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27 June 2024

Farzad Zamani
Wellington City Council
PO Box 2199
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Dear Farzad

City to Sea Bridge Seismic Assessment Peer Review Summary

We are writing to summarise our peer review work of the structural and geotechnical aspects of seismic assessments undertaken for Wellington City Council (WCC) on the City to Sea Bridge.

The structural assessment of bridge was undertaken by Hoffcon and Tonkin & Taylor provided geotechnical engineering to inform the seismic assessment.

Scope of Review

The scope of our review consisted of the following:

- High level qualitative review of the findings of the Hoffcon structural assessment of the bridge.
- High level review of the findings of the Tonkin & Taylor geotechnical assessment for the bridge.
- Engagement with assessing consultants to understand assessment methodology and assumptions adopted.
- Detailed review of geotechnical assessment relied on by the structural engineer including an independent assessment of the geotechnical conclusions in relation to liquefaction and seawall stability.

We note our structural engineering scope did not include a detailed review of the structural assessment as it was high level and qualitative in nature. We have therefore not carried our any calculations of our own nor have we reviewed calculations carried out by Hoffcon.

City to Sea Bridge Review Findings

Based on our review of the both the geotechnical and structural assessment reports and discussions with the assessing consultants we note the following key observations:

- It appears that both the geotechnical and structural assessment of the bridge have been undertaken using appropriate methodologies as set-out in industry seismic assessment guidance.
- We agree with the Tonkin and Taylor assessment that the seawall is likely to move in the event of liquefaction of soils behind and in front of the toe of the seawall. The magnitude of the movement is difficult to quantify but we consider the order of magnitude remains significant for the bridge based on our review the structural assessment.
- Given agreement with the above point the methodology and conclusions on the structural capacity of the bridge outlined in the Hoffcon report appear reasonable. The assessment appears to have been thorough and considered a number of different load cases considering both liquefied and unliquefied soil conditions to test the impact of the various ground states.

make everyday better.

- We consider that a viable retrofit solution may be possible via the installation of a new deadman/retaining wall system within the current traffic lanes under the bridge that would reduce the displacements to a more manageable level and enable mobilisation of the central pier piles. This would likely also need to include ground improvement such as in-ground walls forming lattice cells that isolates the piles from ground movements, which could be formed by overlapping jet-grout columns down to the underlying dense alluvium. With more manageable displacements on the bridge piles, we believe the beam-column joint issue identified by Hoffcon could potentially be considered to not represent a significant life safety hazard as gravity support would be unlikely to be lost at smaller displacements. This would allow the bridge to achieve a higher rating without retrofit of the joints.

A full summary of our geotechnical review is included in the enclosed geotechnical file note if you wish to understand further detail in relation to our geotechnical review.

In summary both the geotechnical and structural assessments appear to have been carried out generally in accordance with industry guidance and the findings of the reports appear to be reasonable.

We trust this letter is satisfactory. If you have any questions, please contact the undersigned.

Yours sincerely

on behalf of
Beca Limited

Encl: File Note - City to Sea Bridge DSA Review

File Note

By:		**Date:**	16 April 2024
Subject:	File Note - City to Sea Bridge DSA Review	**Our Ref:**	3190846

1 City to Sea Bridge DSA Review

1.1 Introduction

Beca have been engaged by Wellington City Council (WCC) to review the Detailed Seismic Assessment (DSA) undertaken for the City to Sea Bridge. A structural assessment was undertaken in late 2023 by Hoff Consultants Ltd (Hoffcon). It was informed by outputs from a Desktop Geotechnical Seismic Assessment undertaken by Tonkin & Taylor Ltd (T+T). Reference has been made to the Engineering Assessment Guidelines (2017).

1.1.1 Reports provided for review

The Following reports have been provided by WCC for Beca to review:

- Hoffcon (2023) City to Sea Bridge. Detailed Seismic Assessment.
- T+T (2023) City of Sea Bridge, Wellington, Desktop Geotechnical Seismic Assessment.

T+T also provided the following information on the ground model:

- Te Ngakau Civic Precinct Geotechnical Hazard Review, Oct. 2023.

1.1.2 Summary of key findings of the DSA

Key findings of the Structural DSA were documented as follows:

- Hoffcon report that the City to Sea Bridge has an assessed seismic performance rating (%NBS, New Building Standard) of 20% NBS (IL3 structure).

  "The governing factor that has determined this rating is the step-change in the seismic response of the bridge and surrounding ground when liquefaction is triggered. This includes phenomena such as lateral spreading and cyclic displacement of the ground, in addition to movement of the underlying seawall. However, there are other structural weaknesses that would see a rating less than 34% NBS even if liquefaction was not triggered."

- A rating less than 34% NBS indicates it is an earthquake-prone building (EPB) in terms of the Building Act 2004.
- Options discussed by Hoffcon include demolition, strengthening, or risk management.

Key findings of the T+T assessment were as follows:

- Liquefaction triggering hazard:
  - Widespread liquefaction triggering of reclamation fill and beach deposits at ~ PGA of 0.2g (M 7.1) to 0.25g, approximately corresponding to 35-40% NBS.
  - Underlying Alluvium not considered to liquefy uniformly, with pockets only within the upper-part of the layer at ~ 0.3 to 0.4g, M7.1 (50 – 70% NBS).

- Liquefaction impacts:
    - Lateral spreading: Sea wall failure and movement 100's mm to metres towards Whairepo lagoon. Sea wall unstable (sliding & rotation) in an earthquake event triggering "widespread liquefaction". Unstable with retained liquefied soils and no inertia.
        - Kinematic (ground displacement) loads on piles.
        - Differential lateral spread across the bridge (lateral stretch)
    - Cyclic ground displacements: In the order of 150mm in any direction.
        - Kinematic (ground displacement) loads on piles.
    - Loss of vertical (compression and uplift) and lateral support to foundations during shaking.
    - Free field settlement in the order of 100-200mm, with additional settlement due to sand boils.
        - Negative skin friction (down drag loading) on piles post-shaking.

2 Ground Model

2.1 Soil profile

2.1.1 Available information

The relevant available geotechnical investigation information is summarised below, and as shown spatially relative to the bridge in Figure 2-1.

- Boreholes are provided along the alignment of the Sea wall / Eastern Abutment: BH-A1, BH-A2 and BH-A3. (completed in 1993)
- Borehole near the western abutment/ Capital E building (completed 1988): CSD-B9.
- Nearby CPT, north of western abutment, such as: CPT_156035, CPT_156021

2.1.2 T+T Ground Model

The T+T ground model adopted for the assessment is as shown in Figure 2-2. The main geological units are:

- Reclamation fill (placed in 1889), comprising silty/sandy GRAVEL with occasional lenses of SILT. Generally medium dense/ dense above groundwater table, and loose below groundwater table.
    - Considered susceptible to liquefaction below groundwater table.
- Beach Deposits (Holocene), loose to medium dense SAND with shells.
    - Considered susceptible to liquefaction.
- Alluvium (Pleistocene), typically silty sandy GRAVEL with lenses of sandy SILT.
- Average groundwater level +0.4m RL (varies -0.2 to +1m). A level of +0.2m has been adopted for the assessment as more critical.

File Note

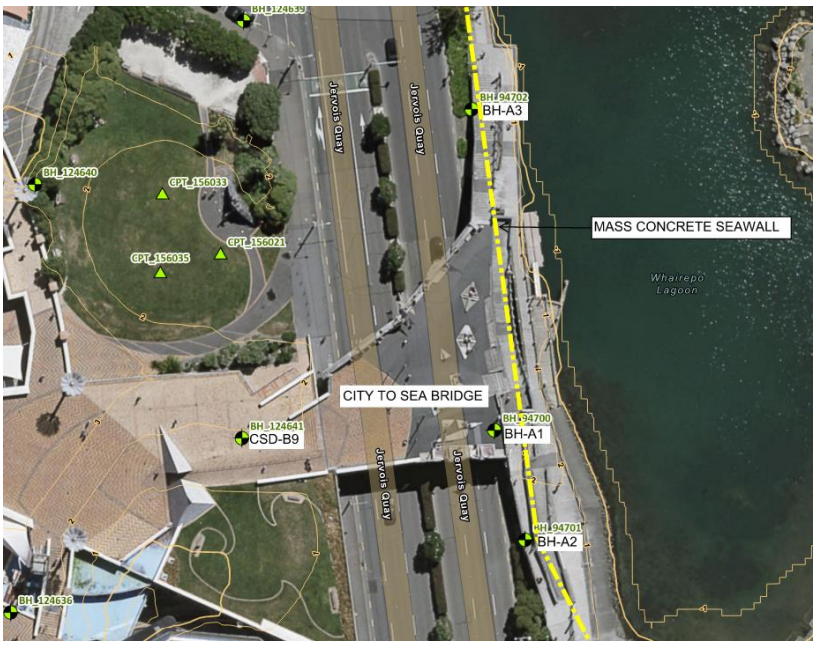


Figure 2-1: Ground Investigation location plan: Source: T+T City to Sea Bridge Geotech Report.

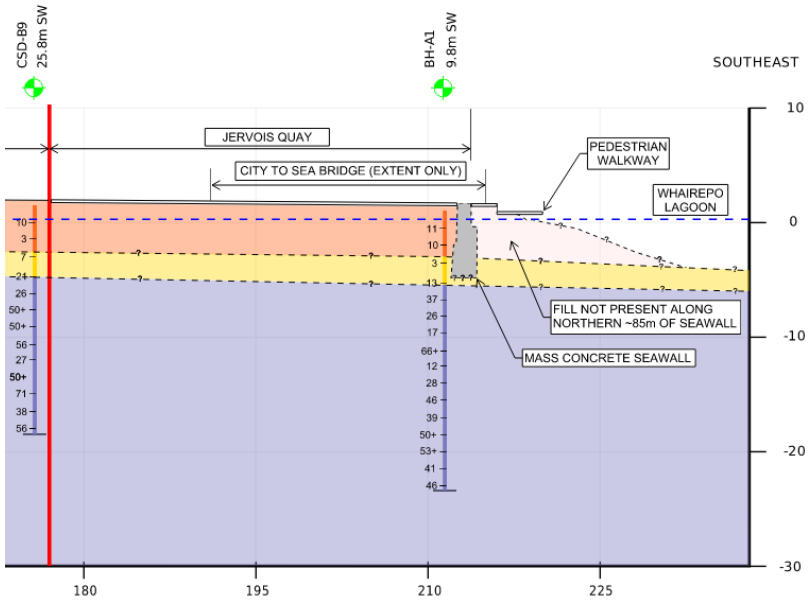


Figure 2-2: T+T Ground model excerpt.

In general, we concur with this ground model as summarised by T+T, however note comments related to the founding of the sea wall and assessed level of toe support in Section 3 below.

2.2 Shaking hazard for geotechnical assessment

The assessment of existing buildings adopts MBIE Module 1 (2016). This document provides the following parameters for geotechnical assessment:

- Site Class: C
- Importance Level 3.

- Peak Ground Acceleration: 0.59g
- Moment Magnitude: 6.7 (using the charts in MBIE Module 1, Note that Table C6.1 Bridge Manual conservatively provides M 7.1 for Wellington, which is what T+T have adopted).

We generally concur with the values T+T have adopted, while noting M7.1 is slightly conservative for the DSA when compared to the values provided in Module 1. This is not expected to have a significant impact on the assessment.

The adoption of IL3 for the assessment might be a matter of discussion, since the bridge has been assessed for importance using the Bridge Manual, not AS/NZS1170.0:2002. The Bridge Manual assesses importance level based on the assigned route priority. In terms of life safety alone, the importance level could potentially be assessed as IL2. If this were considered, the PGA for the assessment would reduce to 0.45g.

T+T refer to recent historical earthquakes, notably the M 7.8 Kaikoura (Nov. 2016) earthquake, where a PGA of 0.16g was recorded at the nearby Frank Kitts Park. They proposed normalising to M7.1 to compare to estimate a PGA of 0.18g (via magnitude scaling factor of Boulanger & Idriss (2014), presumably adopting an SPT N of ~ 15), approximately corresponding to 30% NBS (IL3) or 40% NBS (IL2).

2.3 Liquefaction triggering assessment

To check the assessed %NBS we have checked the triggering of “extensive” liquefaction in nearby boreholes. “Extensive” liquefaction is considered to result in triggering of both the saturated reclamation fills and the underlying loose marine sediments, such that the Sea Wall is no longer stable (see below assessment). This requires triggering of 3.5m of soil or more.

We have reviewed the triggering of the nearby boreholes and note that ‘extensive’ liquefaction is predicted in the reclamation fill (as well as the underlying beach alluvium) during the 2016 Kaikoura earthquake. This initially assumed the historical boreholes had an SPT hammer energy efficiency of 60%. Only minor triggering was noted to have occurred near the lagoon during the event. Adopting 70%+ resulted in a reduced extent of triggering of the reclamation fill notably in borehole BH-A3 and was therefore adopted for all subsequent assessments. This remains within the range suggested for both safety and automatic trip hammers (Idriss & Boulanger 2008).

File Note

Table 1: Summary of liquefaction triggering assessment on relevant borehole data

Borehole	Thickness of Liquefaction in 2016 Kaikoura Earthquake [m] (PGA - 0.16g; Mw 7.8)	'Calibrated' SPT Hammer Energy [%] to prevent Extensive triggering during Kaikoura event	"Extensive" Triggering PGA [g] (Seawall instability)	"Extensive" Triggering PGA [g] (Pile Instability)	Thickness of Liquefaction in 'NBS' Earthquake (IL3) [m] (PGA – 0.59g; Mw 6.7)
BH-A1	2.1m (4 – 6.1m)	60%+	0.28g	0.26g	4.9m
BH-A2	2.8m (3.8 – 6.6m)	60%+	0.31g	0.22g	4.1m
BH-A3	1.6m (5.4 – 7.0)	70%+	0.22g	0.22g	5.1m
CSD-B9 [1]	3.8m (2.4 – 6.2)	70%+	0.17g	0.17g	>4.65m

[1] There is a lack of reliable SPT test data in CSD-B9 between the depths of 5.0m and 13.0m that can be used for assessment, which may have overall effects on the results. The results presented are based on the available information.

3 Sea Wall Stability

The stability of the gravity wall was assessed using software GEO5. The wall was assessed to be stable under static conditions in both sliding and overturning.

It is considered reasonable to expect the wall would have been founded on competent soils (i.e., medium dense to dense) that are not likely to liquefy/ flow under moderate shaking caused by the Kaikoura earthquake. A friction angle of 38 deg has been assumed for this founding soil.

We consider the toe support provided by lagoon marine sediments to be only present under static conditions. Under moderate to strong shaking this toe support may be largely lost due to liquefaction.

The wall was found to become unstable with more than 3.5m of liquefied soils in the backfill, and with liquefied soils at the toe. The PGA corresponding to this point of instability was estimated for each of the nearby boreholes and ranged from 0.22 to 0.31g due to variability in the SPT *N* values of the reclamation fills. The weakest is BH-A3 at the northern end of the bridge. An average value of 0.27g might be considered reasonable to adopt for a 'best estimate' appraisal of the triggering point for wall instability.

Large displacements are expected to occur once the wall loses stability due to liquefaction of backfill and toe soils. As a yield acceleration cannot be determined when the soils are liquefied, only empirical lateral spreading evaluation methods (based on regression analyses of case history data) are available which are highly approximate. The Youd et al. (2002) method estimates in the order of 2.7m displacement at the free face under the 100% NBS event, with a level of accuracy of 50% - 200%. This is unlikely to occur at the triggering level, however.

While higher than estimated by T+T, with step-change factoring applied, would continue to result in a %NBS score less than 34%.

File Note

4 Ground Oscillation Displacements

Ground oscillation may be estimated using the chart in Tokimatsu & Asaka (1998). Assuming cyclic shear strains of 4% in loose soils, and “extensive” triggering corresponding to more than 115mm of lateral ground displacement (corresponding to pile/ joint failure), the thickness of triggering this corresponds to is around 2.9m or more. The PGA required to initiate this amount of triggering is 0.22 to 0.26g across the three boreholes, with an average of 0.23g, or 20% NBS with step change applied (IL3) or 25% NBS (IL2).

These values broadly concur with the T+T assessment.

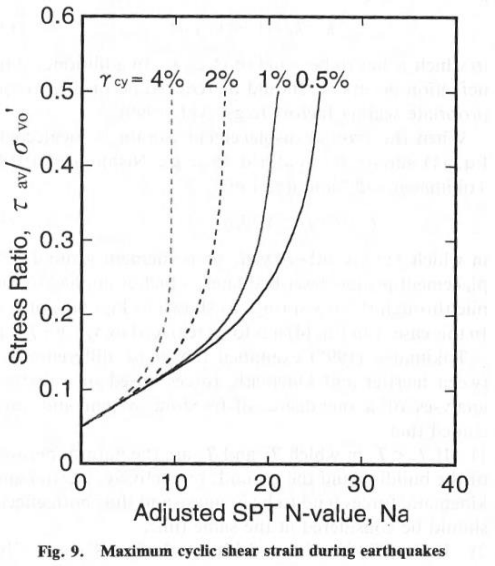


Figure 4-1: Cyclic ground strains as a function of Cyclic stress ratio and density of soil as determined by SPT N value. (Tokimatsu & Asaka, 1998).

5 Summary

In general, our parallel checks of the nature of the ground conditions and their likely performance during design levels of earthquake shaking concur with those of T&T. The impact of liquefaction triggering of the backfill and toe support for the gravity sea wall is significant and leads to instability without inertia applied to the wall. Once stability is lost, large displacements are expected. The accuracy of the assessed displacements when this occurs is fairly low, but the order of magnitude remains significant for the bridge. The application of step-change provisions for such a scenario in Section C4 of the Seismic Assessment Guidelines is considered appropriate.

Even were the wall to remain in place and not move laterally, cyclic ground displacements (oscillations) during shaking, due to liquefaction of the reclamation fills surrounding the piles, would be in the order noted to cause loss of gravity support for the bridge.

On the basis of this assessment, we therefore take no exceptions to the geotechnical assessment undertaken by T&T and the implications that follow to the Bridge, as documented by HoffCon.