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Nelson City Council
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Attention: Mr Drew Hayes

Dear Sir

Re: Peer Review of Trafalgar Centre Seismic Evaluation

1. Background

As requested a review has been carried out for the recent geotechnical seismic evaluation of the Trafalgar centre. This review is based on the following documents

- Trafalgar Centre Seismic Evaluation Report, Holmes Consulting Group, May 2013
- Trafalgar Centre Geotechnical Report, Tonkin and Taylor Ltd., June 2013

Comments and conclusions are given below.

2. General philosophy and approach

The Tonkin and Taylor (T&T) report follows generally accepted design philosophies and is appropriate for the purpose of evaluating geotechnical seismic stability of the Trafalgar Centre. The central issue is whether or not liquefaction will occur within the soils beneath the Centre under design earthquake conditions.

3. Interpretation of ground conditions

In my view sufficient investigations were carried out and an appropriately accurate model of sub-surface conditions is given.

4. Seismic design parameters

The report presents a conservative picture for the design earthquake. Applying importance level 4 (*IL4*) to the Trafalgar Centre places a severe seismic demand on the structure and its foundations. This is appropriate provided the Centre remains designated a post-disaster evacuation and welfare centre. The *IL4* earthquake conditions with magnitude 7.5 and peak ground acceleration 0.65g will be referred to below as the design earthquake.

5. Liquefaction susceptibility

The central issue in this investigation is liquefaction. Nearly all aspects of stability of both the foundations and the land beneath the Trafalgar Centre hinge on the question of whether or not liquefaction will occur in the design earthquake. The T&T report concludes that the risk of liquefaction is very significant. All the conceptual proposals made by T&T for various aspects of remediation hinge on the conclusion that liquefaction will occur. As a result, the likelihood of liquefaction becomes the primary focus of this review. Arguments may be made both for and against the liquefaction potential of the Trafalgar Centre soils. A summary is given below.

In the comments for and against liquefaction shown below the *IL4* design earthquake has been assumed. In all calculations the water table was assumed to lie at a depth of 2.5m.

The case against liquefaction

- Soils above the water table are generally considered non-liquefiable. Given that the water table lies at 2.5 to 3m below the ground surface, it may be safely assumed no liquefaction will occur in the shallow deposits above 2.5m depth.

- It must be acknowledged that the soils beneath the Trafalgar Centre are not of a nature to immediately suggest liquefaction. Most cases of liquefaction occur in more or less uniform fine sands and silty sands. The soils beneath the Trafalgar Centre are characterised by much more coarse gradation than is usually identified as liquefiable. This fact is clearly pointed out in the T&T report and is emphasised by the comparison of Trafalgar Centre gradation curves with the range of gradations usually considered liquefiable shown in Figure 6.2 of the T&T report.

- In order that liquefaction occur the soil must contract when shaken. The soil particles must attempt to move into a more dense configuration causing the pore water pressure to increase. This is the case for loosely packed soil particles. Densely packed particles on the other hand will tend to dilate which leads to tension in the pore water and no liquefaction. Whether a soil is dense or loose is often characterised by the *state parameter* ψ introduced by Been and Jeffries (1985). If ψ is positive the soil is loosely packed, if negative it is densely packed. When ψ is zero the soil is said to be at critical state. We can approximate the value of ψ based on CPT data (Robertson, 2009).

  Figure 1 shows the state parameter profile beneath the Trafalgar Centre obtained from CPT10. Clearly the state parameter lies entirely in negative values, indicating that the soils beneath the Trafalgar Centre are dilative and may not be susceptible to liquefaction.

- A relatively new approach to analysis of liquefaction potential involves the shear wave velocity of the affected soil. Until recently this method was not widely used due to there being only a relatively small number of case histories for which reliable shear wave data was available. Recently, however, Kayen, *et al.* (2013) have published a much more comprehensive analysis based on extensive shear wave velocity measurements at sites where liquefaction has occurred. Their analysis is based on 422 case histories.

  In their investigation of the Trafalgar Centre soils T&T obtained shear wave data from analysis of surface wave excitation (MASW). The T&T report contains five separate profiles of shear wave velocity surrounding the Trafalgar Centre. Figure 2 shows liquefaction factor of safety obtained from MASW Line 1 using the analysis of Kayen, *et al.* (2013). Regions where the liquefaction FoS is equal or smaller than 1 are expected to liquefy in the design earthquake. The data shown in Figure 2 suggest liquefaction is unlikely to occur except in the shallow reclamation fill above about 4m depth.

  Note that the data in Figure 2 represents only one part of a much larger collection of shear wave measurements. Nevertheless it is typical of the majority of Trafalgar Centre shear wave data. MASW line 5 presents a somewhat different picture and it will be considered later.

The case for liquefaction

- Nearly all case histories of liquefaction involve young soils. The soils beneath the Trafalgar Centre clearly fall into this category. The reclamation deposits are very young and the deeper soils were all formed during the Holocene, the era since the last ice age.

- Despite the general picture that liquefaction may only occur in fine sands and silty sands, there are case histories of liquefaction involving coarse sands and gravels beginning with the Borah Peak earthquake of 1983 (Youd, *et al*., 1985). Other examples are the Armenian earthquake of 1988 (Yegian, *et al*., 1994) and the Wenchuan earthquake of 2008 (Cao, *et al*., 2011). There is also strong geologic evidence for liquefaction in gravels (Moura-Lime, *et al*., 2011). Liquefaction of gravely soil has also been observed in the laboratory (Choi and Arduino, 2004).

- As foreshadowed above MASW line 5 appears to differ from the other lines 1 through 4. Significantly lower shear wave velocities were measured on this line, which is evidently closer to the actual Trafalgar Centre footprint than most of the other MASW profiles. Figure 3 shows the liquefaction FoS for MASW line 5 at Ch12.3. In this case the liquefaction FoS remains smaller than 1 up to a depth of nearly 8m indicating a considerably greater potential for liquefaction.

- By far the most widely used method for analysis of liquefaction susceptibility is the so-called “simplified procedure” originally developed by Seed and co-workers. The method has been extended to encompass CPT data as described in the summary paper by Youd, *et al*. (2001). Several researchers have proposed slightly different versions of the CPT analysis but all methods give essentially the same result.

Raw digitised CPT data for three separate CPTs were provided by T&T for this review. The individual CPTs were numbers 6, 10 and 11. Numbers 6 and 11 are considered to represent slightly lower liquefaction potential while number 10 represents higher liquefaction potential. All three sets of CPT data were analysed using the method of Robertson and Cabal (2010).

Figure 4 shows the liquefaction FoS plotted against depth for CPT11. This CPT gave the smallest liquefaction susceptibility of the three considered. Nevertheless it is clear that significant portions of the soil profile above the 12m depth may have a liquefaction FoS smaller than 1. Note that gaps occur in the plotted FoS points. These arise because the results have been filtered to remove points where the soil behaviour index I_c is greater than 2.6. A value of I_c greater than 2.6 is regarded as identifying soil with sufficient silt and/or clay content to preclude development of liquefaction.

Figure5 shows the liquefaction FoS for CPT10. In this case most of the soil profile between the water table and a depth of 12m is indicated as liquefying under design earthquake conditions.

Evidently strong arguments both for and against the likelihood of liquefaction exist. Nevertheless it is my view the arguments for liquefaction must be taken seriously, and therefore liquefaction of the Trafalgar Centre soils in the design earthquake must be assumed to occur.

6. Comparison of IL3 and IL4 earthquakes

As noted above Importance Level 4 places a large demand on any seismic design. The lower level *IL3* eases the peak ground acceleration by about 30 percent, from 0.65g to 0.47g. It is worth considering whether designing for *IL3* may significantly affect the likelihood of liquefaction for the Trafalgar Centre soils. One approach is to consider the Liquefaction Potential Index.

The Liquefaction Potential Index (LPI) was introduced by Japanese engineers in the 1970’s and has found wide application as a screening tool. LPI offers a single number to represent the effects of depth, thickness and factor of safety of a liquefiable deposit. Since only one number is involved, comparison of different conditions is facilitated. It is easily determined from a CPT liquefaction analysis such as those used here. See the summary paper by Toprak and Holtzer (2003) for details.

Table 1 gives values of LPI for CPTs 6, 10 and 11 for importance levels *IL4* and *IL3* as well as for the SLS2 level for *IL4*. It is widely thought that for a given earthquake a value of LPI smaller than 5 indicates that surface manifestations of liquefaction (*e.g.* sand boils, surface cracking, lateral spreading, subsidence) are unlikely. At the other extreme, an LPI of 15 or more indicates extensive, severe manifestations of liquefaction.

Once again the liquefaction picture is not totally clear. For the *IL4* ULS earthquake liquefaction is expected to result in surface manifestations for all three CPTs. The result for the *IL3* earthquake is mixed. For the *IL4* SLS2 earthquakes it appears the effects of liquefaction will not be manifest at the ground surface. Table 1 further emphasises that designing for extensive liquefaction is not a clear cut decision.

Table 1. Liquefaction Potential Index for Various Conditions

	CPT6	CPT10	CPT11
IL4: PGA = 0.65g	8.6	12.3	7.8
IL3: PGA = 0.47g	5.6	6.5	4.2
SLS2: PGA = 0.36g	3.9	3.3	2.4

7. Lateral spreading

The report states that modelling of lateral spreading suggests a M7.5 earthquake with peak ground acceleration of 0.18g may be sufficient to initiate lateral spreading. No details of this analysis are provided.

A widely used screening procedure for lateral spreading is the analysis of Youd, Hansen and Bartlett (2002). Based on their analysis it is possible to estimate the lateral displacement given earthquake magnitude, epicentral distance, free face geometry, mean grain size and fines content. Using the attenuation relationship of McVerry, *et al.* (2006) the most likely epicentral distance giving a PGA of 0.18g in a M7.5 earthquake is 32km. Use of this distance together with appropriate parameters suggests a likely lateral displacement of slightly less than 50mm at a horizontal distance of 12m from the free face. The displacement will be smaller at greater horizontal distances.

Increasing peak ground acceleration to the SLS2 condition of 0.36g would result in an epicentral distance of 10km. In this case the Youd, Hansen and Bartlett model suggests a lateral displacement of about 230mm. Further increases in PGA will result in greater lateral displacement. It is clear that lateral spreading presents a significant risk to the Trafalgar Centre foundations.

8. Foundation design

Given the conclusion that liquefaction will be prevalent in soils beneath the Trafalgar Centre, demands on the Centre foundations will be severe. Foundations of the existing buildings will be expected to perform poorly under ULS design earthquake conditions with settlement, lateral deformation and rotation all occurring. Demands caused by lateral spreading may be solved by the proposed strip of improved ground around the Centre, but liquefied soil at the level of founding of existing foundations will result in loss of bearing capacity and consequent large deformations.

The conceptual design proposed in the T&T report is considered appropriate. If detailed design is undertaken refinements may be possible. It is fortunate the Port Hills Gravel formation lies relatively close to the ground surface.

9. Ground improvement

The proposal for a strip of treated ground surrounding the Centre given in the report is appropriate for purposes of concept design and initial costing given that extensive liquefaction is anticipated. The report suggests vibro-replacement stone columns as the most appropriate solution for resisting lateral spreading. Other methods including deep soil mixing and EQDrains might also be considered during detailed design.

10. Conclusions

All issues raised in the Tonkin and Taylor report hinge on the conclusion that the Trafalgar Centre soils will suffer significant liquefaction under design earthquake conditions. I support that conclusion but note that for weaker earthquake excitation the extent of liquefaction and its potential for damage is not clear. Some further analysis may also be useful in regard to lateral spreading, particularly since the remediation measures proposed involve considerable expense.

Yours sincerely

R.O. Davis PhD., FRSNZ

Figures

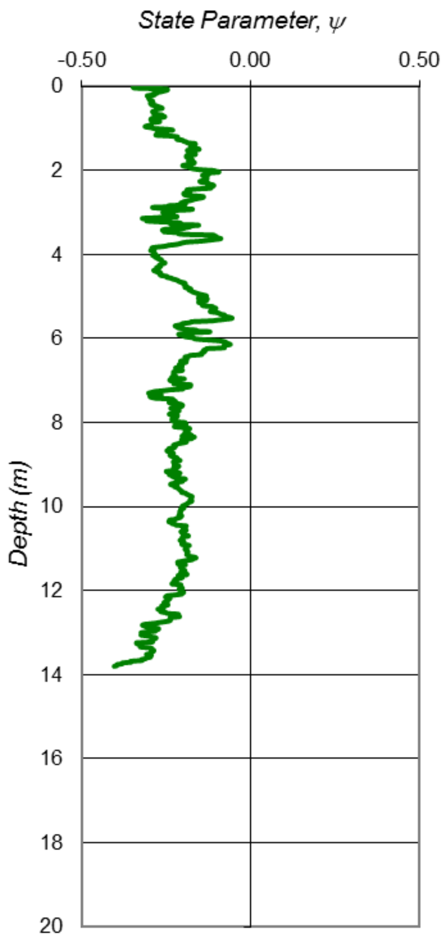

Figure 1. State Parameter Profile, CPT10

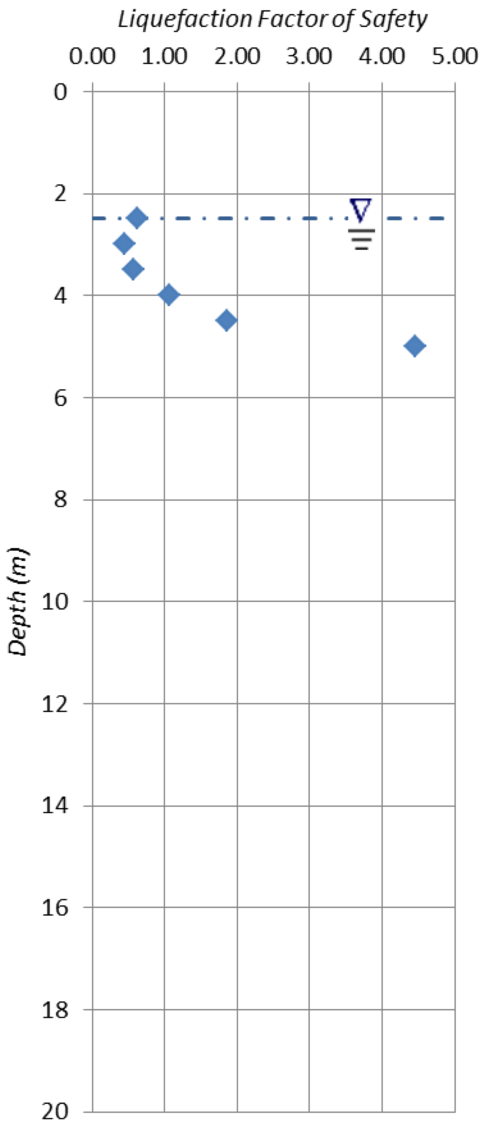

Figure 2. Liquefaction Factor of Safety from Shear Wave velocity Data for MASW 1, Ch10

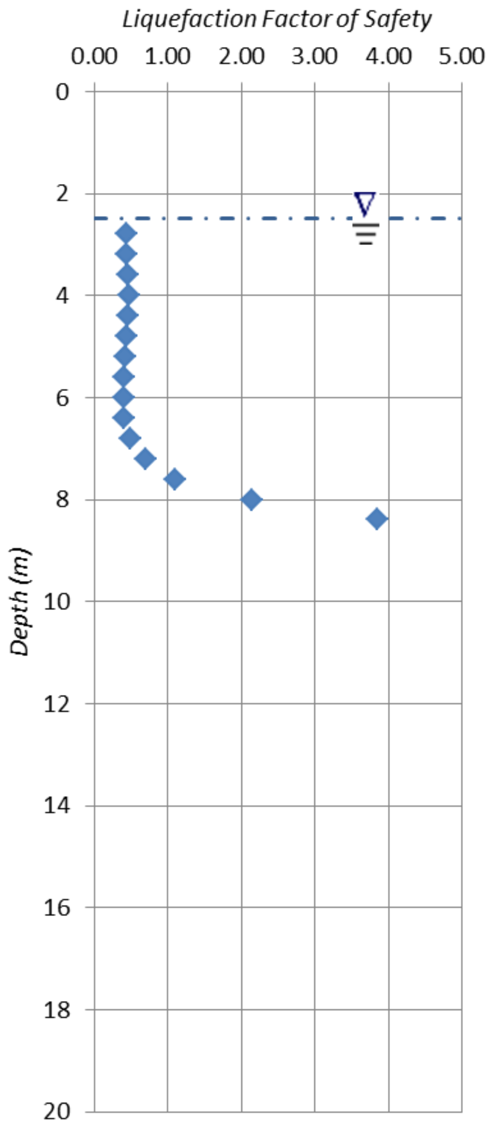

Figure 3. Liquefaction Factor of Safety from Shear Wave velocity Data for MASW 5, Ch12.30

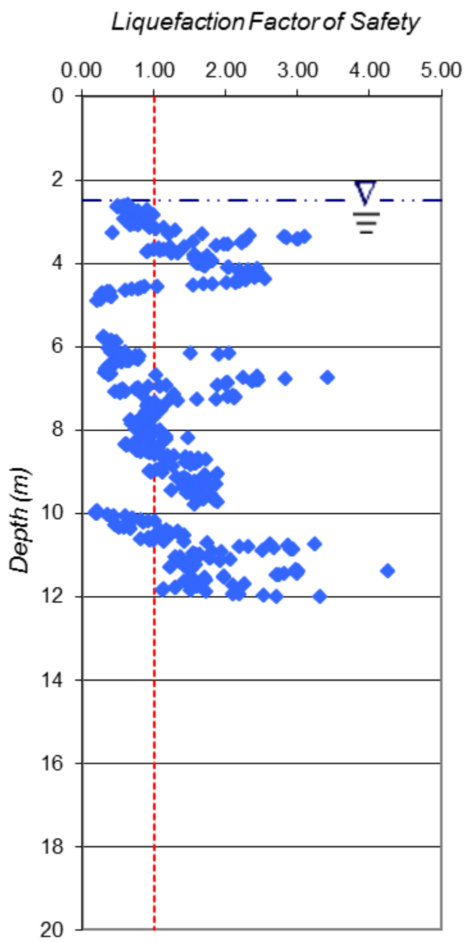

Figure 4. Liquefaction Factor of Safety from CPT11 using the Method of Robertson and Cabal

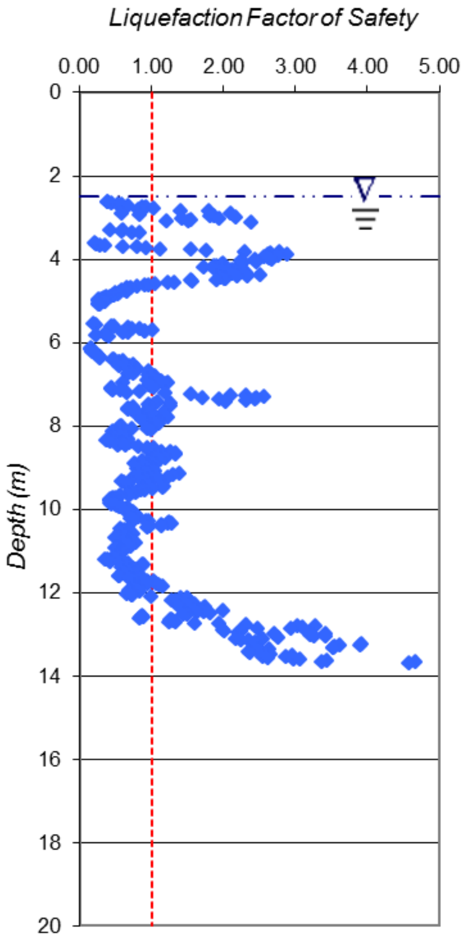

Figure 5. Liquefaction Factor of Safety from CPT10 using the Method of Robertson and Cabal