APPENDIX Q
Geotechnical Assessment

Fibreco Export Terminal Enhancement
Permit Application Document

APPENDIX Q

Geotechnical Assessment - Executive Summary

GeoPacific Consultants Ltd. provided geotechnical input for the preliminary design of the referenced Project at the Fibreco site. The improvements proposed include the modification of the rail car unloading pit and tunnel, new conveyor transfer towers, conveyor bents supporting an elevated conveyor system, at grade conveyors, storage silos for grain, a new ship loader, the extension of the existing ship berth structure to accommodate Panamax vessels.

This report summarizes the results of a site specific investigation completed on the Fiberco property, in the vicinity of the proposed improvements, and provides recommendations for the design and construction of the new improvements.

- Based on the investigation, to support the heavy structures and the structure with lateral loading, large diameter steel piles are recommenced

- Lighter structures might be supported using conventional shallow foundation, however for the structure subject to lateral and overturning loads or where spatial constraints exist small diameter pipe piles pile foundations may be preferred

- The site is prone to liquefaction, ground improvement may be required for heavy structures and structures near the shoreline, based on the allowable seismic induced ground motion

- Foundations detail specification and bearing capacity presented are presented in section 6.3 of the full report

- Shoring would be required for the temporary deep excavations such as the tunnel, temporary excavation recommendation provided in section 6.4 of this report

- Deep excavation is subject to the groundwater flow, therefore based of the excavation could be sealed or de-watering system might be used alternatively

- Underground structures are subject to lateral loading and should be design based on lateral earth pressure which is provided in section 6.5 of this report

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(Revision)

Attn: Glenn C. Dempster

Re: Geotechnical Investigation Report: Proposed Fibreco Terminal Enhancement Project 1209 McKeen Avenue, North Vancouver, B.C.

1.0 INTRODUCTION

GeoPacific Consultants Ltd. has been retained to provide geotechnical input for the preliminary design of the referenced Project at the Fibreco site.

This report summarizes the results of a site specific investigation completed on the Fiberco property, in the vicinity of the proposed improvements, and provides recommendations for the design and construction of the new improvements.

1.1 Executive Summary

- Based on the investigation, to support the heavy structures and the structure with lateral loading, large diameter steel piles are recommenced.
- Light structures might be supported using conventional shallow foundation, however for the structure subject to lateral and overturing loads or where spatial constraints exist small diameter pipe piles pile foundations may be preferred.
- The site is prone to liquefaction, ground improvement may be required for heavy structures and structures near the shoreline, based on the allowable seismic induced ground motion.
- Foundations detail specification and bearing capacity presented in section 6.3 of this report.
- Shoring would be required for the temporary deep excavations such as the tunnel, temporary excavation recommendation provided in section 6.4 of this report.
- Deep excavation is subject to the groundwater flow, therefore based of the excavation could be sealed or de-watering system might be used alternatively.
- Underground structure are subject to lateral loading and should be design based on lateral earth pressure which is provided in section 6.5 of this report.

2.0 SITE DESCRIPTION

The Fibreco site is located south of McKeen Avenue and west of Pemberton Avenue in North Vancouver. The site is a large operating facility on private and port lands that currently processes and stores wood chips/pellets. The site is bounded by McKeen Avenue to the north, the Seaspan site and a narrow drainage channel to the east, adjacent industrial lands to the west and Burrard Inlet to the south. The new improvements would be located at the south end of the Fibreco, in an area currently used for chip storage. The site is mainly level, except for product storage areas.

The location of the site and existing improvements is shown on the attached site plan, Drawing 13026-01.

3.0 SITE INVESTIGATION

An investigation of the subsurface conditions was completed by GeoPacific from June 16 to 18, 2015. At that time, a total of 7 solid stem auger test holes and 7 electric Cone Penetration Test (CPT) soundings were completed to depths of up to 41 and 36 m below grade, respectively. Two seismic shear wave velocity profiles were completed on site to assist with the seismic evaluation. Prior to any drilling all test hole locations were cleared by Quadra Utility Locating. The test holes and CPT sounding were completed using a truck-mounted drill rig supplied and operated by Uniwide Drilling of Burnaby, B.C. The test holes and CPT sounding were sealed immediately following the completion of testing, sampling and logging in accordance with Provincial abandonment requirements. The site investigation was supervised and the soils encountered were logged and sampled in the field by a geologist from our office. The test hole and CPT sounding logs are presented in Appendix A and B of this report, respectively. Interpreted shear strengths and liquefaction analyses are presented in Appendix C and D, respectively. The shear wave velocity data is presented both in tabular and graphical form in Appendix E.

The approximate test holes and CPT locations are presented on Drawing 13026-01, following the text of this report.

4.0 SUBSURFACE CONDITIONS

The site is located within the Capilano River and Mackay Creek/Mosquito Creek alluvial fans. Based on the GSC Surficial Geology Map 1486A, the site is underlain by stream and channel deposits of sand and gravel to sand. Filling of the lands with dredged materials is know to have occurred on most of the port lands in North Vancouver.

At our test hole locations, subsurface soil conditions were noted to consist of 5 to 6 m of fill which varies from mainly dense sand and gravel in the upper several metres, becoming silty sand and gravel to mainly silt and gravel below this. Below about 2.5 m the fill becomes loose to compact.

The fills are underlain by natural deposits comprised of firm to stiff silt locally and then sand interbedded with silty sand and silt layers and zones of sand and gravel. The sand contains frequent shell fragments, which is typical for marine sands in the area. Based on deep test holes completed in the vicinity of the site, the sand to sand and gravel is expected to be underlain by glacial till at about 50 m below present site grades.

Groundwater was noted at between 4.4 and 4.8 m below current site grades, which is near mean sea level as might be expected given the sites proximity to Burrard Inlet. We anticipate some tidal fluctuation, though it is likely to be less than 0.3 m inland from the top of slope.

5.0 SEISMIC CONSIDERATIONS

The 1 in 2,475 year design earthquake, as defined in the 2010 National Building Code would result in a firm ground acceleration of 0.45 g at the site. The depth to firm ground will result in a reduction of the surface acceleration to about 0.35 g.

We have completed a liquefaction assessment of the subsurface soil profile, the results of which are in Appendix D of this report. Localized zones of sand are predicted to liquefy below a depth of a about 10 m during a major earthquake such as the 1 in 2,475 year design earthquake event. This would result in ground movements, both vertically and horizontally. Vertical settlement is expected to vary from 25 mm to 200 mm at the site. Lateral displacement is expected to be in the range of 100 mm to 400 mm on level ground and up to several meters near the existing shoreline, where lateral spread would be likely to occur. Since liquefaction is predicted below 10 m depth, light structures founded nominally below grade would not be subject to a significant loss in bearing capacity. Heavier structures such as the grain silos would likely be impacted by liquefaction.

For heavy structures or structures near the shoreline, including the berth structure, some ground improvement may be required depending upon allowable seismic induced ground movements.

5.1. Site Specific Dynamic Analysis

We have completed a linear site specific dynamic analysis in accordance with the our site investigation information and the shear wave velocity profile. Detailed description for the subsurface materials to about 40 m depth were provided in Section 4.0. Based on our experience in the area, the sand to sand and gravel is expected to be underlain by glacial till at about 50 m below present site grades.

The shear wave velocity profile in conjunction with our test hole information were used to establish a soil column model employing the computer program SHAKE 2000 (Version 6.0), developed by GeoMotions LLC (2009). SHAKE 2000 is a one dimensional equivalent linear elastic analysis software program which calculates the surficial earthquake response based on an earthquake imposed at depth on firm ground and propagating shear waves from that earthquake upwards through the soil column model.

We have undertaken our analyses employing six different earthquake ground motions which have been spectrally matched to the design spectra outlined for firm ground sites, defined in the 2012 BCBC, in the Lower Mainland. The six earthquakes employed in our analysis include modified motions from the Chi-Chi Earthquake (Taiwan, 1999), Landers Earthquake (California, 1992), and Loma Prieta Earthquake (California, 1989). The north-south and east-west responses for each of the three earthquakes were employed for a total of six motions.

We have plotted on the attached Figure 1 (Appendix G) the results from our SHAKE 2000 analysis as well as the BCBC design spectra for Site Class D and Site Class E for reference. Based on the results of our analysis we recommend that the "Recommended Design Spectra" shown on Figure 1, and numerically presented in Table 1, be employed in the structural design.

It should be noted that this analysis has been undertaken based on design spectral acceleration responses prescribed in the 2012 BCBC. Earthquake ground motions occurring in the future which do not conform to the BCBC design spectra should be expected to result in varying ground motions at the surface not in conformance with the results of this analysis.

Table 1 - Recommended Design Spectra

Natural Period (seconds)	Spectral Acceleration (g)
0	0.98
0.5	0.98
1.3	0.78
2	0.31
3	0.19
4	0.11

6.0 RECOMMENDATIONS FOR DEVELOPMENT

6.1 General

The new structures proposed have not been designed, though preliminarily we anticipate much of the work would occur at the south end of the existing Fibreco site.

Improvements located in close proximity to the foreshore, or off shore, will include the ship berth upgrading and conveyor transfer towers. Conveyor transfer towers, ship loader and the ship berth facility will be subject to significant lateral loads in addition to substantial axial loads. Unsupported lengths of foundation piles could be up to 10 m. For this type of loading larger diameter steel pipe piles are preferred.

Light structures, including grade supported and elevated conveyors could be supported conventionally using shallow foundations. For the elevated portion where overturing loads are likely to govern foundation design or where spatial constraints exist, pile foundations may be preferred. For these structures small diameter pipe piles, H-piles, helical piles or drilled micropiles are all considered feasible. Where vibration sensitivity exists drilled micropiles or helical pipes may be preferred to driven piles.

The grain storage silos are expected to be on the order of 32 m high and exert average ground pressures of up to 200 kPa at foundation level. As indicated above, nominal liquefaction is predicted below about 10 m depth. This could result in some differential settlement however it is unlikely to result in a significant loss of bearing capacity. The upper fill soils, up to 6 m deep at the site, contain some loose to compact zones that may settle excessively if left unimproved. To mitigate the potential for larger than acceptable post construction settlement under both static and seismic loading scenarios, stone column improvement of the subgrade is recommended for the grain silos. Piles can also be considered, however this option is considered more costly and time consuming.

6.2 Site Preparation and Ground Densification

Our test holes showed little if any topsoil and deleterious materials within the upper fills. Thus stripping should be minimal, though existing buried utilities and structures should be removed prior to undertaking any foundation preparation or densification works.

Densification/ground improvement type and depth will depend upon a number of factors including the proximity to the top of bank (Burrard Inlet), average loading and other factors that will likely be determined at the time of detailed design. In general we would expect that two type of densification would be feasible at the site, shallow surface compaction, using the Rapid Impact Compaction (RIC) method and deep densification using full displacement stone columns or timber compaction piles. Deep densification would be used where heavy structures are proposed on grade or where liquefaction mitigation to significant depth is required.

6.3 Foundations

6.3.1 Foundations for Light at Grade Structures

As noted above, conventional foundations are considered feasible for light at grade structures such as the at grade conveyors and rail facilities. The exiting fill soils are considered adequate as is for these structures.

To limit differential settlements, for linear structures such as the conveyors and rail lines, the alignment should be treated by surface compaction using the RIC method, noted above.

Shallow foundations on fill should be designed for bearing pressures of 150 kPa and 300 kPa at SLS and factored ULS conditions, respectively. Settlement of foundation at the bearing pressures given should be less than 25 mm total and 2 mm per metre differential.

6.3.2 Foundations for Storage Silos

The storage silos are understood to be up to 32.5 m tall and are expected to impose high localized stresses. To accommodate these stresses, some ground improvement is considered necessary. Alternatively pile foundations can be used. Locally a firm to stiff silt was identified beneath the fill on the site. In addition some of the fills in the upper 5 to 6 m of the subsurface profile were noted to be loose to compact.

Full displacement stone columns installed using the rammed impact pier (RIP) method or the dry bottom feed vibro compaction methods are considered most appropriate to improve the ground condition sufficiently to support the silos on a mat foundation. Stone columns are expected to be up to 15 m long with a spacing of 1.5m to 2.0m and a diameter of 600 mm to 700 mm. Bearing pressures of up to 300 kPa are considered feasible for improved ground at the site. Settlement should be limited to 70 mm total and 2 mm per metre differential at the bearing pressures given.

Pile foundations can also be used to support the silos. While a number of piles can be considered, steel pile piles are expected to be most economical given the anticipated loading. The pile axial capacity has been calculated based on CPT data using the Fellenius method for the steel pile piles with 305, 610 and 915 mm diameters. The factored ULS capacity of the piles are summarize in Table 2. when driven to depths of 20 to 22 m. The results of the pile axial capacity analysis is presence in Appendix G following the text of this report.

Table 2-Summary of the pile capacity for the steel pipe pile at 20m depth

Test (Location)	Pile Diameter (mm)		
	305	610	915
	Factored (ULS) Pile Capacity (kN)		
CPT15-01	775	2100	3800
CPT15-03	680	2000	3800
CPT15-04	620	1750	3200
CPT15-05	620	1750	3200
CPT15-07	800	2500	5500

6.3.3 Foundations for Berth Structure, Ship Loader, Conveyor Transfer Towers and Conveyor Bents

These structures are expected to resist relatively heavy axial and lateral loads. The berth structure and the ship loader structure will be constructed off shore. Given the relatively high axial and lateral demand for these structures steel pipe pile foundations are considered most appropriate. The piles should be driven closed ended.

Our liquefaction assessment indicates only marginal liquefaction at the site. Thus ground improvement is not considered necessary to achieve reasonable performance under a major earthquake, such as the 1:2,475 design earthquake.

We have completed a series of pile analysis under lateral loading employing the computer modeling software, LPILE (Ensoft Inc. 2013) to assess theoretical depth of 18m for the proposed pipe piles. We have considered a series of steel pipe piles with diameters ranging from 305 mm (12") to 915 mm (36") and with a wall thickness of 9.5 mm (3/8") to 15.88 mm (5/8").

In our analyses, we assumed both fix and free condition at pile heads and various lateral loads provided for tower 2 and tower 3 were applied at the pile head. Material properties of the steel pipe piles include a Young's modulus of 210 GPa and a yield strength of 248 MPa for both tensile and compressive loading conditions were considered. The results of our analyses are presented in Appendix H.

Detailed pile design can be provided once final loading has been determined.

6.3.4 Foundations for Rail Car Unloading Pit and Tunnel

The depth of the rail car unloading pit is understood to be 12 m and the tunnel will vary from at grade to 12 m deep. The water table at the site was identified at between 4.4 and 4.8 m below current site grades. Some seasonal and tidal fluctuation should be anticipated so that a static water level of say 4 m below grade should be considered for the design. Thus lateral and uplift pressures due to groundwater would be up to 80 kPa.

The rail car unloading pit and tunnel will be constructed of reinforced concrete construction. Given the position of the water table and the relatively high permeability sand to sand and gravel soils at the site, the major design challenges will be related to excavation support and groundwater control during construction and buoyancy and later pressures in the final condition.

Foundation design will be governed by uplift pressures rather than tunnel or pit loads. Where net gravity loads are being applied, we recommend limiting bearing pressures to 150 kPa and 300 kPa for SLS and ULS conditions, respectively.

Uplift forces can be resisted using shear keys or other structural elements that engage the backfill soils. Shear keys can be designed using a friction angle of 35 degrees, a dry buoyant weight of 18 kN/m3 and a buoyant unit weight of 9 kN/m3.

Alternatively hold down anchors can be considered. Drilled anchors are feasible where they can be installed above the water table. Helical anchors are likely more economical and can be installed below water if necessary. Service uplift loads of up to 50 to 100 tonne can be reasonably resisted with drilled anchors or helical anchors.

6.4 Temporary Excavations

Temporary excavations for the tunnel would require shoring given the contemplated depth of construction. A number of methods could be utilized including sheet pile wall, secant pile wall or jet grout wall. Sheet pile is the most economical alternative however it is also likely to allow the most seepage through the wall. We expect that bracing would be preferred to using tie backs since the tie backs would introduce perforations in the wall that would need to be sealed. The base of the excavation could be sealed against groundwater intrusion or alternatively the excavation could be de-watered. De-watering would take the form of deep wells. For an excavation without a sealed base, groundwater flows would be quite significant. Depending upon environmental restrictions and treatment requirements, it may be more economical to seal the base of the excavation.

Open cuts stabilized by temporary de-watering are considered feasible for excavations up to 6 m below grade.

Given the configuration of the tunnel, a portion of it would be founded on the existing on site fills. Locally these fills are loose to compact and underlain by firm to stiff silt. To avoid differential settlement of the tunnel, we recommend surface compaction of the tunnel alignment using the RIC method described above. Only that portion of the alignment shallower than 6 m below grade would require RIC treatment. Where silt is encountered at subgrade elevation, it should be sub-excavated and replaced with 300 mm minus fractured rock fill, compacted to 100 percent of its ASTM D698 (Standard Proctor) maximum dry density.

6.5 Earth Pressures

Earth pressures against the retaining walls are dependent on factors such as lateral restraint along the wall, surcharge loads, backfill materials, groundwater elevation and drainage conditions.

The lateral pressure on the rail car unloading pit retaining wall would depend on the type of backfill materials, the level of compaction and groundwater elevation. Considering a static water level of around 4m below grade, we recommend that retaining walls be designed to resist the following lateral earth pressures:

Table 3 - Lateral soil pressure

	Static *	
	Active Pressure (kPa)	Passive Pressure (kPa)
Above Groundwater	4.0H	50H
Below Groundwater	11.5H	25H

* The loads are based on triangular soil pressure distribution, where H is equal to the total wall height in metres.

The walls should be designed to withstand an additional seismic earth pressure of 3.5H kPa with an inverted triangular pressure distribution. Any additional surcharge loads located near the foundation walls should be added to the earth pressures given.

7.0 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 this project. If you would like further details or require clarification, please do not hesitate to contact us.

For:
GeoPacific Consultants Ltd.

Reviewed by:

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