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Marzara Ventures Ltd.
#3 – 1680 Lloyd Avenue
North Vancouver, BC
V7P 2N6

April 1, 2022
File: 13958

Attention: Ghol Marzara

Re: Geotechnical Investigation Report – Commercial/Residential Building
2101 Clarke Street, Port Moody, BC

1.0 INTRODUCTION

We understand that it is proposed to construct a new mixed use residential and commercial development at the above referenced property. Architectural drawings prepared by Lo Studio Architecture dated December 6, 2021 show the new development consisting of up to six levels of above grade construction underlain by two levels of below grade parking. The lowest parkade level elevation is shown at 49 feet geodetic which equates to 2 levels buried where the site is topographically highest fronting the Barnet Highway.

This report presents the results of our field investigation and provides geotechnical recommendations for the design and construction of the new development proposed. This report was prepared exclusively for our client, for their use and the use of others on their design and construction team for this project.

As the development is still in the conceptual stage of design, the recommendations provided herein should be considered preliminary and subject to update once the design is further refined.

2.0 SITE DESCRIPTION

The site is bounded by Clarke Street to the north, Barnet Highway to the west, St Johns Street to the south and residential properties to the east. Schoolhouse South Creek runs through the property from south to north. There is a boarded up house located at the north end of the property. The southern half of the property is heavily vegetated with very dense trees and bushes. The topographic information provided shows elevations in the vicinity of Barnett Highway at approximately 21.3 m sloping down gradually towards the north and east to elevations of approximately 15 m at Clarke Street and Schoolhouse South Creek.

The site location relative to the surrounding improvements is shown on our Drawing No. 13958-01 attached to this report.

3.0 FIELD INVESTIGATION

GeoPacific Consultants Ltd. conducted an investigation on May 31, 2016. A total of 4 solid stem auger test holes and 4 dynamic cone penetrometer test (DCPT) soundings were conducted using a track mounted drill rig supplied by On Track Drilling of Coquitlam, BC. The test holes were drilled to a maximum depth of 12.2 m below existing grades. The DCPT's were driven to refusal with a maximum depth of 9.2m below existing grade. Two cone penetration test (CPT) soundings were advanced to refusal depths of 3.75 and 5.70 metres below existing grades. All the test holes were sealed immediately in accordance with provincial abandonment requirement upon completion of logging and sampling. The site investigation was supervised and the soils encountered were logged and collected for laboratory analysis in the field by a geotechnical technician from our office.

The CPT is an in-situ testing device which is advanced into the ground 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 cm increments. The data obtained may be correlated to estimate engineering parameters such as shear strength, relative density, soil behaviour type, and consolidation coefficients.

A set of test hole logs including laboratory test results are presented in Appendix A. The results of the CPT soundings are presented in Appendix B. The interpreted soil parameters from the CPT data are presented in Appendix C. All depths are referenced from the existing ground surface at the test hole locations and elevations are approximated from Port Moody GIS contour maps.

The approximate locations of the test holes on the property are shown on our Drawing 13958-01.

4.0 SOIL CONDITIONS

4.1 Soil Profile

According to "Surficial Geology – New Westminster (MAP 1484A)" published by Geological Survey of Canada, this region is understood to be underlain by Capilano Sediments.

In general, the soil profile noted in our test hole locations, from the surface downwards, consists of a thin layer of topsoil, overlying up to 2.9 metres of compact sand and gravel fill, overlying up to 4.1 metres of very loose to dense native sand and gravel, overlying dense to very dense glacial till to the maximum depth of exploration. At TH16-04 a layer of soft organic silt was encountered between the fill and native sands with a thickness of 0.6m.

For a detailed description of the soil conditions encountered at the specific test hole locations, please refer to Appendix A attached to this report. All depths are referenced from the existing ground surface at the test hole locations and levels are approximated from Port Moody GIS contour maps.

4.2 Groundwater Conditions

The static water table has been estimated based on the observed consistency and oxidation of the soils within the boreholes, which indicate a water table depth of 2.3 to 3.4 m below current grades. The position of the water table should be expected to vary seasonally with higher levels during wetter periods of the year.

Based on the gradation of the soils observed above the glacial till, we expect excavations extending below the water table to experience heavy seepage at this site.

5.0 DISCUSSION

5.1 General

The proposed mixed use buildings may consist of 6 storeys of above grade construction over up to 2 levels of below grade reinforced concrete construction built into the existing slope. We expect loading induced by the proposed apartment buildings will be moderately heavy. Floor loading is expected to be approximately 5 kPa.

To accommodate the proposed underground structures, we anticipate temporary excavations of up to 7 m. We expect that a shoring system which includes a groundwater cut off will be required. Conventional shoring with dewatering is not considered feasible based on the observed soil conditions and relatively high groundwater table.

Based on a floor slab elevation, we envision that very dense glacial till would likely be encountered at the west are of the site which grades into very loose to very dense sand to sand and gravel towards the northeast. Conventional strip and pad foundations may be employed for foundations on very dense glacial till. Due to the variable relative density of the sand to sand and gravel above the glacial till it is not suitable to support the contemplated structure on conventional foundations. Where these soils are encountered we envisage that pile foundations or ground improvement such as jet grouting, for example, to reinforce the loose soils may be employed to support the structure. Deepening the parkade at the east end of the structure may be considered to reduce the piling/soil reinforcement requirements. A detailed site preparation design may be prepared once the development design is further refined.

We confirm from a geotechnical point of view, that the proposed development is feasible, subject to the incorporation of our recommendations.

5.2 Seismic Analysis

The condition of the sand to sand and gravel overlying the glacial till deposits is highly variable ranging from very loose to very dense. The very loose to compact portions of the deposit are considered potentially liquefiable during the design earthquake defined in the 2018 British Columbia Building Code.

The consequence of liquefactions include permanent vertical and horizontal movements, and potential punching/bearing capacity failure of foundations where the loads exceed the strength of the liquefied soil below. The incorporation of pile foundations or reinforcement of these soils with jet grout, for example, and noted in Section 5.1, would be required to support the structure on these materials

6.0 RECOMMENDATIONS

6.1 Conventional Strip & Pad Foundations

6.1.1 Site Preparation

Prior to construction of conventional foundations and grade supported slabs, the site must be stripped to expose very dense glacial till or very loose to very dense sand to sand and gravel. We expect that the proposed parkade elevations will dictate most excavation depths rather than the soil conditions.

The sand to sand and gravel is not suitable to support the proposed structure on conventional strip and pads. These soils may be reinforced with the use of jet grout, for example, to allow for the use of strips and pads, or pile foundations discussed in Section 6.2 may be employed.

The glacial till deposits are typically sensitive to moisture. The site should be graded to inhibit ponding of water and the exposed subgrade should be blinded with 50 mm of 10 MPa concrete for protection. Where vehicles or equipment must traverse the site we recommend the use of at least 300 mm of engineered fill as defined below as a running surface.

Grade may be reinstated for foundations and slabs with engineered fill. "Engineered Fill" is generally defined as ***clean sand to sand and gravel*** *containing silt and clay less than 5% by weight*, compacted in 300 mm loose lifts to a minimum of 98% of the ASTM D698 (Standard Proctor) maximum dry density at a moisture content that is within 2% of optimum for compaction.

GeoPacific is to review stripping and compaction of engineered fill.

6.1.2 Recommended Bearing Pressures

We recommend that the foundations supported on very dense glacial till or jet grouted reinforced sand to sand and gravel may be designed using a serviceability limit state (SLS) bearing pressure of 500 kPa, and a factored ultimate limit state (ULS) bearing pressure of 750 kPa.

We expect that the settlement of foundations designed as recommended above should be within the normally acceptable limits of 25 mm maximum and up to about 20 mm differential over a 10 m span.

Irrespective of bearing pressures, foundations should not be less than 450 mm in width for strip foundations and not less than 600 mm in width for square or rectangular foundations. Foundations should also be buried a minimum of 450 mm below the surface for frost protection.

All foundation subgrade must be reviewed by GeoPacific prior to foundation construction.

6.2 Pile Foundations

6.2.1 Site Preparation

No special site preparation measures are required in advance of the installation of pile foundations.

6.2.2 Axial Pile Capacity

There are a number of piling options likely to be feasible at this site including, for example, steel pipe piles, ICP spun concrete piles, and steel screw piles.

We expect that piles driven to effective refusal within the very dense glacial till deposits can likely be designed to the safe structural capacity of the piles. Penetration of 1 to 2 m into the till may be required before refusal is achieved. The pile types and capacities must be confirmed during detailed design with the structural engineer. However, for example, a 400 mm steel pipe pile with a wall thickness of 12.7 mm and 250 MPa (36 ksi) strength steel may be designed on the basis of a factored ultimate axial capacity of 1800 kN (200 tons). For preliminary design, we recommend a service axial capacity of 1300 kN (150 tons).

We recommend as part of the piling program that Pile Driver Analyzer (PDA) testing be completed on at least 3 test piles. The PDA system measures the compressive wave imposed in the pile by the piling hammer. Processing of the data yields predicted capacities and movements. It has been our general experience that the PDA may be employed to more accurately assess the capacity of the piles than conventional empirical approaches. The piles are typically tested immediately at the end of driving and then after several days which allows for water pressures around the piles to dissipate, which normally results in an improvement in pile capacity. We suggest that re-striking of the piles occur 14 days after initial driving.

Piles should be separated by a minimum distance of 3 pile diameters to minimize group effects.

In accordance with the requirements of the 2018 BC Building Code, full-time monitoring of the installation of pile foundations is required by the geotechnical engineer.

6.2.3 Lateral Pile Capacity

Piles at this site are anticipated to be driven to deeper depths at the northeast end of the building. The lateral capacity of the piles is a function of their length. In general, shorter piles provide stiffer foundation elements and will deflect less than a longer pile that is exposed to the same load. Detailed lateral pile analysis in consultation with the structural engineer will be required to confirm lateral capacities and anticipated structural movements.

Limited lateral pile capacity may be required if the lateral loads can be resisted by the basement walls by relying on the passive resistance of the backfill material.

6.3 Temporary Excavation and Shoring

We expect that temporary excavations would be sloped where possible since it is typically more economical to do so. We would expect that slopes cut at a maximum gradient of 1:1 (H:V) can be constructed in the surficial fill and sand to sand and gravel above the groundwater table.

Due to heavy seepage that would emanate from the sand to sand and gravels above the glacial till below the static groundwater table, a groundwater cut off will need to be incorporated into the shoring design. Typical methods to form a groundwater cut off in these conditions typically consist of jet grout columns or concrete secant piles installed in an overlapping pattern to create a sealed wall surrounding the site. Reinforcement of the groundwater cut off wall is normally required with steel I-beams and reinforced concrete walers. The entire wall is supported with pre-tensioned grouted tie back anchors drilled into the soils extending beyond the building excavation. Drilling will occur through cohesionless granular deposits which will require the use of hollow core "IBO" anchors which are continuously grouted during installation.

The groundwater cut off will be permanent in order to allow for construction of conventional foundations and a perimeter drainage system. In order to be permanent the groundwater cut off must be contained on the development property. We recommend that the architect provide at least 450 mm of space between the property line and the foundation wall for the groundwater cut off. If this cannot be accommodated then the cut off must be assumed to be temporary and subject to possible future removal. In this case the structure would have to be designed with a "tanked" foundation consisting of a waterproofed raft and foundation walls.

Irrespective of the use of a groundwater cut off, some seepage into the excavation will occur, particularly at anchor head locations. We expect that excavation inflows would be handled with conventional sumps and sump pumps.

Excavation induced movements are possible due to the contemplated depth of the cut. These movements are typically generated by elastic relaxation of the soil as the weight of material is removed from the site. We envision movements at the top of the excavation at the west end of the site to be approximately 25 mm or less with negligible movements 5 m back of the excavation.

All excavations exceeding 1.2 m in depth requiring worker entry must be reviewed by a professional geotechnical engineer in advance of the work.

GeoPacific may prepare an excavation design upon request when the architectural and structural design is further refined.

6.4 Seismic Design of Foundations

Due to the potential for liquefaction caused by the design earthquake, the subgrade conditions underlying this site are classified as Site Class F as defined in Table 4.1.8.4.A of the 2018 British Columbia Building Code. Class "F" properties require that a site specific dynamic analysis be completed unless the natural period of the structure is less than 0.5 seconds. For this case, the site class may be assumed to be Site Class E. The structural designer will have to confirm the natural period of the structure. GeoPacific may undertake a site specific dynamic analysis upon request.

The impacts of liquefaction are negated in the site class assessment if the soils supporting the foundations consist entirely of glacial till or improved soils, as discussed in Section 5.1. For this case, Class "C" may be employed.

6.5 Slab-on-Grade Floors

The floor slabs should be underlain by a minimum of 150 mm thick of 19 mm clear crushed gravel fill to inhibit upward migration of moisture beneath the slab. The crushed gravel fill should be compacted to a minimum of 98% of the ASTM D698 (Standard Proctor) maximum dry density at a moisture content that is within 2% of optimum for compaction.

Compaction of the slab-on-grade fill must be reviewed by GeoPacific.

6.6 Foundation Drainage Systems

Provided that a groundwater cut off is incorporated into the excavation design, a conventional perimeter drainage system may be employed. For initial sizing of the system, we recommend the mechanical designer assume 250 L/min. Groundwater inflows should be confirmed at the time of excavation by the mechanical designer.

6.7 Earth Pressures on Foundation Walls

We recommend that the basement walls should be designed for static and seismic earth pressures. Hydrostatic pressures on the outside of the groundwater cut off wall will also be transferred through the shoring to the structure, and these forces must be permanently retained as well. Earth pressures depend on many factors including the rigidity of the wall, excavation procedure, type of backfill, and the level of compaction.

We envision that the excavation would be completed with a single sided forming system and synthetic flat drain with no working space. This method is considered preferential for this specific site provided that the groundwater cut-off can be installed on the development property. For this case the stiffness of the groundwater cut-off can be used for the benefit of the structure, as described further below.

Active earth pressure conditions are expected to develop due to the gradual elastic relaxation of the shoring and retained soils as the excavation proceeds. The loading provided below is also based on this assumption.

We recommend that the walls are designed for a static earth pressures of 5.4H kPa triangular above the water table, and increasing below the water table at 13H kPa triangular, where H is the height of the retained soil in metres. For design purposes, we recommend that the water table be assumed to be 2 m below current grades. Seismic earth pressures of 3.0H kPa inverted triangular may also be assumed. The loading is based on unfactored soil parameters and so the loading should be assumed to be unfactored as well.

Provided that the groundwater cut off is on the development property, and remains in place after construction, the structural designer may assume that the above referenced loading is transferred directly to the suspended floor slabs in the parkade rather than evenly distributed through the foundation wall. This is possible due to the stiffness and strength of the steel reinforced groundwater cut-off.

6.8 Utility Design and Installation

Site utilities are anticipated to be required beneath the slabs-on-grade. The design of these systems must consider the locations and elevations of the foundations. The service trenches and excavations required for the installation of the underground pipes, vaults and/or manholes should be located outside of a 1.5:1 (H:V) slope measured downward from the edge of adjacent foundations.

All excavations and trenches must conform to the latest Occupational Health and Safety Regulation supplied by the Worker Compensation Board of British Columbia.

Excavation in excess of 1.2 m in depth requiring worker-entry must be reviewed by a geotechnical engineer.

6.9 Methane Potential

No organic soils were identified in our test holes below the foundation level therefore a methane control system is not required for this project.

7.0 DESIGN REVIEWS AND CONSTRUCTION INSPECTIONS

The preceding sections provide preliminary recommendations for the design and construction of the proposed development. We have recommended the review of certain aspects of the design and construction. It is important that these reviews are carried out to ensure that our intentions have been adequately communicated. It is also important that any contractors working on the site review this document prior to commencing their work.

It is the responsibility of the contractors working on-site to inform GeoPacific a minimum of 48 hours in advance that a filed review is required. In summary, reviews are required by geotechnical engineer for the following portions of the work.

1. Stripping	Review of stripping depth
2. Excavation	Review of temporary slopes, and shoring installation/decommissioning
3. Engineered Fill	Review of materials and compaction degree
4. Foundation	Review of conventional and pile foundation installation
5. Slab on Grade	Review of foundation subgrade/ under slab fill materials and compaction
6. Backfill	Review of placement of backfill along foundation walls.

8.0 CLOSURE

We are pleased to be of assistance to you on this project and we trust that our comments and 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 contact us.

For:
GeoPacific Consultants Ltd.

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
P. M. MARTZ
52736

Permit to Practice EGBC
1000782

APR 0 4 2022

Patrick Martz, B.A.Sc., P.Eng.
Project Engineer

Reviewed by:

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

LEGEND:

- CPT#-# – CONE PENETRATION TEST (CPT) LOCATION
- TH#-# – TEST HOLE (TH) LOCATION

SITE PLAN

*TEST LOCATIONS ARE APPROXIMATE

REFERENCE:

GEOPACIFIC VANCOUVER KAMLOOPS CALGARY

DATE: 2016-May-31			COMMERCIAL/ RESIDENTIAL BUILDING 2101 CLARKE STREET, PORT MOODY, BC TEST HOLE SITE PLAN	FILE NO.: 13958 DWG. NO.: 13958-01	REVISIONS: A. B. C.
DRAWN BY: ED	APPROVED BY:	REVIEWED BY: JC			
SCALE: AS SHOWN					

ORIGINAL PAPER SIZE 8.5"x11"

APPENDIX A - TEST HOLE LOGS

Test Hole Log: TH16-01 (CPT16-01)

File: 13958
Project: COMMERCIAL/ RESIDENTIAL BUILDING
Client: MARZARA VENTURES LTD
Site Location: 2101 CLARKE STREET PORT MOODY BC

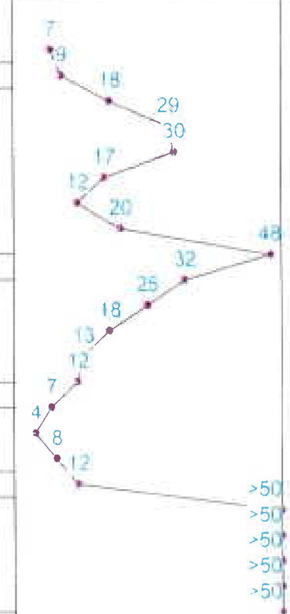
GEOPACIFIC CONSULTANTS
215 - 1200 West 73rd Avenue, Vancouver, BC. V6P 6G5
Tel: 604-439-0922 Fax 604-439-9189

INFERRED PROFILE

Depth (m)/Elev (m)	SOIL DESCRIPTION
20.0 / 0.0	Ground Surface
19.2 / 0.8	**Topsoil** bushes over SAND TOPSOIL, trace gravel and roots, brown, dry
18.2 / 1.8	**Sand and gravel** compact SAND and GRAVEL fill, trace organics, trace fines, brown, slightly moist
	Sand and gravel compact SAND and GRAVEL fill, brown, slightly moist; some asphalt at 1.4m
15.7 / 4.3	**Sand** dense to compact SAND, light brown, slightly moist; gravelly SAND after 2.7m; moist after 3.0m, wet after 3.4m; trace gravel after 3.4m; fine grained SAND, trace fines, after 4.0m
15.3 / 4.7	
14.1 / 5.9	**Silty sand** compact silty SAND, trace gravel, grey, wet
	Silt very stiff SILT, light brown, moist to wet
11.8 / 8.2	**Silt sand and gravel** very dense SILT SAND and GRAVEL, light brown, wet
10.2 / 9.8	**Silty gravelly sand** very dense silty gravelly SAND, till like, grey, wet
7.8 / 12.2	**Silty sand and gravel** very dense silty SAND and GRAVEL, glacial till, grey, wet
	End of Borehole

Moisture Content (%): 41.8, 16.2, 22.2, 24.0, 19.5, 12.9, 10.8, 13.9

DCPT (blows per foot)



Groundwater / Well

Remarks

- elevation estimated based on Port Moody GIS contours
- 3.4m estimated water table depth
- DCPT refusal at 7.0m

Logged: ED
Method: Solid stem auger/CPT/DCPT
Date: 2016-May-31
Datum: Ground elevation
Figure Number: A 01
Page: 1 of 1

Test Hole Log: TH16-02

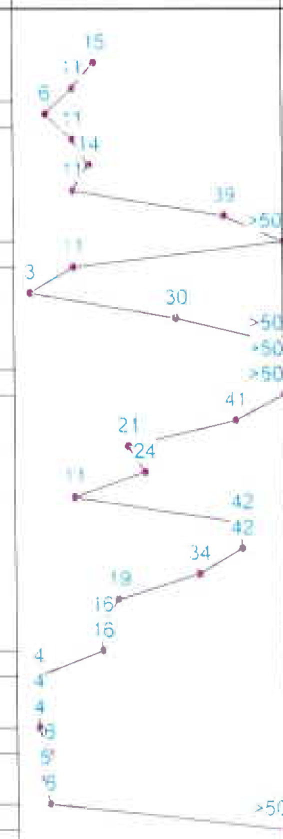
File: 13958
Project: COMMERCIAL/ RESIDENTIAL BUILDING
Client: MARZARA VENTURES LTD
Site Location: 2101 CLARKE STREET PORT MOODY BC

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

INFERRED PROFILE

Soil Description	Depth (m)/Elev (m)	Moisture Content (%)	Remarks
Ground Surface	17.5 / 0.0		elevation estimated based on Port Moody GIS contours
Topsoil compact gravelly SAND TOPSOIL, some rootlets, brown, dry		8.2	
Sand and gravel compact SAND and GRAVEL fill, trace cobbles, trace fines, brown, dry	16.0 / 1.5		
Gravelly sand compact gravelly SAND fill, some organics, brown, wet	15.7 / 1.8	18.7	
Silty sand and gravel dense silty SAND and GRAVEL fill, trace garbage, grey, slightly moist	15.2 / 2.3		2.3m estimated water table depth
Sand and gravel compact to dense SAND and GRAVEL, trace to some woody organics, brown, wet	13.8 / 3.7	16.2	
Sand and gravel dense medium to coarse grained SAND and GRAVEL, brown, wet; compact to dense, medium grained after 4.6m			
Silt very stiff sandy SILT to SILT, grey, wet; some sand and gravel after 7.9m	11.4 / 6.1	30.3	
Silt sand and gravel loose SILT SAND and GRAVEL, grey, wet	9.3 / 8.2	22.9	
Silty sand and gravel very dense cobbly silty SAND and GRAVEL, glacial till, brown, wet	8.4 / 9.1	9.9	DCPT refusal at 9.2m
End of Borehole	6.5 / 11.0		auger refusal at 11.0m

DCPT (blows per foot)



Logged: ED
Method: Solid stem auger/DCPT
Date: 2016-May-31

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

Test Hole Log: TH16-03

File: 13958
Project: COMMERCIAL/ RESIDENTIAL BUILDING
Client: MARZARA VENTURES LTD
Site Location: 2101 CLARKE STREET, PORT MOODY, BC

GEOPACIFIC CONSULTANTS

215 - 1200 West 73rd Avenue, Vancouver, BC, V6P 6G5
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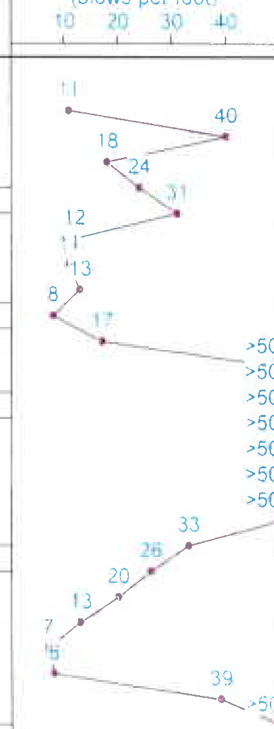
INFERRED PROFILE: SOIL DESCRIPTION	Depth (m)/Elev (m)	Moisture Content (%)	Groundwater / Well	Remarks
Ground Surface	16.0 / 0.0			elevation estimated based on Port Moody GIS contours
Cobbly sand and gravel compact to dense cobbly SAND and GRAVEL fill, brown, moist	14.2 / 1.8	18.4		
Gravelly sand compact gravelly SAND fill, brown moist wet after 2.6m	13.1 / 2.9	26.8	▼	2.6m estimated water table depth
Sand and gravel dense medium to coarse grained SAND and GRAVEL, washed out, brown, wet	11.1 / 4.9	14.2		
Sand dense to compact fine to medium grained SAND, trace fines, grey-brown, wet	9.0 / 7.0	26.4		
Silt sand and gravel dense to very dense SILT SAND and GRAVEL, light brown, wet silty SAND and GRAVEL glacial till after 8.5m cobbly after 9.1m	5.3 / 10.7	30.0 16.4		DCPT refusal at 7.6m
End of Borehole				

DCPT (blows per foot)



Logged: ED
Method: Solid stem auger DCPT
Date: 2016-May-31

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

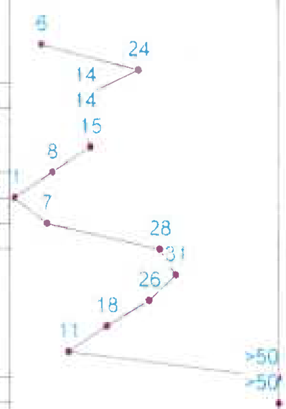
Test Hole Log: TH16-04 (CPT16-02)

File: 13958
Project: COMMERCIAL/ RESIDENTIAL BUILDING
Client: MARZARA VENTURES LTD
Site Location: 2101 CLARKE STREET, PORT MOODY, BC

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

INFERRED PROFILE: SOIL DESCRIPTION	Depth (m)/Elev (m)	Moisture Content (%)	Remarks
Ground Surface	15.0 / 0.0		elevation estimated based on Port Moody GIS contours
Topsoil compact SAND TOPSOIL, trace organics, dark brown, moist	14.5 / 0.5	11.6	
Sand compact SAND fill, light brown, moist; sand and gravel at 1.1m	13.3 / 1.7	81.1	
Wood and silt stiff to firm SILT and WOOD FIBRE, dark brown, moist	12.7 / 2.3	29.3	2.3m estimated water table depth
Sand compact fine to medium grained SAND, some fines, grey, wet	12.3 / 2.7		
Sand and gravel compact medium to coarse grained SAND and GRAVEL, grey, wet	10.9 / 4.1	16.8	DCPT refusal at 4.6m
Silty sand and gravel dense silty SAND and GRAVEL, till like, grey, wet; very dense after 4.6m; cobbly after 5.8m	8.9 / 6.1	14.0	auger refusal at 6.1m on cobble or boulder
End of Borehole			

DCPT (blows per foot)



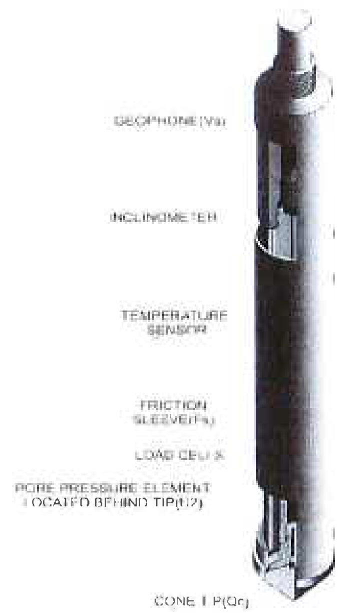
Logged: ED
Method: Solid stem auger/DCPT/CPT
Date: 2016-May-31

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

APPENDIX B - ELECTRONIC CONE PENETRATION RESULTS

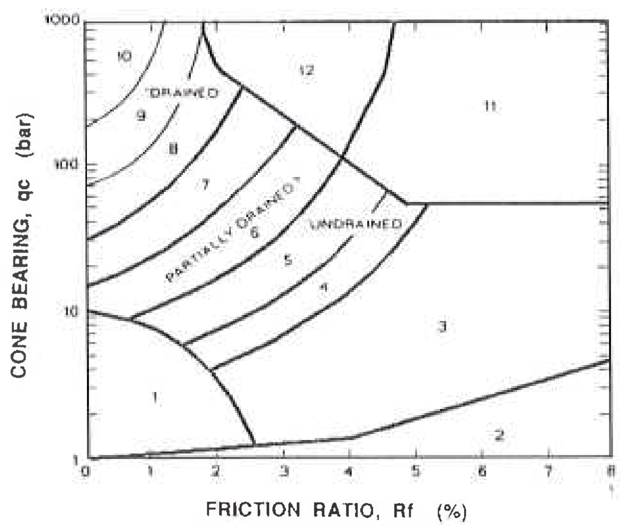
The system used is owned and operated by GeoPacific and employs a 35.7 mm diameter cone that records tip resistance, sleeve friction, dynamic pore pressure, inclination and temperature at 5 cm intervals on a digital computer system. The system is a Hogentogler electronic cone system and the cone used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

Electronic Cone Penetrometer



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

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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

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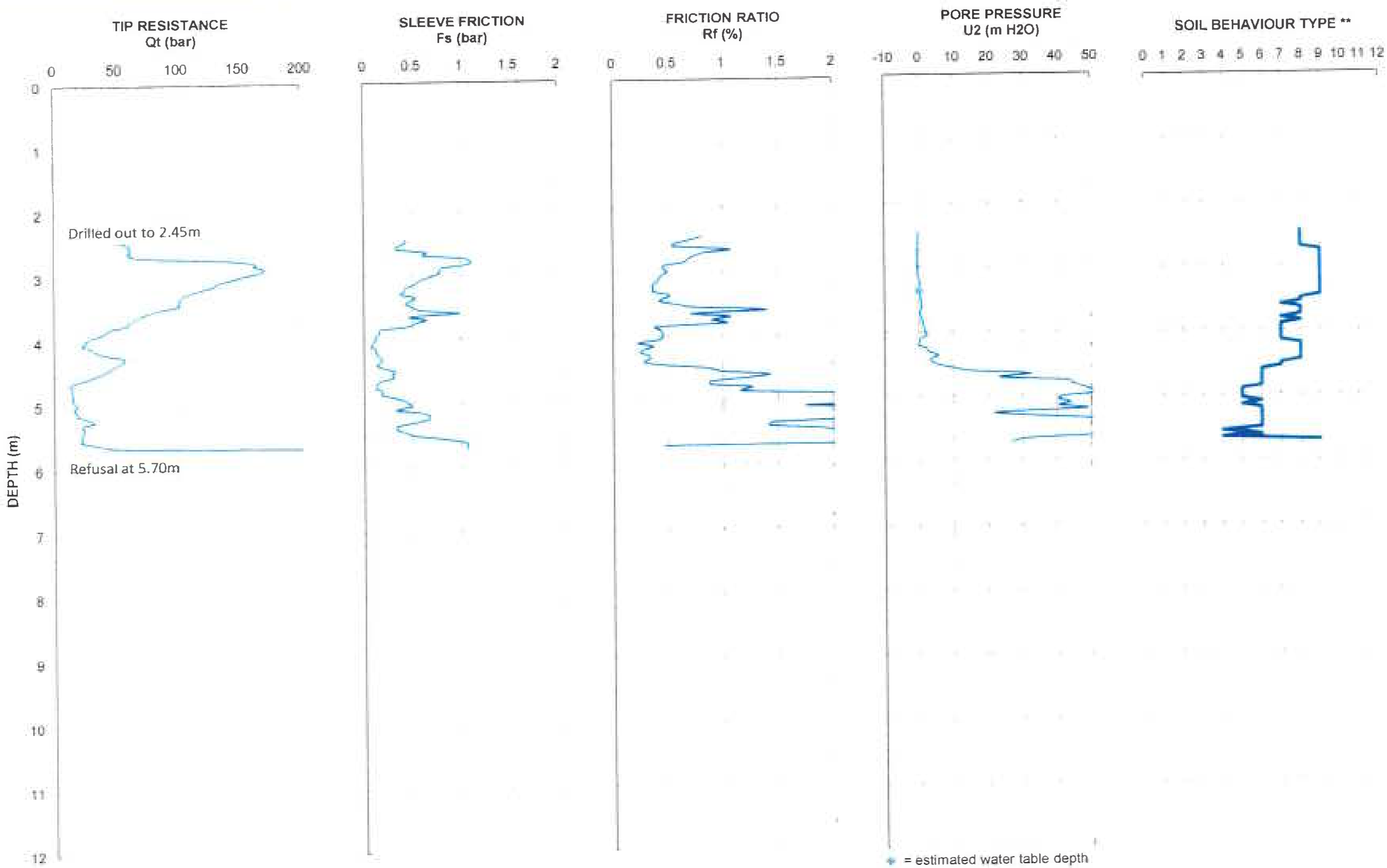
	Sounding: CPT16-01	2101 CLARKE STREET, PORT MOODY	Figure: B.01



** 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

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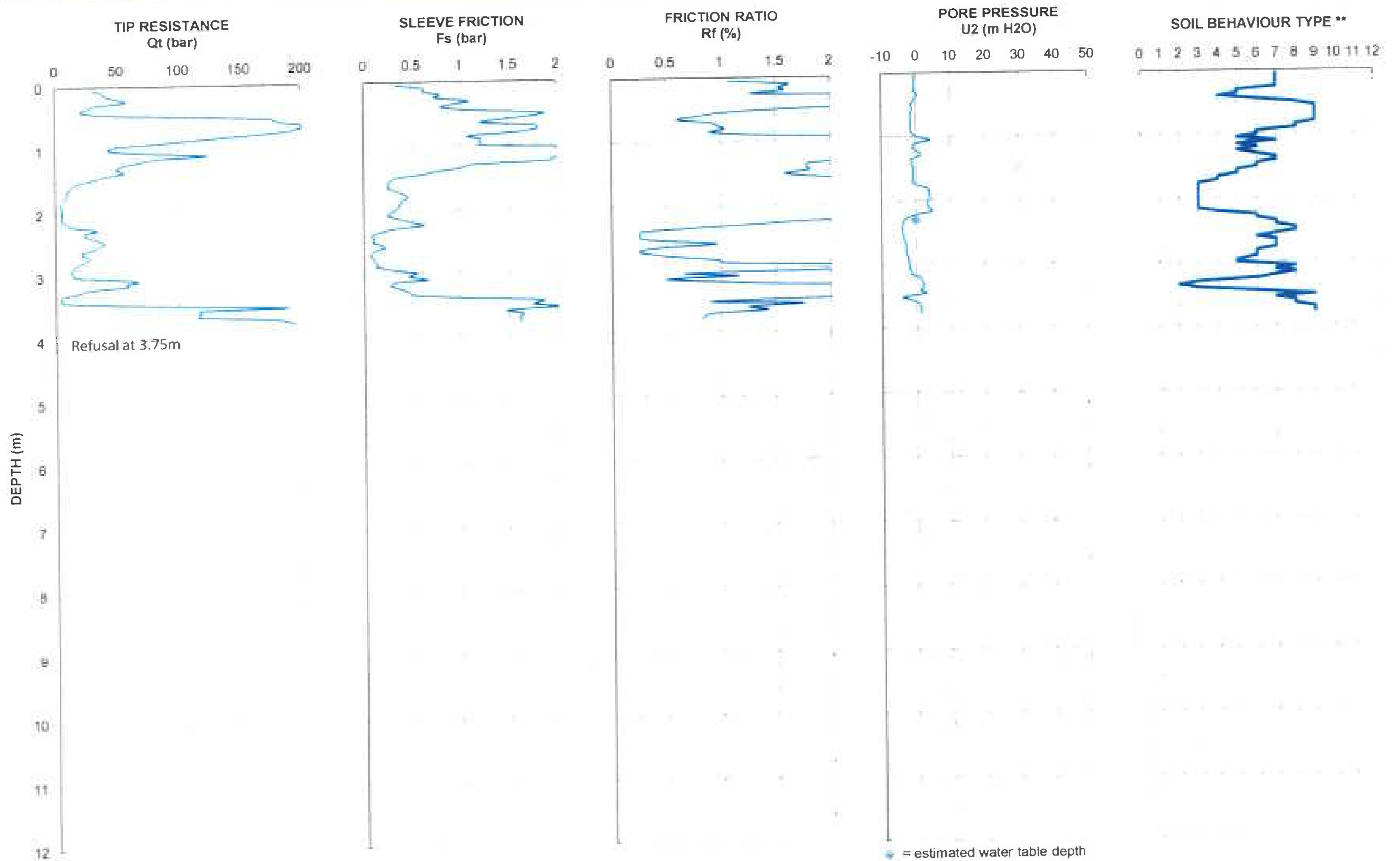

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APPENDIX C - INTERPRETED PARAMETERS

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.

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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

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:

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

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.

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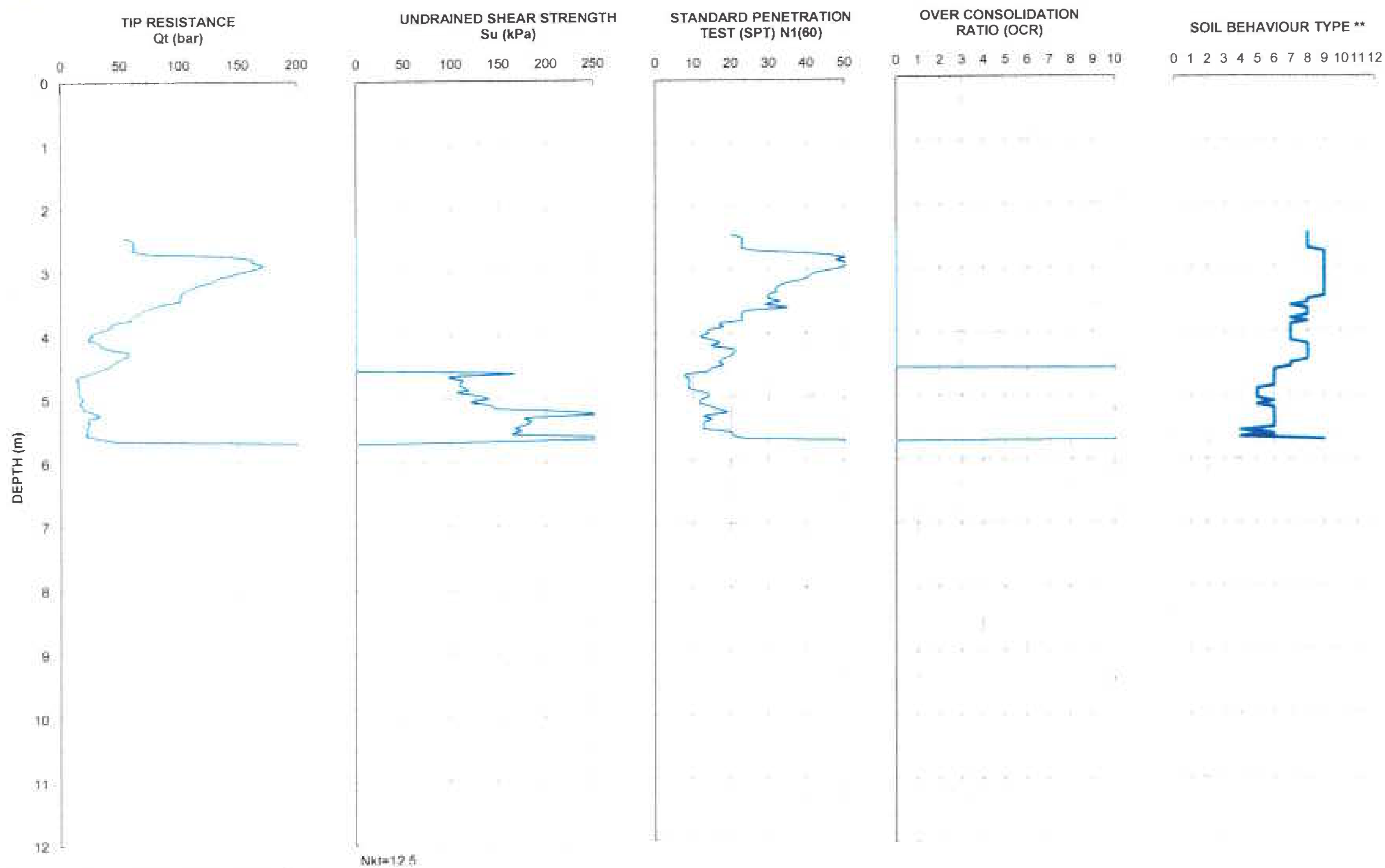

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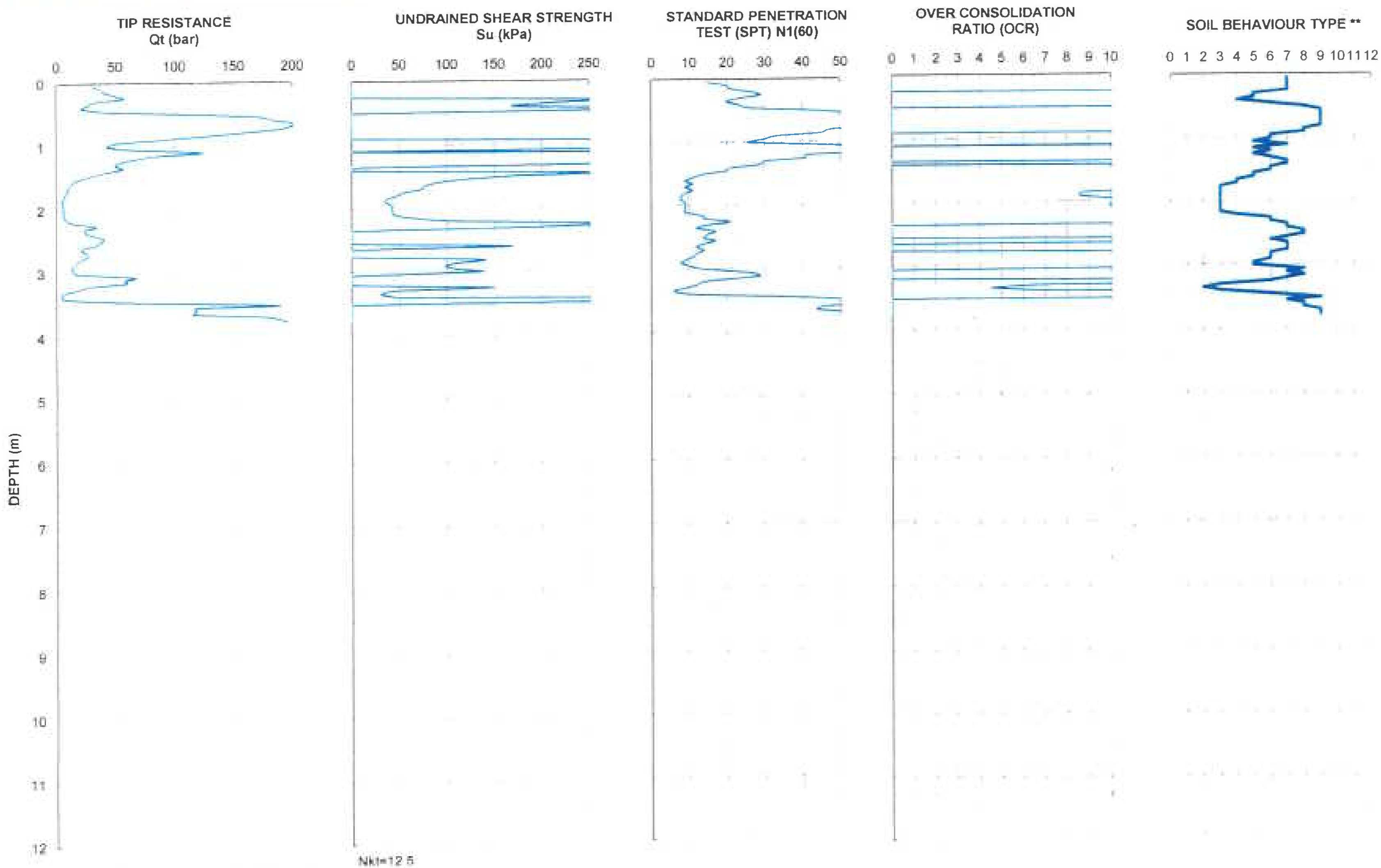

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APPENDIX D - LIQUEFACTION ANALYSIS

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.

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

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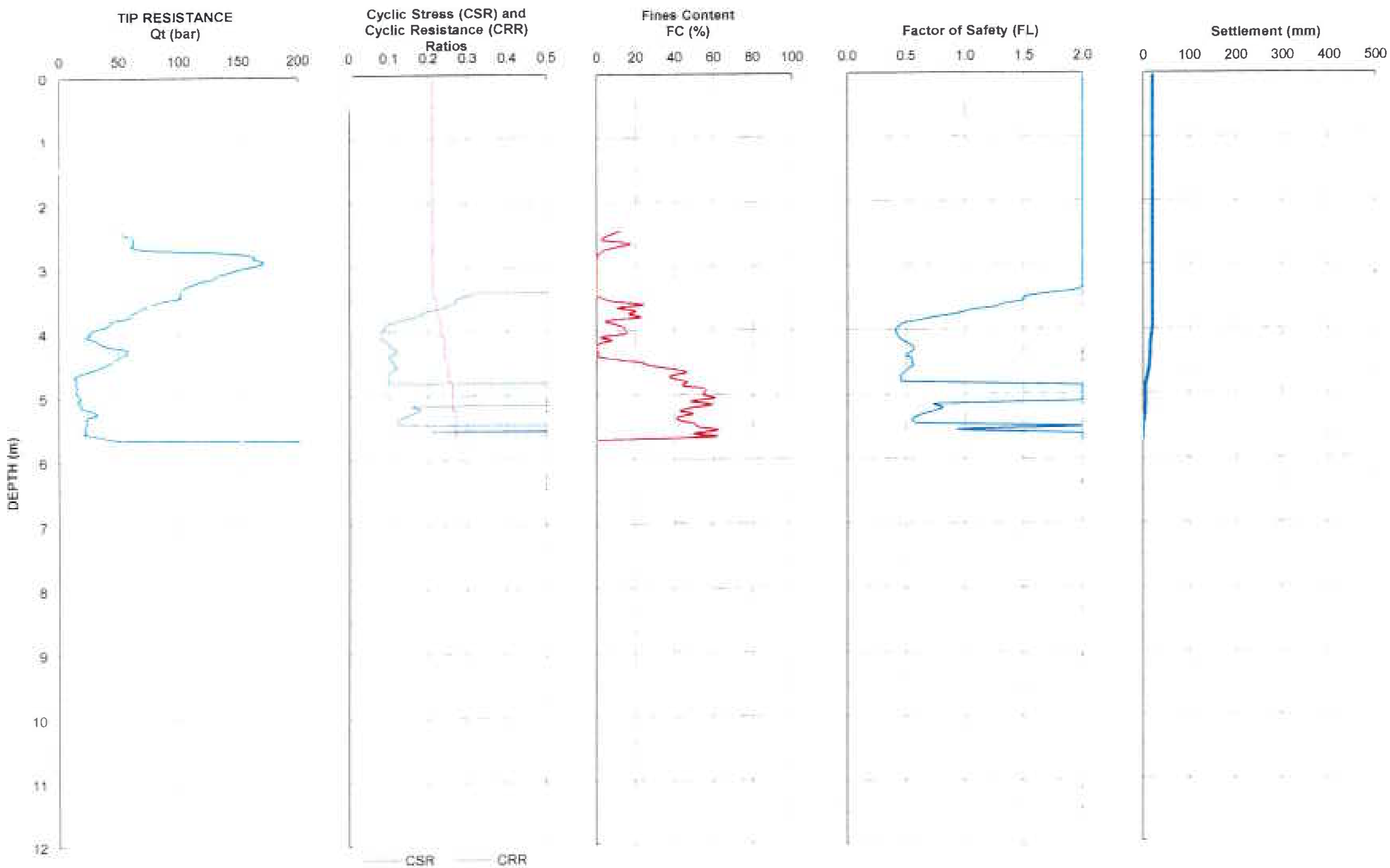

Liquefaction interpretation ran with PGA = 0.33g (2015 National Building Code interpolated seismic hazard value 2%/50 years probability)

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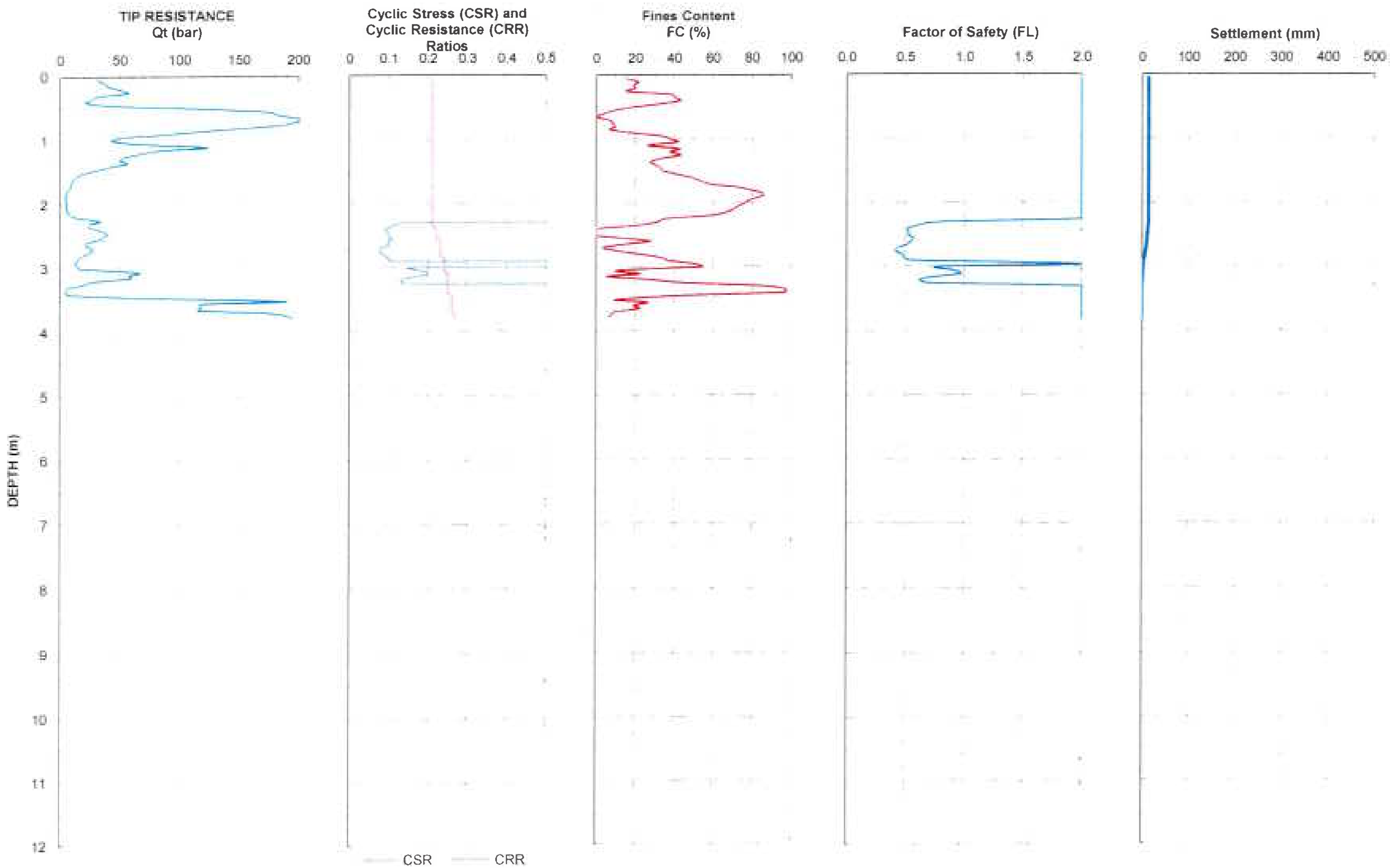

Liquefaction interpretation ran with PGA = 0.33g (2015 National Building Code interpolated seismic hazard value 2%/50 years probability)