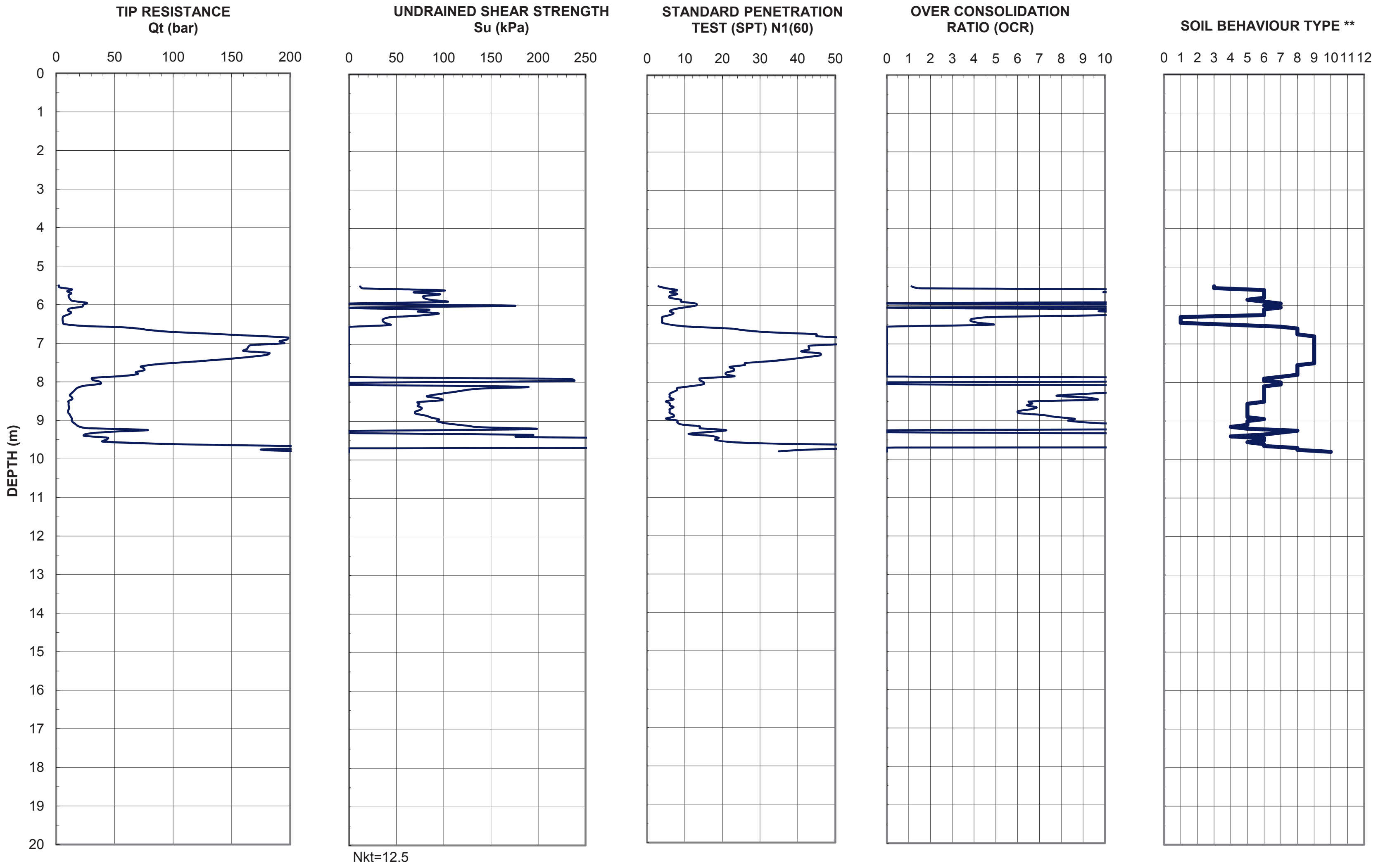
** Based on Robertson et. al 1986

1 Sensitive Fine Grained	4 Silty Clay to Clay	7 Silty Sand to Sandy Silt	10 Gravelly Sand to Sand
2 Organic Material	5 Clayey Silt to Silty Clay	8 Sand to Silty Sand	11 Very Stiff Fine Grained
3 Clay	6 Sandy Silt to Clayey Silt	9 Sand	12 Sand to Clayey Sand

Attachment 11

GEOPACIFIC
VANCOUVER KAMLOOPS CALGARY

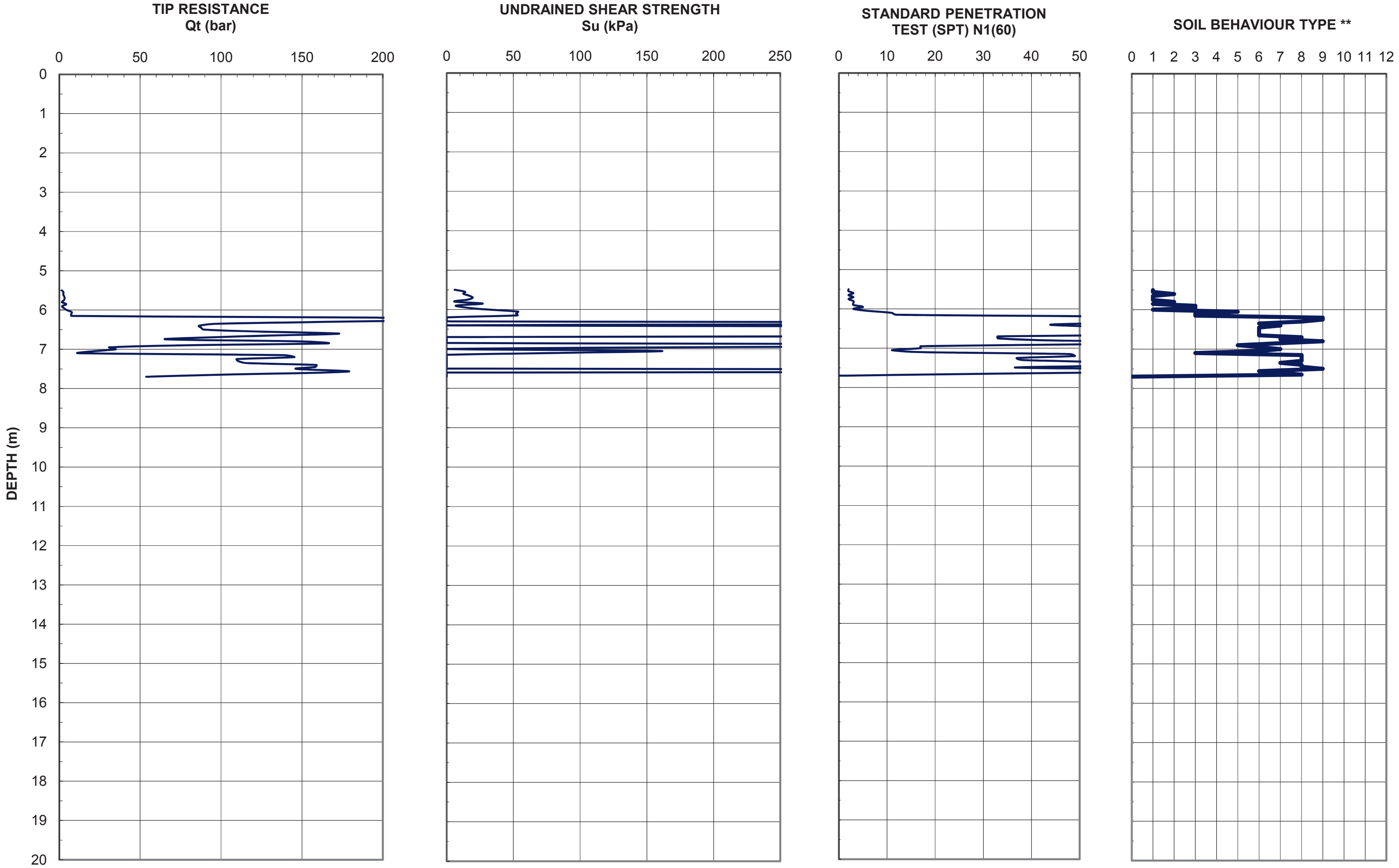
2018-Apr-4	**PCI DEVELOPMENTS CORP**	**GeoPacific Project #: 16004**
Sounding: CPT18-02	**3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY**	**Figure: C.02**



** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
	Sounding: CPT18-03	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: C.03



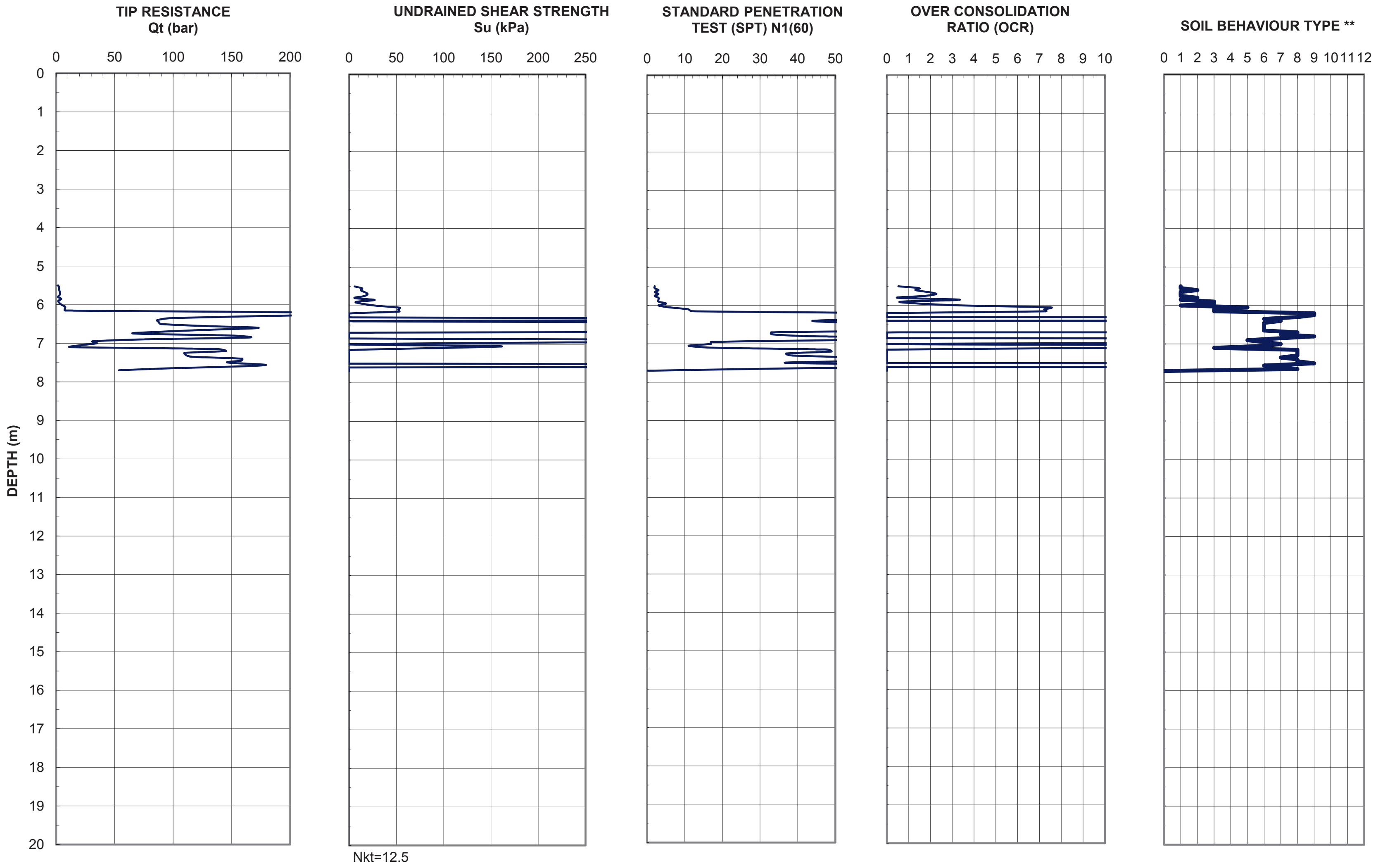
Nkt=12.5

** Based on Robertson et. al 1986

1 Sensitive Fine Grained
2 Organic Material
3 Clay
4 Silty Clay to Clay
5 Clayey Silt to Silty Clay
6 Sandy Silt to Clayey Silt
7 Silty Sand to Sandy Silt
8 Sand to Silty Sand
9 Sand
10 Gravelly Sand to Sand
11 Very Stiff Fine Grained
12 Sand to Clayey Sand

Attachment 11

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
	Sounding: CPT18-03	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: C.03



** Based on Robertson et. al 1986

1 Sensitive Fine Grained	4 Silty Clay to Clay	7 Silty Sand to Sandy Silt	10 Gravelly Sand to Sand
2 Organic Material	5 Clayey Silt to Silty Clay	8 Sand to Silty Sand	11 Very Stiff Fine Grained
3 Clay	6 Sandy Silt to Clayey Silt	9 Sand	12 Sand to Clayey Sand

Attachment 11

APPENDIX D - LIQUEFACTION ANALYSIS

Assessment of the liquefaction potential of the ground has been determined by the Cone Penetration Test (CPT). The method of analysis is presented in the following sections.

FACTOR OF SAFETY AGAINST LIQUEFACTION

The factor of safety against liquefaction calculated here is the ratio of the cyclic resistance of the soil (CRR) to the cyclic stresses induced by the design earthquake (CSR). Where the ratio of CRR/CSR is greater than unity the soils ability to resist cyclic stresses is greater than the cyclic stresses induced by the earthquake and liquefaction will be unlikely. Where the CRR/CSR is less than unity then liquefaction could occur. This ratio is presented as the FOS against Liquefaction on the following charts. Calculation of the factor of safety is based on NCEER (1998)[1] which evaluates the CRR directly from cone penetration test sounding data. The value of the cyclic stress ratio has been calculated based on peak horizontal ground acceleration of the 2015 National Building Code interpolated seismic hazard value.

SEISMIC INDUCED SETTLEMENT

In the event of a significant earthquake, settlement of the ground surface could occur as a result of densification of the looser soil layers as a result of liquefaction or due to the expulsion of sand in the form of sand dykes or sills from beneath the site. Tokimatsu and Seed (1987)[2] suggest a method of analysis for estimating vertical settlements as a result of earthquake induced accelerations. In this method the normalized standard penetration blow counts ($N_{1(60)}$) is compared with the cyclic stress ratio for the induced earthquake to determine the volumetric strain resulting from the earthquake shaking. The volumetric strain is assumed to result in only vertical settlement. The vertical settlement is summed for each depth at which settlement is predicted to occur and accumulated from the bottom of the test hole. The results are presented on the following charts labelled as Settlement.

HORIZONTAL DISPLACEMENT

Horizontal ground displacements known as "free field" displacements occur as a result of liquefaction of the ground and are assumed to occur without the influence of any structures. The horizontal displacements presented in our report are generally based upon the lateral spread method by of Youd, Bartlett, & Hansen (2002). Displacements are calculated based on an empirical relationship developed from observations from other earthquake sites on sloping ground or near a free face, such as an abrupt slope. The presence of the proposed embankment on-site is expected to induce a static bias within the soils at the margin of the embankment making the soils and embankment in this area subject to lateral spread induced movements. In the event of a real earthquake of significant magnitude to cause limited liquefaction, actual movements will be influenced by a wide variety of factors including the characteristics of the earthquake including duration, number of significant cycles, variations in peak particle velocity, wavelength, amplitude and frequencies as well as soil damping and variations in density and continuity of the soil layers.

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[1] Youd, T. L., Idriss, I. M. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, Vol 127, 10, pp. 817-833

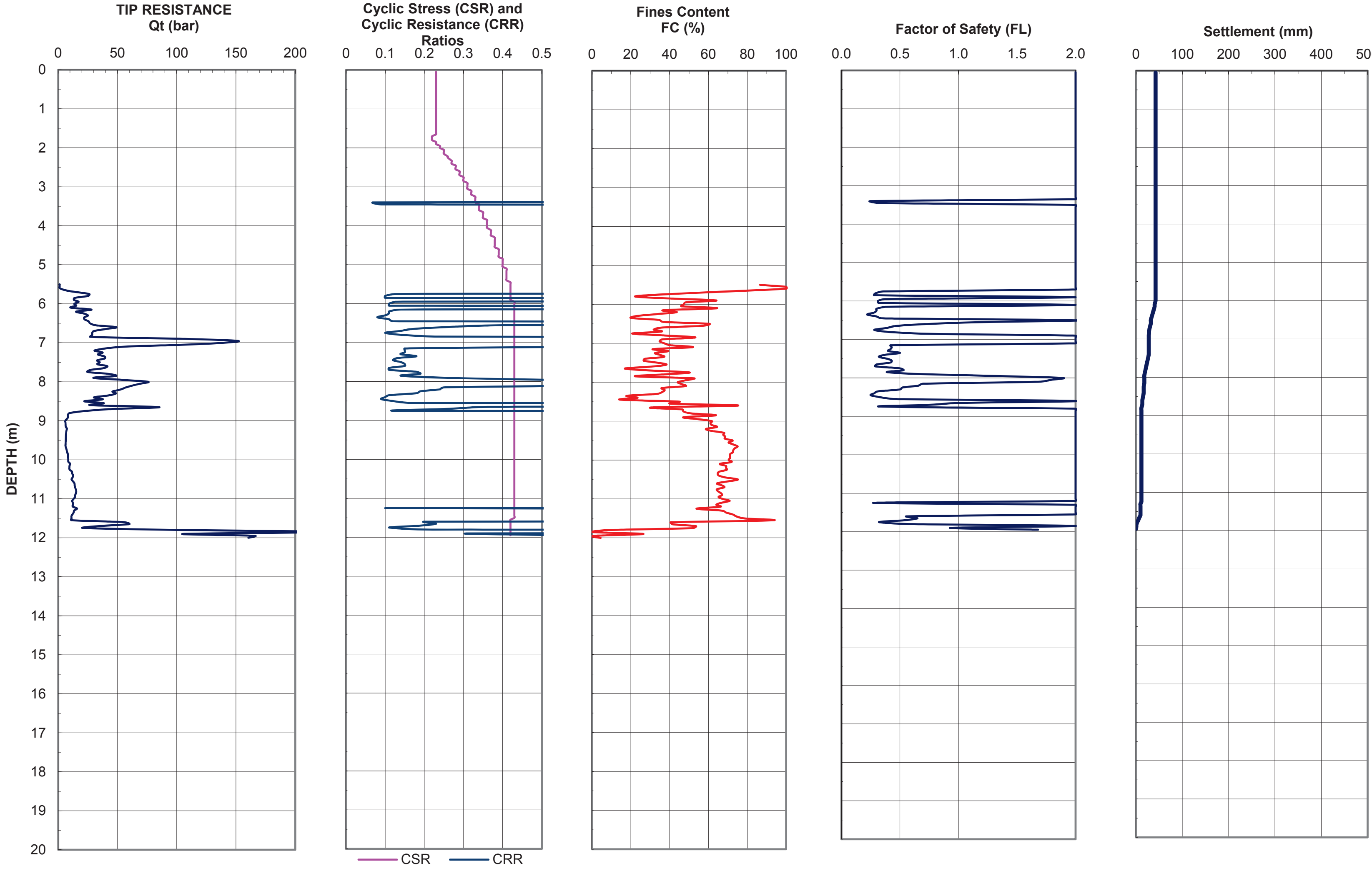
[2] Tokimatsu, K.A.M. and Seed, H.B., 1987. "Evaluation of Settlement in Sands Due to Earthquake Shaking", Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, pp. 861-878.

[3] Youd, T.L., Bartlett, S.F., Hansen, C.M. (2002), "Revised MultiLinear Regression Equations for Prediction of Lateral Spread Displacements", Journal of Geotechnical and GeoEnvironmental Engineering, Vol. 128, No. 12, pp. 1007-1017

Attachment 11

GEOPACIFIC
VANCOUVER KAMLOOPS CALGARY

2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
Sounding: CPT18-01	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: D.01



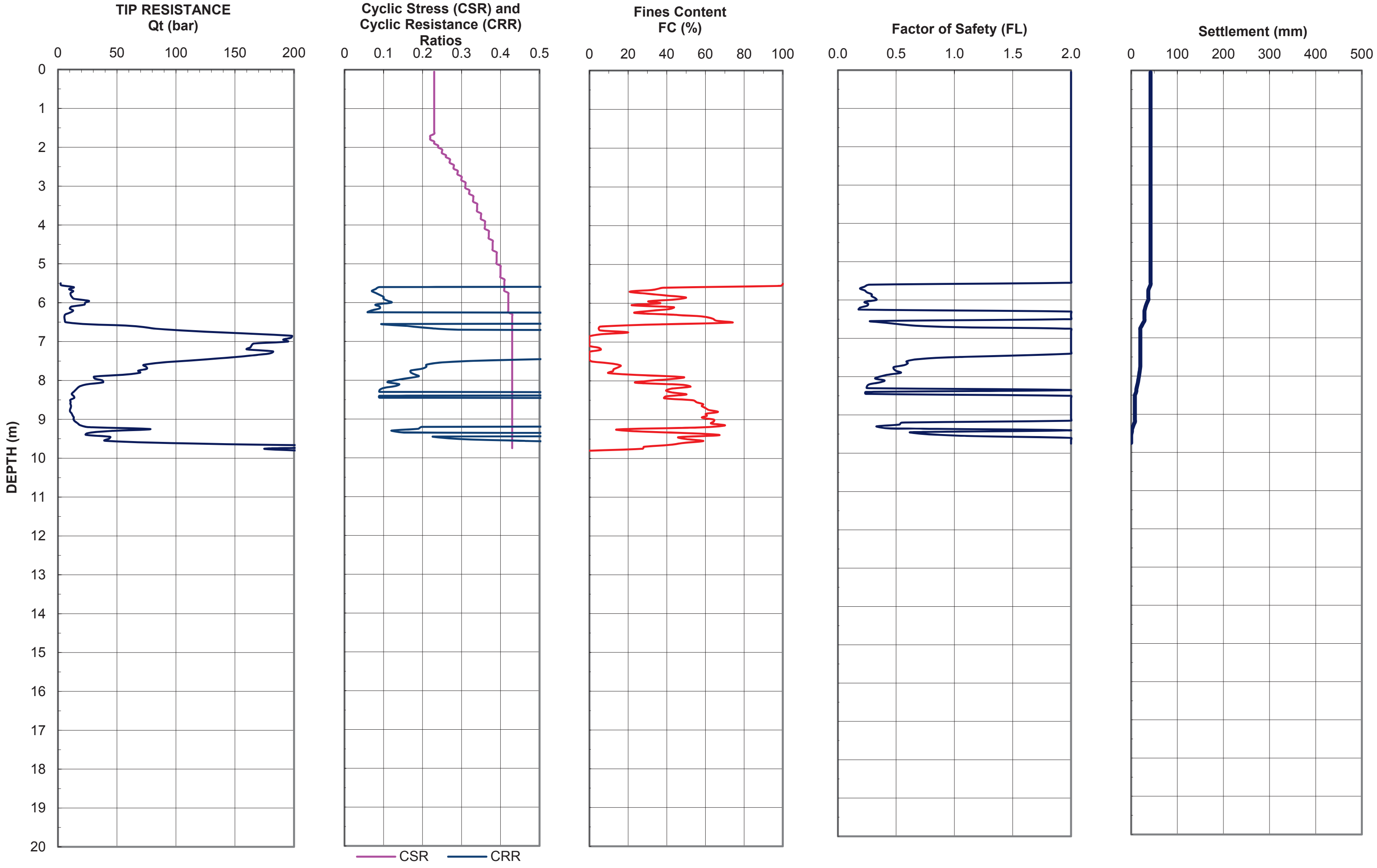
Liquefaction interpretation: PGA = 0.35

magnitude = 7.0

settlement accumulation max depth = 15m

Attachment 11

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	**2018-Apr-4**	**PCI DEVELOPMENTS CORP**	**GeoPacific Project #: 16004**
	Sounding: CPT18-02	**3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY**	**Figure: D.02**



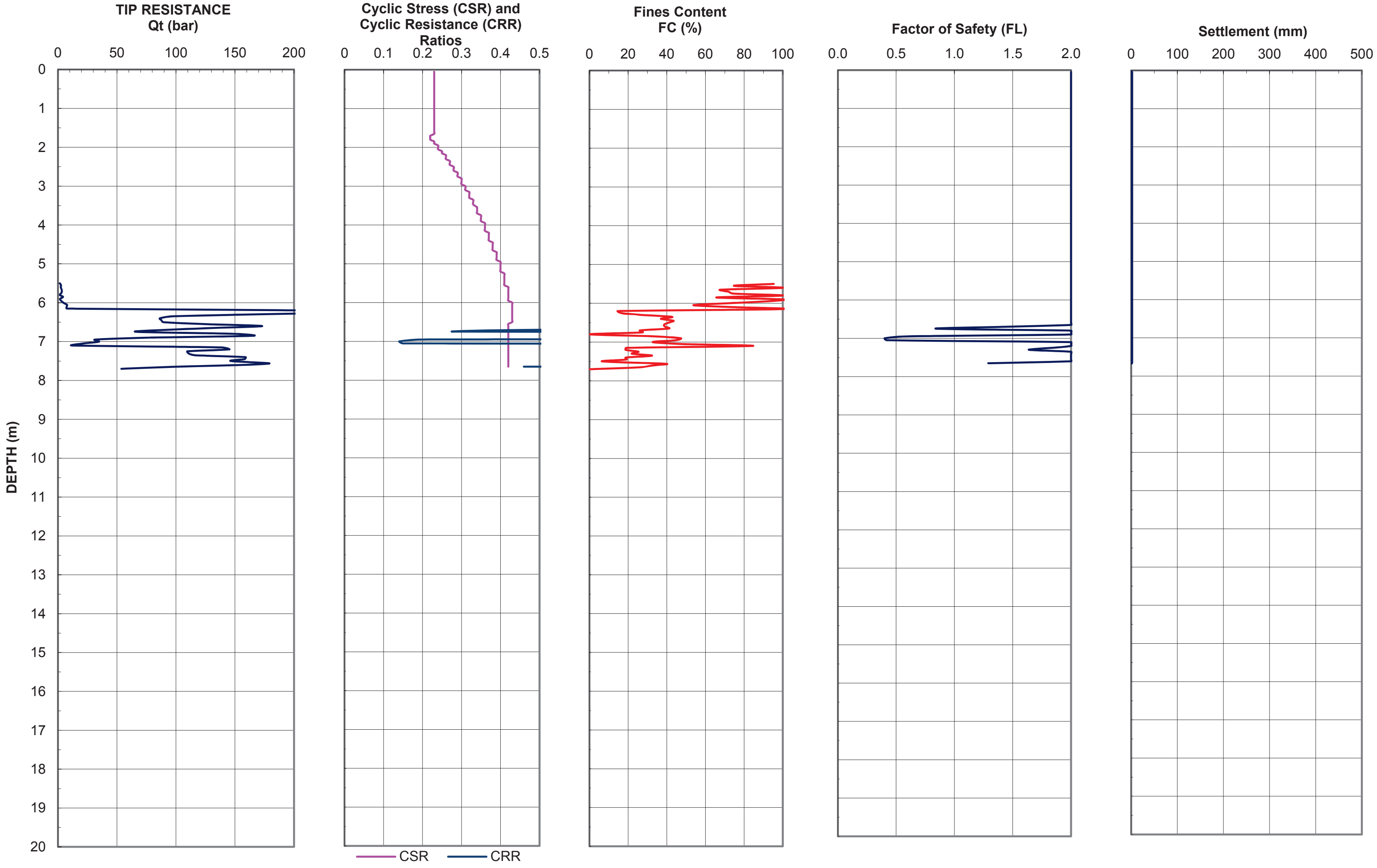
Liquefaction interpretation: PGA = 0.35

magnitude = 7.0

settlement accumulation max depth = 15m

Attachment 11

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY	2018-Apr-4	PCI DEVELOPMENTS CORP	GeoPacific Project #: 16004
	Sounding: CPT18-03	3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY	Figure: D.03



Liquefaction interpretation: PGA = 0.35

magnitude = 7.0

settlement accumulation max depth = 15m

Attachment 11

APPENDIX E - SHEAR WAVE VELOCITY DATA (Vs)

Attachment 11

GEOPACIFIC
VANCOUVER KAMLOOPS CALGARY

File: 16004
Project: MIXED USE DEVELOPMENT
Client: PCI DEVELOPMENTS
Location: 3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY, BC
Sounding: SCPT18-01
Date: 2017-Apr-04

Seismic Source: Beam
Source to cone (m): 0.4

Shear Wave Velocity Data (Vs)

Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference d (m)	Midpoint (m)	Time Difference (ms)	Shear Wave Velocity Vs (m/s)	d/Vs
3.95	3.75	3.77	3.77	1.88	28.98	130	0.0290
5.25	5.05	5.07	1.29	4.40	3.77	344	0.0038
6.25	6.05	6.06	1.00	5.55	4.03	247	0.0040
7.25	7.05	7.06	1.00	6.55	4.61	216	0.0046
8.25	8.05	8.06	1.00	7.55	4.02	248	0.0040
9.25	9.05	9.06	1.00	8.55	5.98	167	0.0060
10.25	10.05	10.06	1.00	9.55	5.35	187	0.0054
11.20	11.00	11.01	0.95	10.53	4.49	211	0.0045
						Σ(d/Vs)	0.0612
						average Vs = Σd / Σ(d/Vs)	180

Attachment 11

File: 16004
Project: MIXED USE DEVELOPMENT
Client: PCI DEVELOPMENTS
Location: 3006, 3010, 3060 SPRING STREET AND 3001 MURRAY STREET, PORT MOODY, BCSounding:
SCPT18-01
Date: 2018-April-4