OUR REF 10113

ABUILD™
Consulting Engineers Ltd
Geotechnical and Civil Engineers

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OTAKI

ATTENTION: Bryce Holmes

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Dear Sir

SITE LIQUEFACTION POTENTIAL FOR PROPOSED SUBDIVISION
DUCK CREEK NORTH, WHITBY, PORIRUA

Following a meeting at Porirua City Council Chambers on September 24, 2015, a request was made to carry out a more extensive review of the site liquefaction potential and, in particular, to address the following aspects:

- Identify areas on site that are susceptible with respect to potential liquefaction.
- Design earthquake shaking and liquefaction trigger.
- Consequences of potential liquefaction.
- Mitigation options.

Site liquefaction potential was broadly discussed in the ABUILD™ geotechnical report dated August 2015 which should be read in conjunction with this letter. This report highlighted subsoil variability and the broad range of soil strength and density of subsoils across the site. Some sediments encountered in BH1 put down in the northern part of the site were considered susceptible to liquefaction under earthquake shaking and mitigation measures comprising engineered fill to 'raft out' impacts and dissipation of pore water pressures through the installation of permeable drains were noted.

The above aspects are discussed in turn as follows:

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1.0 AREAS THAT ARE SUSCEPTIBLE WITH RESPECT TO LIQUEFACTION

1.1 Susceptible Soils

Soils which are most susceptible to liquefaction are uniformly graded fill sands, and to a lesser extent silty sands and sandy silts. Cohesive soils such as clays are generally not considered susceptible to liquefaction. Coarse granular soils such as gravel usually have a high permeability and are also generally nonliquefiable. The New Zealand Geotechnical Society's "Guidelines for the Identification, Assessment and Mitigation of Liquefaction Hazards" (NZS 2010) provides criteria for the assessment of soils susceptible to liquefaction. Soils that are susceptible to liquefaction exhibit "sand-like behaviour" and those soils not susceptible to liquefaction exhibit clay-like behaviour. A further qualification is provided with respect to soil plasticity as follows:

Soil Plasticity (PI)	Risk of Liquefaction
>12	Low Risk
>-12	Medium Risk
<7	High Risk

1.2 Subsoil Conditions

The variability of subsoil conditions across the site precludes the development of a simple soil model that may be used to identify zones of potentially liquefiable soils.

Subsoil conditions in the ABUILD™ report were generally divided into areas described as follows:

1.2.1 North Area

BH1 put down in the northern part of the site encountered sequences of bedded silty sands, sandy silts and silts that were typically weak and compressible to a depth of 6.6 metres at this location. Dense alluvial gravels were encountered below the bedded sands and silts and extended to the depths explored.

1.2.2 North West Area

Alluvial silt in the northernmost part of this area (TP1) was typically stiff to very stiff to the depth explored of 2.7 metres below ground level.

CPT6 encountered a weak layer of soil between 2.1 and 4.1 metres below existing ground level. Based on the friction ratio and the cone resistance we interpreted this layer as comprising organic clays and compressible silty clay. BH2 put down in the southern part of this area encountered loose silty sand between 0.5 and 1.9 and soft silt between 2.5 and 3.3 metres below existing level. These soils were underlain with competent gravel and sand to a depth of 6.7 metres below ground level and consequently underlain with bedrock to the depth explored.

1.2.3 North East Area

Test pits put down in this area encountered typically very stiff fine grained soils underlain by medium dense gravel. CPT4 put down in the main body of this area encountered loose sand to a depth of about 2.1 metres below ground level. The sand deposit is underlain by inferred dense gravel.

1.2.4 South West Area

CPT2 was terminated at a depth of less than 1.0 metre in inferred greywacke rock. TP8 put down reasonably close to CPT2 encountered cobbles/gravel, below a topsoil layer, that extended to a depth of about 1.8 metres. The gravel was underlain with firm to stiff clay to the depth explored.

1.2.5 South East Area

We interpret subsoils encountered at CPT1 location in the southernmost part of the site as comprising medium dense gravel overlying very stiff sandy silt and weak organic silt/clay to a depth of 3.6 metres below ground level. We interpret that this CPT was terminated at or close to weathered rock.

2.0 DESIGN EARTHQUAKE SHAKING

The extent of liquefaction and/or cyclic strain softening will be governed in part by the peak ground acceleration (intensity of earthquake shaking). With respect to the criteria in NZS 1170.5:2004 Structural Design Actions Part 5 Earthquake Actions, Clause 2.1.5 specifies that in order to comply with the Building Code design of structures two earthquake cases must be considered:

1. SLS Case – under an earthquake event with an annual probability of exceedance of 1 in 25 year return period, damage to any building shall be avoided.

2. ULS Case – an earthquake event with an annual probability of exceedance of 1 in 500 year return period, shall not cause collapse of any building and shall not threaten the lives of any occupants within these buildings.

Peak ground acceleration associated with each design case is as follows:

Design Case	Peak Ground Acceleration (PGA)	Magnitude (M)	Return Period (years)
SLS	0.11 g	6.5	25
ULS	0.43 g	7.5	500

The above accelerations have been derived using a site subsoil Class C and have been used to assess the site liquefaction potential as follows.

3.0 LIQUEFACTION ASSESSMENT

The evaluation of the site liquefaction potential has been assessed using two methodologies as follows:

1. A simplified procedure by Berrill has been adopted using SPT tests obtained from BH1 and BH2.

2. A detailed soil model has been generated from CPT data input into a software programme "CPT Liquefaction Assessment Software" (CLIQ Version 17.6.34).

The analysis has shown that the upper sand layers in BH1 in the northern part of the site are expected to liquefy under a ULS level of ground shaking. The sand layer between 1.0 and 2.0 metres depth in BH1 is not expected to liquefy under an SLS level of ground shaking, however, the lower sand deposit between 3.0 and 5.0 metres depth is expected to liquefy under an SLS level of ground shaking.

BH2 in the north western part of the site encountered loose sand deposits to a depth of 1.9 metres below ground level. This layer would be expected to liquefy under an SLS level of ground shaking. Ground improvement by compaction will densify this layer and improve its resistance to liquefaction under an SLS level of ground shaking and will be rafted over by the proposed filling.

CPT6 put down in the north western part of the site indicates that the inferred sand-like deposits are not expected to liquefy under an SLS level of ground shaking and that isolated relatively thin zones to a depth of 9.2 metres may be liquefiable under a ULS level of ground shaking.

CPT1 put down in the south eastern part of the site indicates that the upper sediments are not expected to liquefy under an SLS level of ground shaking and that isolated and interbedded zones of sand-like deposits may be liquefiable under a ULS level of ground shaking.

By inspection of CPT4 and CPT5 the variable depth of sand-like deposit extending to depths of between 1.2 and 2.2 metres below existing ground level have some resistance to liquefaction under an SLS level of ground shaking and in their present conditions would be expected to possibly liquefy under a ULS level of ground shaking. This analysis and review indicates that the sand-like susceptible soils are essentially subsurface and that the density of these deposits may be improved significantly by compaction to provide resistance to at least an SLS level of ground shaking and will be rafter over by the proposed filling.

4.0 CONSEQUENCES OF LIQUEFACTION

The analysis has indicated that the liquefaction trigger is likely to be about or just above the SLS level of ground shaking and likely in the range of 0.1 to 0.15 G but depending on the actual density of the sand-like deposits across the site. Above the liquefaction trigger level the susceptible soils are expected to undergo some settlement with associated differential movement across the site the magnitude of which would be governed by the thickness and density of the inferred liquefiable soil layers.

These aspects are discussed as follows:

4.1 Settlement Prediction

Estimates of settlement of sand and sand-like deposits under earthquake shaking have been assessed using a method developed by Tokimatsu and Seed (1987). For susceptible sand and sand-like deposits the following settlements may be expected to occur in the existing susceptible deposits at BH1 location.

Layer Depth/ Thickness	SLS Level of Ground Shaking		ULS Level of Ground Shaking	
	Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)
1 – 2	0.1	10	1.8	18 – 20
3.5 - 5	2.2	30 - 35	2.8	40 - 45

Site development comprises the placement of a variable depth of fill, generally between 1.5 to 3.0 metres in depth, to achieve site design levels. The placement of compacted filling will provide a dense surface layer to support structure loads and effectively raft out potential differential movement due to possible isolated settlement of the underlying soils.

Research has shown that liquefaction resistance is increased with increasing thickness of a nonliquefiable crust over a given thickness of liquefiable soil.

Using boundary curves for identification of liquefied-induced damage (Ishihara, 1985) has been applied to the site profiles at BH1 and BH2 locations for a range of depths of fill required to achieve design levels across the site.

Nonliquefiable Crust Thickness (mm)	Liquefiable Layer Thickness (m)	Liquefaction Resistance (PGA) (g)
1.0	1.0	0.18
2.0	1.0	0.27
3.0	1.0	0.4
1.0	2.0	0.13
2.0	2.0	~0.2
3.0	2.0	~0.3

The results indicate with a depth of filling across the site as indicated on Cardno drawings NZS0115065-PL-C111 1 to 3, the resistance of liquefaction is triggered well above an SLS level of seismic ground shaking.

Settlement of dwellings situated on a nonliquefiable crust is therefore expected to be minimal under a seismic event up to the level of resistance shown.

4.2 Differential Settlement of Infrastructure

Differential settlement of proposed development infrastructure including roading, footpaths and underground services is not expected to be significant, requiring mitigation after an earthquake event. This is because the underground services are expected to be located within the proposed filling and the roading network constructed at or close to the surface of the proposed bulk filling.

Similarly buoyancy and uplift of buried pipes and manholes is not likely to be an issue if situated within the filling but depending on the depth of filling.

Differential settlement across the width of the proposed road may be observed under a seismic event where the proposed roadway traverses a variable depth of filling, or filling and cutting in the south western portion of the site. However, at the boundaries of the proposed filling the softer soils are expected to be thinner as the site transitions to the hillside gravels and rock and therefore any settlement should be minimal.

5.0 LIQUEFACTION MITIGATION

Filling over the weak and potentially liquefiable soils has been previously noted and is the predominant mitigation option. This comprises stripping of the surface weaker soils, systematic compaction of the underling soil layer and the placement of a fill layer over the potentially liquefiable areas.

In addition, pore water pressure dissipation was mentioned in the August 2015 report. This ground improvement option comprises the installation of permeable drains to prevent the build-up of pore water pressures and improve the soils resistance to liquefaction.

The most effective liquefaction mitigation option comprises compaction of the surface soils which are potentially prone to liquefaction during earthquake shaking. Initially all areas to be filled will be stripped of unsuitable surface soils. The depth of unsuitable soils across the site was described in ABUILD™'s letter dated May 26, 2015. The practicality of subgrade compaction following stripping of unsuitable soils will be much improved by the placement of a nominal depth of conditioned fill soil. The placement of a nominal depth of conditioned fill soil prior to compaction will reduce weaving which may be significant if compaction is undertaken directly on the natural alluvial soils.

The mitigation option of ground improvement by compaction is considered to be routine and practical and an effective way to improve the density consistency of the near surface sandy subgrade soils and improved resistance to liquefaction. This will be achieved by specific supervision and inspections as the earthworks proceed.

6.0 CONCLUSIONS

The review of the existing subsoil data in the ABUILD™ report dated August 2015 has been undertaken for the purpose of providing further information with respect to the site response to liquefaction during earthquake shaking.

This review has indicated that:

- The subsoils encountered in BH1 put down in the northern part of the site, were potentially liquefiable, whereas the subsoils encountered in the main body of the site were generally considered nonliquefiable under SLS conditions other than isolated thin layers.

- Filling was recommended in conjunction with pore water pressure dissipation through drainage in the northern part of the site. Whilst an appropriate and practical mitigation solution, this option will be time consuming but depending on the fines content in the potentially liquefiable soils in this part of the site. The degree of densification and hence resistance to liquefaction would need to be confirmed by before and after specific penetrometer testing. This method has been undertaken in Shoal Place and Duck Creek South and this site is expected to be similar.

- By contrast, ground improvement over most of the site can be carried out by the practical and routine option of compaction of the subgrade soils. Compaction is likely to be most effective following the placement of a nominal depth of conditioned filling.

- Analysis has shown that the placement of a depth of compacted filling across the site will raft out the potential for any differential movement that could occur during a significant earthquake event.

- Analysis has shown that the placement of a nonliquefiable crust (filling) over potentially liquefiable soil will improve the site's resistance to the potentially damaging effect of liquefaction. The thickness of the filling proposed across the site will increase the trigger level of liquefaction to well above an SLS level of ground shaking.

7.0 LIMITATIONS

This report has been prepared solely for you as our client with respect to the brief provided. Data or opinions contained in this report may not be used in other contexts or for any other purpose without our prior review and agreement.

It is in all parties' interests that we be retained to examine the site during foundation preparation and construction work so that exposed subsoil and actual site conditions can be compared with the report assumptions. In all circumstances, however, if variations in the subsoil occur which differ from that described or are assumed to exist, then the matter should be referred back to us.

Yours faithfully
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