GeoPacific
Consultants Ltd.

#215 -1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
Phone (604) 439-0922 / Fax (604) 439-9189

Squamish United Church
c/o
Cascadia Consulting
Box 1572 ~ #200-37700 2nd Avenue
Squamish, BC V8B 0B2

October 6, 2008
File: 7806

Attention: Mr. Peter Gordon

Re: Preliminary Geotechnical Investigation Report, Proposed Mixed-Use Development, 38014 Fourth Avenue, Squamish, BC

1.0 INTRODUCTION

We understand that it is proposed to construct a multi-use facility at the above referenced address. The project is in the conceptual development stage and there are no architectural or structural drawings available at this time, however, we understand that the structure will be two to three storeys in height.

We expect that the structure will be of reinforced concrete or masonry block construction on the lowest level and be of wood framed construction on the upper floors. We expect that the building will impose an average area load in the range of 300 psf with column and wall loads in the range of 100 kips and 2.5 kips/lineal foot respectively.

This report presents the results of a geotechnical investigation of the soil and groundwater conditions at the site and presents geotechnical recommendations for site preparation and design for new buildings, on-site pavement structures and related earthworks. Our recommendations should be reviewed once the architectural and structural drawings are available to confirm that they remain valid.

2.0 SITE DESCRIPTION

The site is comprised of five properties described as Lots 16 to 20, Block 4, DL 486, Gp 1 NWD, Plan 3960. The site is located on the northeast corner of the intersection of Fourth Avenue and Victoria Street in Squamish. The site is bounded by a residential property to the north and District of Squamish (DOS) lands on three sides; Fourth Avenue to the west, Victoria Street to the south and a Laneway to the east.

The site is improved with the existing Squamish United Church on Lots 16 and 17, a gravel parking lot on Lot 18, an existing structure on Lot 19 and Lot 20 is covered in grass. The site grades range from 1 to 2 m in elevation with generally lower elevations towards the margins of the site.

3.0 FIELD INVESTIGATION

The site was investigated on August 27, 2008, using a truck mounted auger drill rig sub-contracted from Uniwide Drilling Co. Ltd. of Burnaby, BC, along with electronic cone penetration testing (CPT) equipment

owned and operated by GeoPacific. The auger test holes were advanced to depths ranging from 6.1 to 7.5 m below exiting grades. The CPT's were advanced to depths of 30 m. The test hole logs are included in Appendix A.
Analysis of the CPT data allows for classification of the sub-surface stratigraphy based on soil-type behaviour characteristics and the estimation of geotechnical design parameters. As the cone penetrometer is advanced into the ground it records the tip resistance, sleeve friction, pore water pressure, temperature and inclination every 5 cm. The CPT results are presented in Appendix B. The geotechnical parameters calculated from the CPT, including undrained shear strength and standard penetration $N_{1(60)}$ values, are presented in Appendix C. The results of our CPT based liquefaction assessment, based on the estimated peak ground accelerations at the site during the 2006 British Columbia Building Code design earthquake, are presented in Appendix D.

A single seismic cone penetration test (SCPT) was undertaken to collect shear wave velocity data. The data obtained from the SCPT is proportional to the small strain stiffness of the soils which we have considered in the seismic analysis for this site. The shear wave velocity results recorded at SCPT08-01 are presented in Appendix E.

The test hole locations are shown on Drawing 7806-01.

4.0 SUBSURFACE CONDITIONS

4.1 Soil Conditions

The geology of the region is comprised of fluvial deposits typical of those found in the Squamish River valley. A general description of the soils encountered at our test hole locations is given below.

ASPHALT / VARIABLE FILL / TOPSOIL

The original ground surface has been overlain by up to 0.6 m of variable granular fill at test hole locations TH08-01 through TH08-04. Asphalt has been placed over the fill materials in the parking lot area at test holes TH08-01 and TH08-02. Topsoil was noted underlying the fill materials at TH08-01. Where not surfaced in asphalt the site is overlain with topsoil.

SILT

The materials above are underlain by low plastic silt. The silt is highly desiccated and stiff above the water table and becomes firm with depth. Undrained shear strength values as low as 30 kPa were measured below the desiccated crust. Grab samples were found to have moisture contents ranging from 35 to 68 percent. The silt at TH08-05 was noted to include trace amounts of organic materials. The silt was found to extend to depths ranging from 3.0 to 3.8 m.

SAND

The silt is underlain by interbedded fine to coarse grained sand ranging from well graded to well sorted. The sand contains occasional silt lenses, organic inclusions and wood debris. The sand is compact at the top of the stratum and becomes loose with depth. The sand was found to extend to depths in the range of 10 m.

SILT to SAND

The sand is underlain by a deposit of interbedded sediments comprised of silt to sand in varying proportion. The silt is interpreted to be non-plastic. Both the silt and sand are interpreted to be loose to compact. This deposit was found to extend to depths beyond those of our investigation.

For a more detailed description of the sub-surface soil conditions please refer to the individual test hole logs located in Appendix A and the CPT sounding logs located in Appendix B.

4.2 Groundwater Conditions

The groundwater table was noted at depths ranging from 1.2 to 1.4 m at the time of our investigation. Elevated groundwater levels can be expected following periods of extended rainfall and/or snow melt.

5.0 DISCUSSION

5.1 General Comments

5.1 Flood Hazard Considerations

The slab elevation of the proposed structure is not known a this time, however, based on existing frontage grades we expect that it will be in the range of 1.5 to 2.0 m geodetic elevation.

The proposed development site is located in a flood plain and therefore may be subject to flooding. Based on our review of the District of Squamish, Flood Hazard Management Plan, prepared by Klohn Leonoff and dated May 1994, we have interpolated the flood control level (FCL) at this location to be at 3.65 m. Therefore, it is anticipated that, the lowest level of the proposed development will be located below the FCL.

We understand that this portion of Squamish is protected by a dyke system, however, it should be appreciated that should the dyke system be breached and/or overtopped that flooding and associated damage may ensue. All habitable space must be constructed such that the underside of the joist box is at or above the local flood plain elevation.

5.2 Static Analysis

The site is underlain by up to 3.2 m of compressible silt over a thick sequence of compact to loose channel sands overlying a deposit of interbedded silt to sand. The near surface low-plastic silt is considered to be moderately compressible under the anticipated stress increases from the proposed development. The deep interbedded silt to sand should not consolidate significantly under the anticipated building loads.

We expect that the building loads will likely result in excessive ground settlements if placed on site at current grades without removal and replacement of existing fills and preloading the site to limit post construction ground settlements. After completion of the recommended site preparation we expect that it will be possible to found the structure on conventional strip and pad foundations at relatively low bearing pressures.

Once the building elevation has been established, and detailed structural loads are available our recommendations should be reviewed to ensure that they still apply.

5.2 Seismic Analysis

Significant zones within the loose to compact sands and non-plastic silts are prone to liquefaction or strain softening during cyclic loading caused by large earthquakes. The strength reduction caused by soil liquefaction can cause foundations to punch. In addition, once liquefaction has been triggered, significant, permanent, vertical and horizontal movements may be experienced.

Liquefaction and estimated post liquefaction ground settlements are presented in Appendix D of this report. The design earthquake is expected to have a magnitude of 7.0 and peak acceleration of 0.33g on firm ground. Firm ground as defined in the 2006 BCBC was not encountered at the site. Review of our findings and past experience indicates that firm ground is in excess of 50 m and thus significant de-amplification of ground motions in this region of Squamish.

Our review of the anticipated ground motions, foundation loads, building elevation and the residual strength of materials in the event of liquefaction and/or strain softening indicates that there will be sufficient strength, following liquefaction, to support the foundations in the event of the design earthquake.

It should be appreciated that, following a large magnitude, long duration earthquake, some damage is likely due to settlement and horizontal movement of the underlying soils and that repairs to the structure may be required. Post earthquake permanent vertical settlements could be in the range of 100 to 200 mm, while permanent horizontal ground movements could be in the range of 0.3 to 0.8 m.

6.0 DESIGN RECOMMENDATIONS

6.1 Stripping

Site preparation associated with the proposed development includes removing any vegetation, topsoil, asphalt, peat, organic material, debris, fill and any other material considered to compromise the design recommendations. These "unsuitable" materials should be excavated to expose a subgrade consisting of firm native silt deposits. Estimated minimum stripping depths at our test hole locations shown in Table 1 below.

Table 1: Estimated Minimum Stripping Depths for Foundation

Test Hole	Stripping Depth (m)
TH08-01	0.3
TH08-02	0.5
TH08-03	0.6
TH08-04	0.6
TH08-05	0.2

Site stripping must extend beyond the building envelope a distance equivalent to the vertical distance between the building foundation and the stripping depth. For example, if the building foundation will be located 1.5 m above the recommended stripping depth, then, the excavation of unsuitable soil must extend horizontally outside of the building envelope a distance of 1.5 m. Where this 1:1 offset can not be achieved the foundations must be lowered as required to achieve the required offset.

The stripped site should be graded to inhibit ponding of water on the firm native silt. Any water softened soils must be removed.

All stripped subgrades must be reviewed by GeoPacific prior to covering.

6.2 Grade Reinstatement

Once stripping is complete "engineered fill" should be used to raise ground to the underside of foundations. "Engineered fill" should be clean sand to sand and gravel with not more than 10% by weight passing the #200 sieve, compact to a minimum of 98% of its Standard Proctor Maximum Dry Density (ASTM D698) at a moisture content that is within 2% of its optimum for compaction. Engineered fill should be compacted in loose lifts not exceeding 300 mm with a vibratory compactor. However, the first lift placed upon the exposed firm silt subgrade should be at least 600 mm to not disturb the underlying silt during compaction.

6.3 Preload

6.3.1 General

Preloading the building site is the most effective method of reducing long term settlements associated with the consolidation of the surficial silt. As there will be very little stress attenuation between the underside of the foundations and the compressible stratum the pre-load height would be directly proportional to the contact stress at the underside of the foundations. The pre-load duration period is estimated to be 8 to 10 weeks at this location.

We expect that a 3 m preload, measured from the top of slab-on-grade, would be sufficient for this project. Once the building elevation and structural loads are confirmed we could provide a detailed preload design. Where the preload slopes encroach on adjacent facilities or property a reinforced lock-block retaining wall would be required.

We recommend that the building be set back from settlement sensitive structures to minimize the potential for damage caused by preload induced ground settlements. Provisions should be made to repair any damage to neighbouring properties, including roads and services, induced by preloading.

Landscape fills which are to be above current grades must be placed simultaneously with the required building preload. Placement of landscape fills after building construction may result in adverse settlement of the adjacent building.

6.3.2 Construction Monitoring

Monitoring of the preload, adjacent structures, sidewalks, roads and other improvements are recommended to assist in confirming that the actual settlements are as anticipated and to allow modification to the site preparation methodology if required. The monitoring should be completed by a registered BCLS, with results forwarded to our office directly for review. The monitoring should commence at least 2 weeks before commencement of pre-load construction to provide adequate baseline information. Monitoring should be completed weekly for one month and then twice monthly.

6.4 Foundations

6.4.1 Strip and Pad Foundations

Following preloading we recommend that the structure be founded on reinforced strip and pad foundations. The footings should be reinforced to prevent lateral spreading of foundations in a seismic event and therefore all foundations should tied together.

We recommend that the structural design be based on a serviceability limit pressure of 50 kPa (1,000 psf) for foundations constructed on firm silt as described in this report. A factored ultimate resistance of 75 kPa (1,500 psf) can be considered for short term loadings such as those induced by winds or earthquakes.

All footing subgrades must be reviewed by GeoPacific prior to construction.

6.4.2 Settlement of Foundations

Long term (25 year) settlements in the range of 25 to 50 mm should be expected in the area of the development. The long term settlements are expected to be deep seated and therefore related differential settlements are expected to be small.

6.4.3 Seismic Design of Foundations

We have based our seismic recommendations on the test hole information, shear wave velocity profile, our extensive experience in this area of Squamish and numerous site specific dynamic analyses which we have carried out.

Based on the results of our analysis, and an expected building period of less than 0.5 seconds, we recommend that the structure be designed in accordance with "Site Class E" as defined in Table 4.1.8.4.A of the 2006 British Columbia Building Code (2006 BCBC).

6.5 Concrete Slabs on Grade

All grade supported concrete slabs should be underlain by a minimum of 150 mm of 3/4 inch clear crushed gravel to prevent moisture from accumulating below the slab. The gravel should be placed over the firm silt or compacted engineered fill as described in this report. The gravel should be lightly tamped in place. A polyethylene moisture barrier should be placed beneath the grade supported slab in accordance with the Building Code.

6.6 Site and Foundation Drainage Systems

The building floors are anticipated to be above the long term static groundwater level. We recommend that the mechanical design include a conventional perimeter drainage system to intercept and dispose of any migrating groundwater. The under-slab fill should be hydraulically connected to the perimeter drainage system.

6.7 Temporary Excavations

The excavation should be sloped where possible. Temporary slopes constructed in the surficial fill and silt above the water table should be stable for several weeks when sloped at 1H:1V and covered in poly sheeting. Excavation slopes below the water table may have to be flattened to 2H:1V or less and may require the use of Lock-Blocks to maintain a safely sloped excavation. For deeper excavations penetrating below the watertable shoring in the form of steel cages may be necessary.

All excavations must be carried out in accordance with WorkSafe BC standards.

6.8 On-Site Pavement Structures

Following the recommended site preparation, as outlined in Section 6.1 and 6.2 of this report, we are of the opinion that the following pavement structure will be sufficient to carry the expected vehicular loading for on-site roads and parking areas.

Table 2: Recommended <u>Minimum</u> Pavement Structure For Roads and Parking Areas

Material	Thickness (mm)
Asphaltic Concrete	75
19 mm minus crush gravel base	100
100 mm minus, well graded, clean, sand and gravel subbase course	200

All base and sub-base materials should be compacted to a minimum of 98% of their Standard Proctor maximum dry density (ASTM D698), at a moisture content that is within 2% of optimum for compaction.

7.0 FIELD REVIEWS

As required for Municipal "Letters of Assurance", GeoPacific Consultants Ltd. will carry out sufficient field reviews during construction to ensure that the Geotechnical Design recommendations contained within this report have been adequately communicated to the design team and to the contractors implementing the design. These field reviews are not carried out for the benefit of the contractors and therefore do not in any way effect the contractors obligations to perform under the terms of their contract.

It is the contractors' responsibility to advise GeoPacific Consultants Ltd. (a minimum of 48 hours in advance) that a field review is required. Geotechnical field reviews are normally required at the time of the following:

1. Stripping – Review of stripping depth to suitable subgrade materials
2. Pre-loading – Review of pre-load placement and performance
3. Fill – Review of any engineered fill used to raise grades
4. Subgrade – Review of foundation soils for strip and pad foundations
5. Excavations – Review of excavations in excess of 1.2 m in depth requiring man-entry

It is critical that these reviews are carried out to ensure that our intentions have been adequately communicated. It is also critical that contractors working on the site view this document in advance of any work being carried out so that they become familiarised with the sensitive aspects of the works proposed. It is the responsibility of the developer to notify GeoPacific Consultants Ltd. when conditions or situations not outlined within this document are encountered.

8.0 CLOSURE

We are pleased to assist you with this project. Please provide structural loads once they are available so that we can provide the appropriate preload design.

Please contact us if you require any clarification or additional details.

For:
GeoPacific Consultants Ltd.

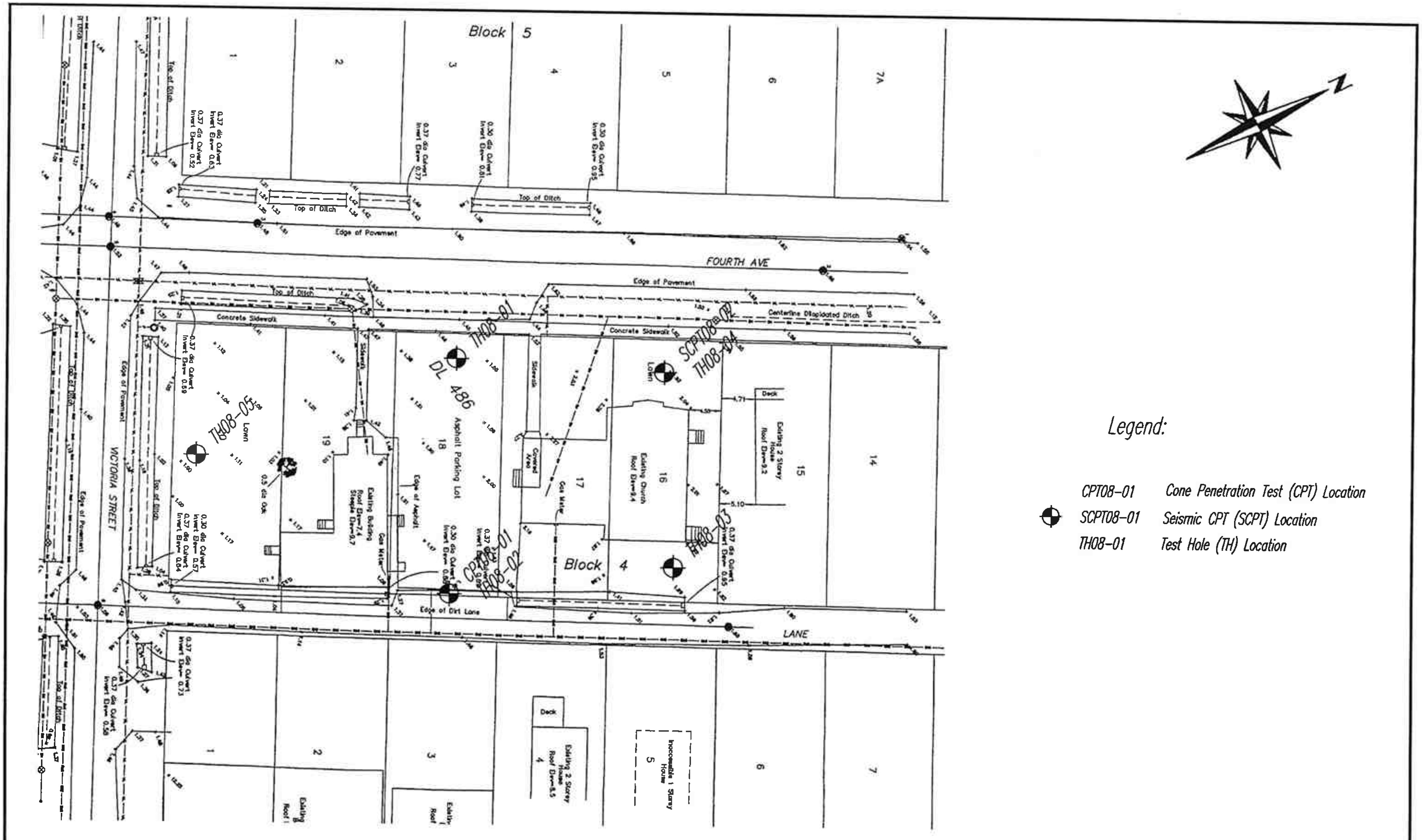
PROFESSIONAL
PROVINCE OF
S. M. FOFONOFF
30836
BRITISH COLUMBIA
ENGINEER

OCT 06 2008

Steven Fofonoff, P.Eng.
Project Engineer

Reviewed By:

Matt Kokan, M.A.Sc., P.Eng.
Principal



REFERENCE:

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GeoPacific Consultants Ltd.

DATE: September 10, 2008
DRN. BY: AM
APP'D.:
SCALE: NOT TO SCALE

Proposed Mixed Use Development
38014 Fourth Avenue, Squamish, BC
Testhole Plan

FILE NO.: 7806
DWG. NO.: 7806-01

REVISIONS:
REV.A
REV.B
REV.C

APPENDIX A - TEST HOLE LOGS

Test Hole Log: TH08-01

File: 7806

Project: Proposed Mixed-Use Development

Client: Cascadia Consulting

Site Location: 38014 Fourth Avenue, Squamish, BC

GeoPacific Consultants Ltd.

215 - 1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
Tel: 604-439-0922 Fax:604-439-9189

INFERRED PROFILE

Depth (ft)	SOIL DESCRIPTION	Depth/Elev (ft)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
0	Ground Surface					
0–0.8	***Asphalt*** 2", distressed	0.0				
0.8–	***Fill*** Compact, dark brown, sandy FILL	0.8	52.7			
	Topsoil					
	Silt Firm SILT, grey, trace/some sand, low plasticity, trace organics. @4' soft, higher organic constituent @4-6' organic SILT @6' firm/stiff, sandier @7' a 4" coarse sand seam noted.		68.4		▼ (4')	4' estimated phreatic level measured from the collar.
7–10	***Silt*** Firm/stiff, sandy, grey.	7.0	31.0			
10–25	***Sand*** Compact, grey, fine to medium grained SAND, wet, poor recovery. @15' trace wood debris noted @18' compact, medium grained.	10.0				
25	End of Borehole	25.0				

Logged: AM

Method: Solid Stem Auger

Date: August 27, 2008

Datum: Ground Elevation

Figure Number: A.1

Page: 1 of 1

Test Hole Log: TH08-02 (CPT08-01)

File: 7806
Project: Proposed Mixed-Use Development
Client: Cascadia Consulting
Site Location: 38014 Fourth Avenue, Squamish, BC

GeoPacific Consultants Ltd.

215 - 1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
Tel: 604-439-0922 Fax:604-439-9189

INFERRED PROFILE

Depth (ft)	SOIL DESCRIPTION	Depth/Elev (ft)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
0	Ground Surface					
0.0–1.5	***Asphalt*** 3", distressed	0.0				
1.5–8.0	***Fill*** Compact, gravelly sand FILL, trace silt, brown. ***Silt*** Stiff SILT, trace/some sand, olive grey. @3.5' firm @4.5' soft/firm, saturated @5-7' organic SILT, trace sand, grey @8' trace thin sand lenses, firm	1.5	35.5; 44.5; 50.8		▼ (4.5')	4.5' estimated phreatic level
8.0–12.5	***Silt*** Firm/stiff sandy SILT, interbedded with silty sand, brown mottled with trace grey.	8.0	30.5			
12.5–20.0	***Sand*** Compact, grey, fine grained. @15' medium grained SAND.	12.5				
20.0	End of Borehole	20.0				

Logged: AM
Method: Solid Stem Auger
Date: August 27, 2008

Datum: Ground Elevation
Figure Number: A.2
Page: 1 of 1

Test Hole Log: TH08-03

File: 7806
Project: Proposed Mixed-Use Development
Client: Cascadia Consulting
Site Location: 38014 Fourth Avenue, Squamish, BC

GeoPacific
Consultants Ltd.

215 - 1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
Tel: 604-439-0922 Fax:604-439-9189

INFERRED PROFILE: Soil Description	Depth/Elev (ft)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
Ground Surface					
Topsoil	0.0				
Fill Dense/compact sand FILL, trace gravels, brown, dry.	2.0				
Silt Firm/stiff SILT, trace to some sand, brown mottled with grey, trace rust staining. @3.5' grey, trace moisture @5' firm, grey brown. @5.5' firm/soft @6' firm/stiff, higher sand content @7.5-8.5' firm/soft @8.5' firm @10-10.5' grey brown, trace organics		45.7 38.4 42.3 44.7		▼ (4')	4' measured phreatic level
Sand Loose/compact fine to silty SAND.	10.5				
Sand Compact/dense, medium grained SAND, grey, saturated. @18' trace silt lenses interbedded.	12.0				
End of Borehole	20.0				

Logged: AM
Method: Solid Stem Auger
Date: August 27, 2008

Datum: Ground Elevation
Figure Number: A.3

Test Hole Log: TH08-04 (SCPT08-02)

File: 7806
Project: Proposed Mixed-Use Development
Client: Cascadia Consulting
Site Location: 38014 Fourth Avenue, Squamish, BC

GeoPacific
Consultants Ltd.

215 - 1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
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INFERRED PROFILE

Depth (ft)	Soil Description	Depth/Elev (ft)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
0	Ground Surface					
0–0.5	***Topsoil***	0.0				
0.5–2	***Fill*** Dense/compact, brown, gravelly sand FILL, trace pebbles.					
2–8	***Silt*** Stiff SILT, trace sand, brown, trace organics, low plasticity. @4' firm, more saturated @5-7' some sand, trace/some organics, firm/stiff, grey/brown. @7' higher sand content, stiff, trace organics (<1%), grey	2.0	38.2; 52.1		▼ (4')	4' estimated phreatic level. 4' Atterberg result: PL= 24.8; Moisture=38.2%; LL= 34.9; PI=10.1. Soil classification=CL
8–9.5	***Silt*** Stiff, sandy SILT, grey, uniform.	8.0	31.2			
9.5–12	***Sand*** Loose/compact, silty SAND to very fine grained SAND, olive grey, wet.	9.5				
12–20	***Sand*** Compact, fine to medium to coarse grained, trace gravel, poorly sorted, wet, grey. @15' more well sorted, medium grained SAND.	12.0				
20	End of Borehole	20.0				

Logged: AM
Method: Solid Stem Auger
Date: August 27, 2008

Datum: Ground Elevation
Figure Number: A.4
Page: 1 of 1

Test Hole Log: TH08-05

File: 7806
Project: Proposed Mixed-Use Development
Client: Cascadia Consulting
Site Location: 38014 Fourth Avenue, Squamish, BC

GeoPacific
Consultants Ltd.

215 - 1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
Tel: 604-439-0922 Fax:604-439-9189

INFERRED PROFILE

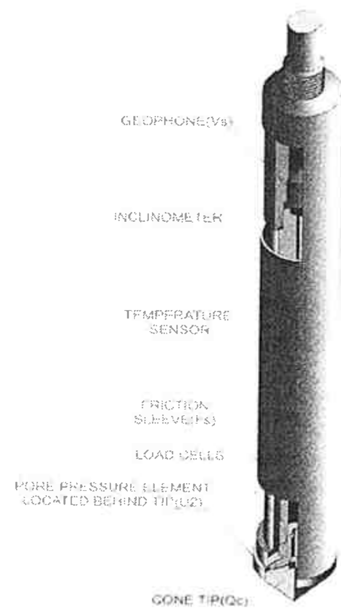
Depth (ft)	Symbol	SOIL DESCRIPTION	Depth/Elev (ft)	Moisture Content (%)	DCPT (blows per foot) 10 20 30 40	Groundwater / Well	Remarks
0		Ground Surface					
0		***Topsoil***	0.0				
		Silt Firm, brown, organic @1' soft, higher organic content, saturated. @3.5' trace grey. @5.5' sand lens (4") noted		66.5 63.9		▼	4' estimated phreatic level measured from the collar.
6		***Silt*** Stiff SILT, some sand, uniform, grey to medium dark	6.0	27.0			7' Sieve analysis passing 200=86.3%
10		***Sand*** Compact, coarse to medium SAND, poorly sorted, grey. @14' trace wood @15.5' trace silt lenses @16 a 2" peat lens. @16.5' more well sorted sand, coarse @18 fine to medium grained.	10.0				
20		End of Borehole	20.0				

Logged: AM
Method: Solid Stem Auger
Date: August 27, 2008

Datum: Ground Elevation
Figure Number: A.5

APPENDIX B - ELECTRONIC CONE PENETRATION RESULTS

Electronic Cone Penetrometer

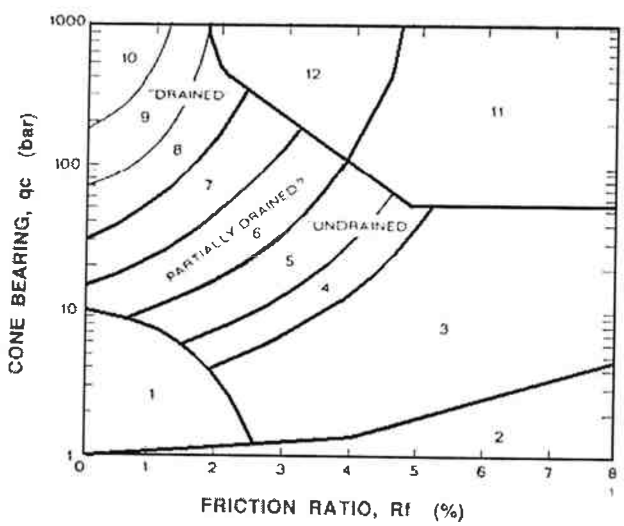


The system used is owned and operated by GeoPacific Consultants Ltd. and employs a 35.7 mm diameter cone that records tip resistance, sleeve friction, dynamic pore pressure, inclination and temperature at 5 cm intervals on a digital computer system. The system is a Hogentogler electronic cone system 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



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

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 above. 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.

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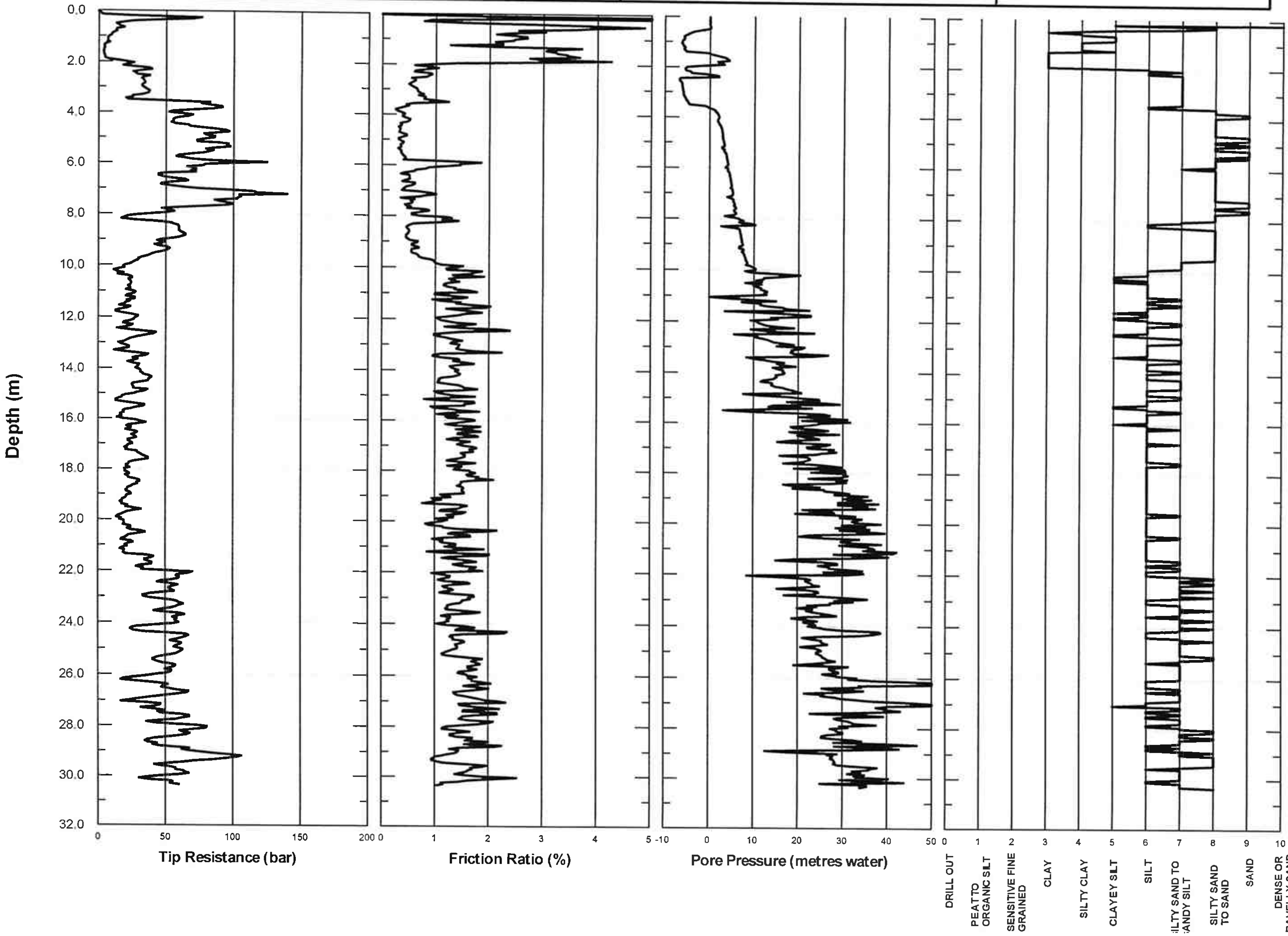
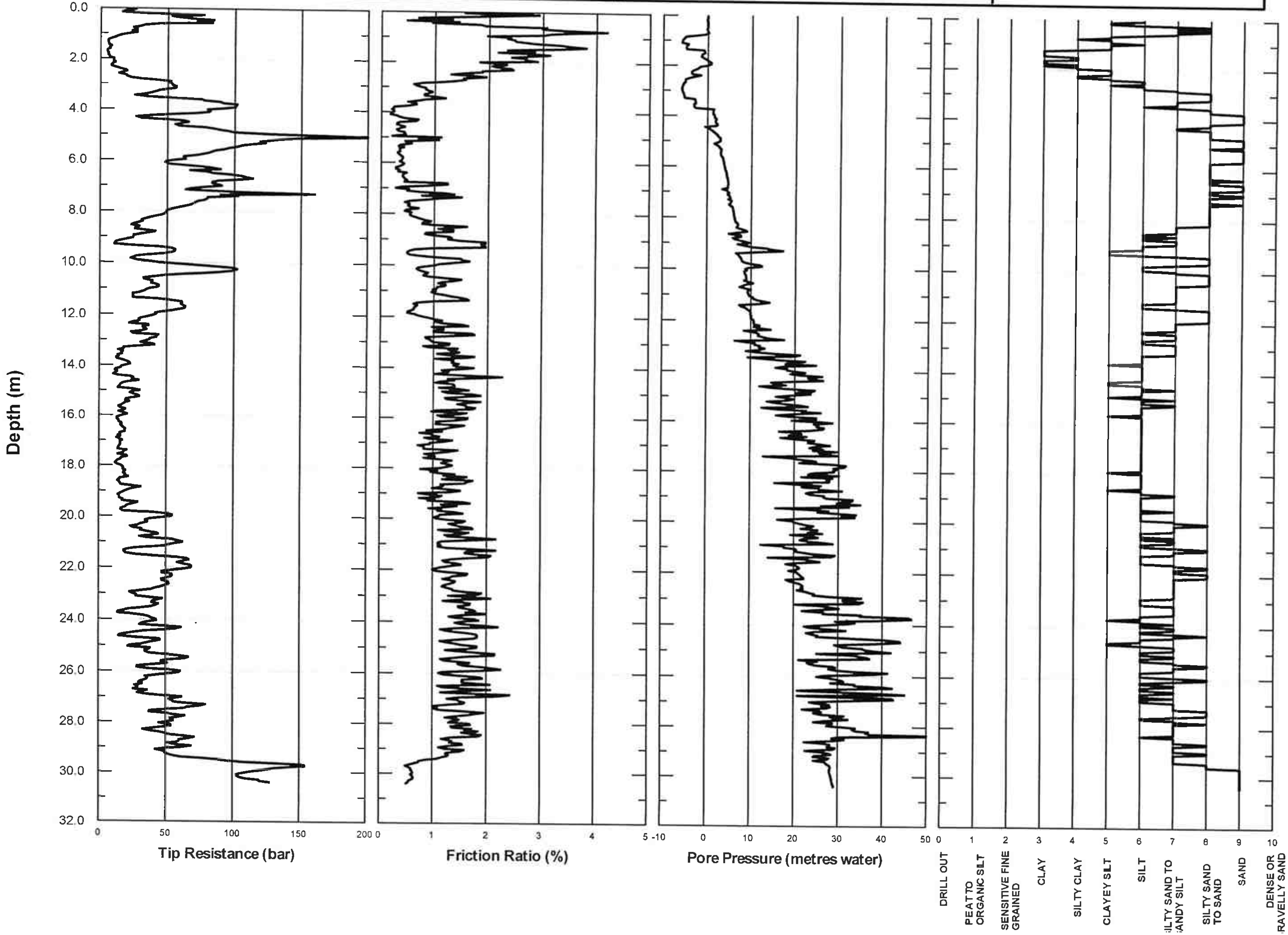
1 Robertson, P.K., 1990, "Soil Classification using the cone penetration test", 1990 Canadian Geotechnical Colloquium, Canadian Geotechnical Journal, Vol. 27, No. 1, 1990

GeoPacific Consultants Ltd.			File: 7806	CPT08-01
August 27, 2008	Cascadia Consulting	38014 Fourth Avenue, Squamish, BC Suamish United Church	Figure: B.01	Page 1 of 1



GeoPacific Consultants Ltd.			File: 7806	SCPT08-02
August 27, 2008	Cascadia Consulting	38014 Fourth Avenue, Squamish, BC Suamish United Church	Figure: B.02	Page 1 of 1



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 CONEVIEW developed by GeoPacific Consultants Ltd. 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_u = 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 CONEVIEW developed by GeoPacific Consultants Ltd. The results are presented on the Figures following.

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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

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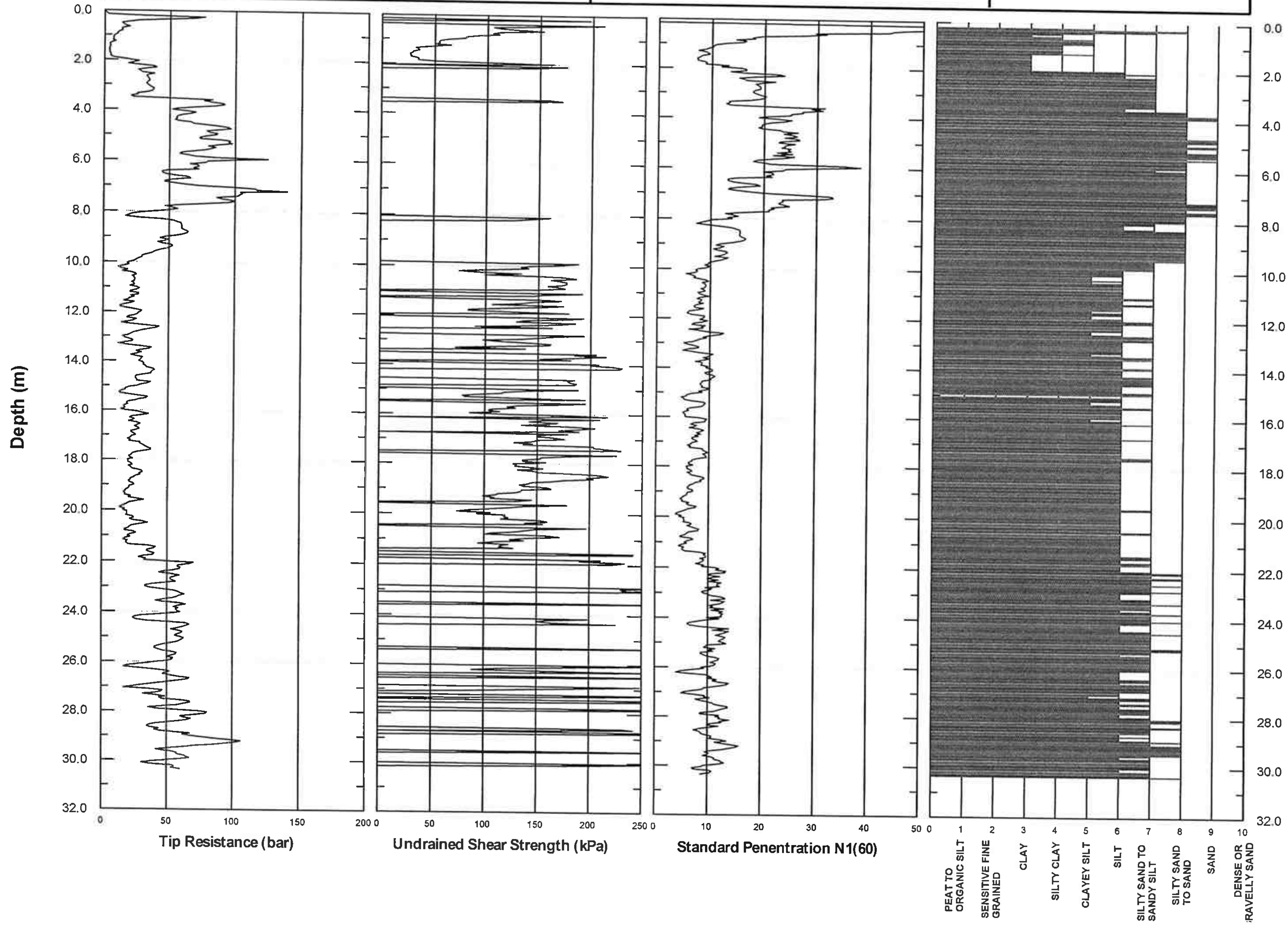
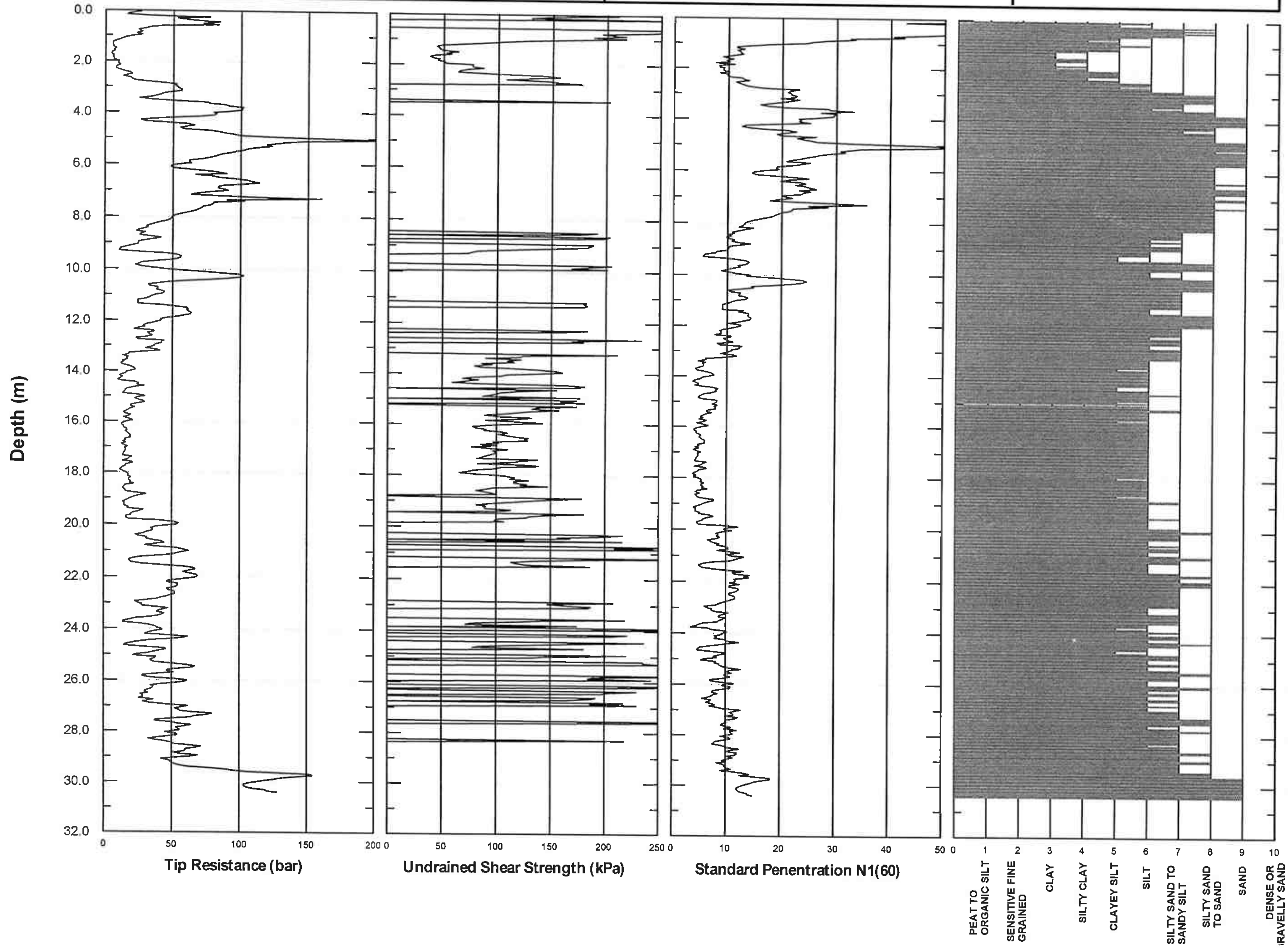


GeoPacific Consultants Ltd.			File: 7806	SCPT08-02
August 27, 2008	Cascadia Consulting	38014 Fourth Avenue, Squamish, BC Squamish United Church	Figure: C.02	Page 1 of 1



APPENDIX D - LIQUEFACTION ASSESSMENT

Assessment of the liquefaction potential of the ground has been determined at the electronic cone penetration test holes. 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 Seed (1984)[3] that determines the CRR from the normalized Standard Penetration Test N1(60) value and the percentage silt content. The value of N1(60) is calculated as described in Appendix C and the silt content is determined from the soil type as described in Appendix B. The value of the cyclic stress ratio is in accordance with standard methods and uses an earthquake magnitude of 7.0 and a design horizontal acceleration of 0.33g and seismic reduction factor characteristic of the Fraser Delta area.

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)[4] 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 (N1(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 labeled as Settlement.

3 Seed, H.B., K. Tokimatsu, L.F. Harder, and R.M. Chung, 1984, "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations", Report No. UBC/EERC-84/15, Earthquake Engineering Research Center, University of California, Berkeley.

4
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.

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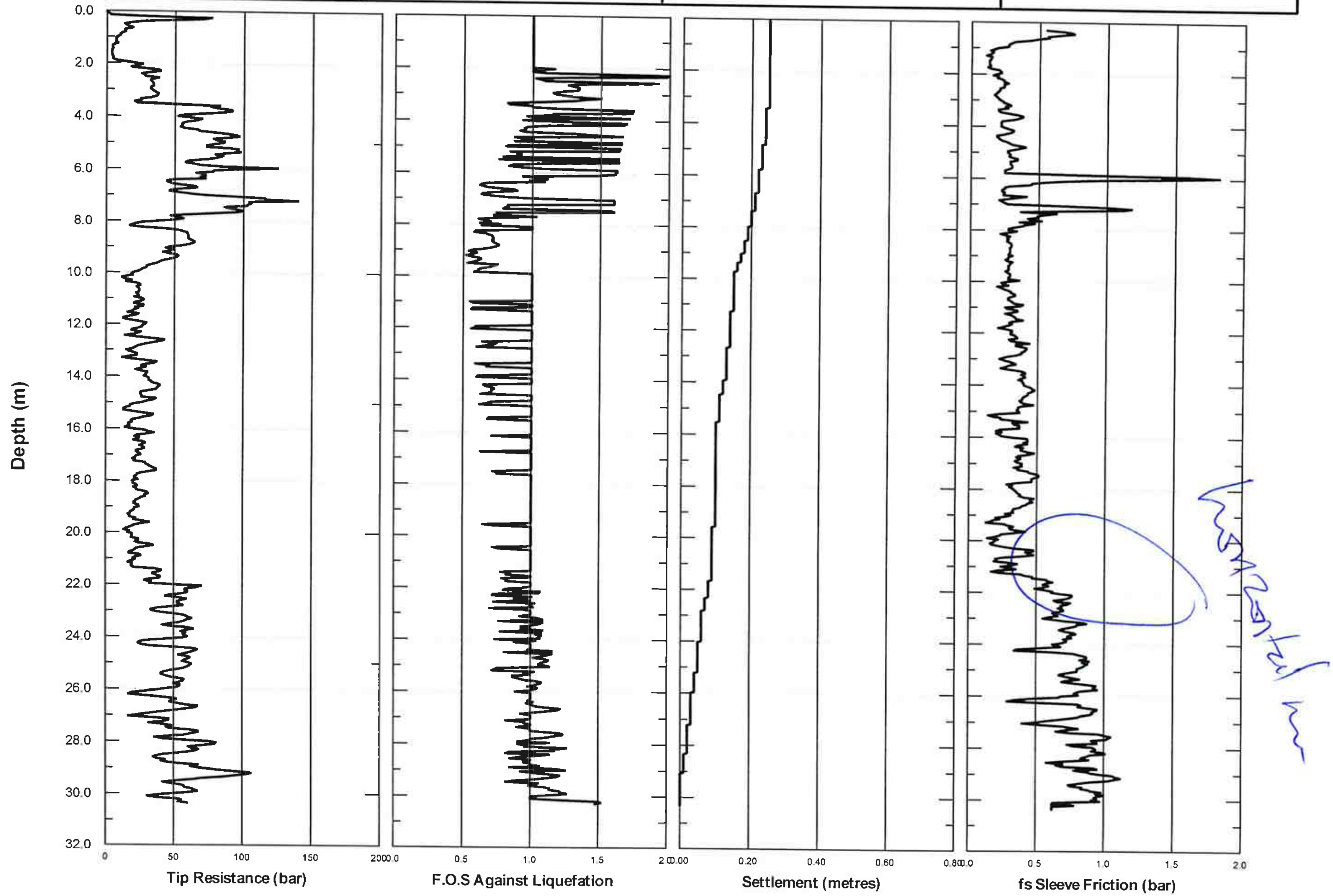

Interpretation processed with PGA = 0.33g

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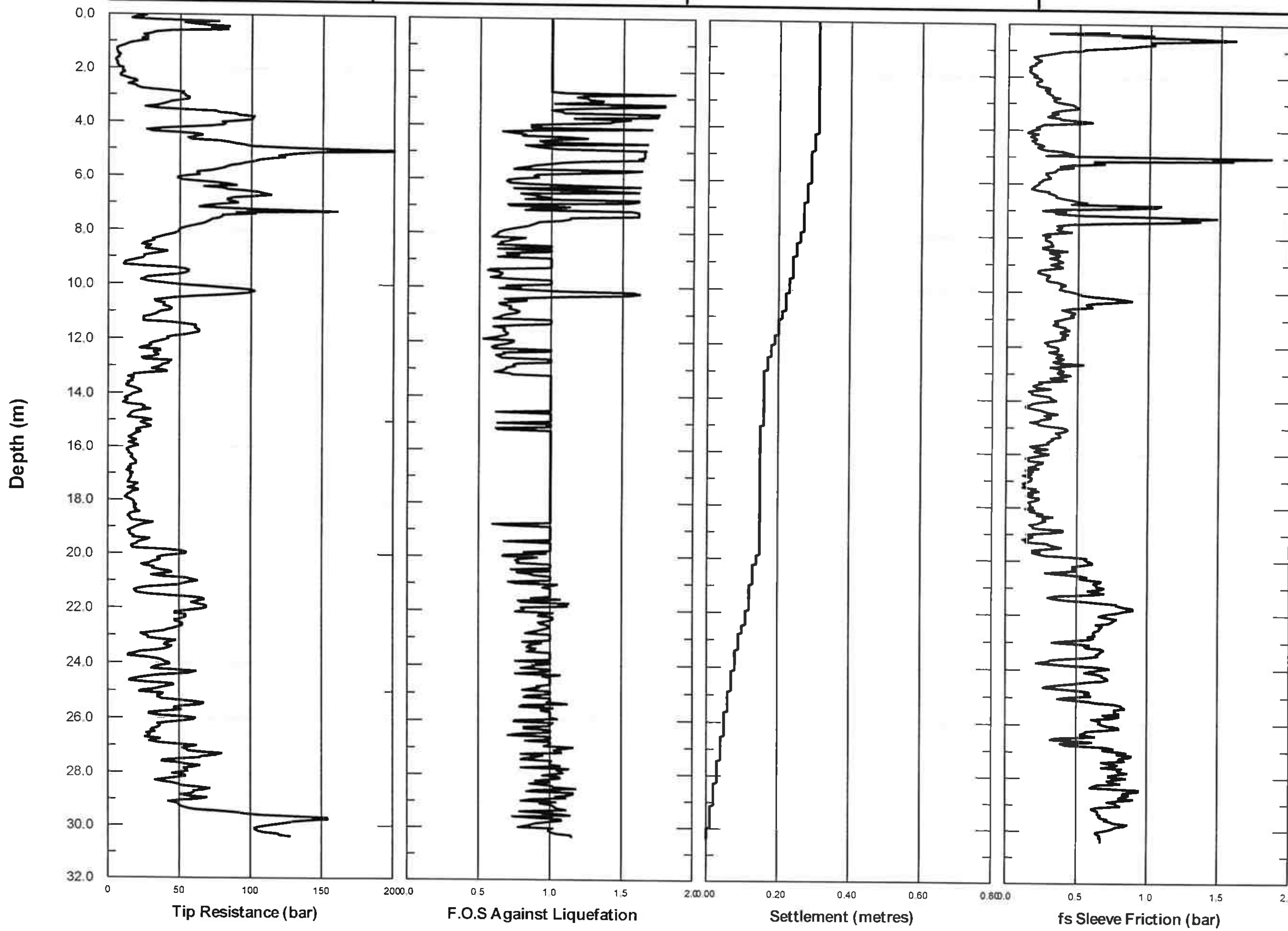

Interpretation processed with PGA = 0.33g

APPENDIX E – SHEAR WAVE VELOCITY TESTING

GeoPacific
Consultants Ltd.

File: 7806
Project: Proposed Mixed-Use Development
Client: Cascadia Consulting
Location: 38014 Fourth Avenue, Squamish, BC
Sounding: SCPT08-02
Date: 27-Sep-08

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

Shear Wave Velocity Data (Vs)

Depth (m)	Depth (ft)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Time Difference (ms)	Shear Wave Velocity (m/s)	Midpoint (m)
1.40	4.59	1.20	1.26				
2.40	7.87	2.20	2.24	0.97	9.97	97	1.70
3.40	11.15	3.20	3.22	0.99	8.75	113	2.70
4.40	14.43	4.20	4.22	0.99	6.56	152	3.70
5.40	17.71	5.20	5.22	1.00	5.12	195	4.70
6.40	20.99	6.20	6.21	1.00	4.58	218	5.70
7.40	24.27	7.20	7.21	1.00	5.11	195	6.70
8.40	27.55	8.20	8.21	1.00	6.31	158	7.70
9.40	30.83	9.20	9.21	1.00	6.78	147	8.70
10.40	34.11	10.20	10.21	1.00	6.80	147	9.70
11.40	37.39	11.20	11.21	1.00	6.05	165	10.70
12.40	40.67	12.20	12.21	1.00	6.05	165	11.70
13.40	43.95	13.20	13.21	1.00	6.64	151	12.70
14.40	47.23	14.20	14.21	1.00	6.26	160	13.70
15.40	50.51	15.20	15.21	1.00	6.07	165	14.70
16.40	53.79	16.20	16.20	1.00	6.04	166	15.70
17.40	57.07	17.20	17.20	1.00	5.86	171	16.70
18.40	60.35	18.20	18.20	1.00	5.94	168	17.70
19.40	63.63	19.20	19.20	1.00	5.63	178	18.70
20.40	66.91	20.20	20.20	1.00	5.33	188	19.70
21.40	70.19	21.20	21.20	1.00	5.24	191	20.70
22.40	73.47	22.20	22.20	1.00	5.51	181	21.70
23.40	76.75	23.20	23.20	1.00	5.29	189	22.70
24.40	80.03	24.20	24.20	1.00	5.19	193	23.70
25.40	83.31	25.20	25.20	1.00	5.00	200	24.70
26.40	86.59	26.20	26.20	1.00	5.12	195	25.70
27.40	89.87	27.20	27.20	1.00	4.92	203	26.70
28.40	93.15	28.20	28.20	1.00	4.98	201	27.70
29.40	96.43	29.20	29.20	1.00	4.94	202	28.70
30.40	99.71	30.20	30.20	1.00	4.80	208	29.70

File: 7806
Project: Proposed Mixed-Use Development
Client: Cascadia Consulting
Site: 38014 Fourth Avenue, Squamish, BC
Sounding: SCPT08-02