SEISMIC STRENGTHENING OF THE TOWN HALL BUILDING IN WELLINGTON, NEW ZEALAND - GEOTECHNICAL

A. Riman¹ and S. Palmer²

ABSTRACT

The Wellington Town Hall is a heritage building located within the Wellington CBD, New Zealand. The Wellington City Council has decided to preserve this earthquake prone building by strengthening it to meet the New Building Standards (NBS). In the options identification and evaluation stage four main options were considered by the project team (building owner, architect, structural engineer, geotechnical engineer and cost estimator). The base isolated substructure founded on screw piles was selected as the option to proceed with to the next design stages with an overall estimated strengthening cost of $90 Million NZD (around $60M USD). The main design risks identified for a level of shaking of 0.65 PGA (M 7.1) are liquefaction, cyclic softening, lateral spread and ground heave under basements. Construction risks are also critical and were considered. This paper discusses the options identification and evaluation from a geotechnical perspective, lightly touching on some structural and architectural aspects that impacted geotechnical related decisions.

¹Senior Geotechnical Engineer at TONKIN & TAYLOR Ltd, New Zealand (email: ARiman@tonkintaylor.co.nz)
²Principal Geotechnical Engineer at TONKIN & TAYLOR Ltd, New Zealand (email: SPalmer@tonkintaylor.co.nz)

Seismic Strengthening of the Town Hall Building in Wellington, New Zealand – Geotechnical

A. Riman¹ and S. Palmer²

ABSTRACT

The Wellington Town Hall is a heritage building located within the Wellington CBD, New Zealand. The Wellington City Council has decided to preserve this earthquake prone building by strengthening it to meet the New Building Standards (NBS). In the options identification and evaluation stage four main options were considered by the project team (building owner, architect, structural engineer, geotechnical engineer and cost estimator). The base isolated substructure founded on screw piles was selected as the option to proceed with to the next design stages with an overall estimated strengthening cost of $90 Million NZD (around $60M USD). The main design risks identified for a level of shaking of 0.65 PGA (M 7.1) are liquefaction, cyclic softening, lateral spread and ground heave under basements. Construction risks are also critical and were considered. This paper discusses the options identification and evaluation from a geotechnical perspective, lightly touching on some structural and architectural aspects that impacted geotechnical related decisions.

Introduction

Wellington City Council (WCC) evaluated options and developed a concept for the seismic strengthening and re-development of Wellington Town Hall (WTH) in 2016. This paper discusses the options’ identification evaluation from a geotechnical perspective, lightly touching on some structural and architectural aspects that impacted geotechnical related decisions.

Wellington Town Hall

The Wellington Town Hall (WTH) is a heritage building designed by Joshua Charlesworth in 1900 and constructed between 1902 and 1904. The land on which the WTH was built is reclaimed land created using fill from the nearby cliff-face on Lambton Quay and hauled by horse and cart. In 2007 WTH was assessed as earthquake-prone. It was assessed to have a seismic stability of less than 33% of New Zealand new building standard (NBS). In November 2013 the WTH was closed to start the earthquake strengthening design / works.

¹Senior Geotechnical Engineer at TONKIN & TAYLOR Ltd, New Zealand (email: ARiman@tonkintaylor.co.nz)
²Principal Geotechnical Engineer at TONKIN & TAYLOR Ltd, New Zealand (email: SPalmer@tonkintaylor.co.nz)

The Wellington Town Hall site is relatively flat, bounded to the south by Wakefield Street and to the north, west and east by existing buildings. The reclamation edge and Wellington harbor is located 100m east of the site (refer Fig.1).

![Figure 1. The Wellington Town Hall location](image)

The existing WTH structure is supported on isolated caps and beam caps bearing on unreinforced concrete piles (Figs. 2 and 3). A basement underlies the south eastern portion of the building. Several modifications to the foundations have been introduced since construction.
Figure 2. A 1901 plan showing the existing foundation layout of WTH substructure [1].

Figure 3. A 1901 section showing the existing foundations and superstructure of WTH [1].
Geotechnical Challenges

This section discusses the geotechnical challenges identified as a consequence of the ground seismic response.

Geology and Soil Profile

The site is located approximately 100m from the current shoreline of the Wellington Harbor (Fig. 1). The geological sequence underlying the site comprises Reclamation Fill overlying shallow marine (Holocene age) Beach Deposits, which in turn overlie alluvial deposits, Wellington Alluvium (Pleistocene age). Reclamation Fill was placed in this location along the Wellington waterfront in a number of stages and includes quarried rock fill, dredged sediments, hydraulic fill and demolition materials [2]. The depth to rock across the site is estimated to be approximately 40m to 60m below ground level [3].

The Wellington Fault is located approximately 1.4km northwest of the site. The Wellington Fault is the most notable active fault in the area where it can be traced from Wellington’s south coast through to the Wairarapa. There are numerous other active and inactive faults mapped across the region that may represent a hazard to the site.

The inferred general soil profile at the WTH site is described in Table 1 and Fig. 4

Table 1. The inferred general soil profile

<table>
<thead>
<tr>
<th>Layer</th>
<th>Layer</th>
<th>Description</th>
<th>Average Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1*</td>
<td>Reclamation Fill</td>
<td>GRAVEL with areas of silty SAND, loose</td>
<td>4m thick; 2m of which below groundwater level</td>
</tr>
<tr>
<td>2</td>
<td>Beach Deposits</td>
<td>Shelly SAND with trace silt. Thin lenses of fine to medium gravel. Medium dense, locally loose.</td>
<td>1m thick</td>
</tr>
<tr>
<td>3</td>
<td>Upper Alluvium</td>
<td>Interbedded: SILT beds; SAND beds; and gravel lenses. Very dense, locally medium dense or very stiff.</td>
<td>5m thick</td>
</tr>
<tr>
<td>4</td>
<td>Lower Alluvium</td>
<td>GRAVELs; Occasional clayey silt, silt and sand lenses. Very dense.</td>
<td>40m thick; Extends down to rock</td>
</tr>
</tbody>
</table>

*Groundwater is at around 2m below ground surface.
Liquefaction

For the WTH building to be strengthened to 100% of the New Zealand New Building Standard (NBS) an ultimate limit state shaking: PGA = 0.65g 7.5MW is to be considered.


Table 2. Liquefaction Potential, Extent and consequences

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Layer</th>
<th>Liquefaction potential and extent*</th>
<th>Potential Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Reclamation Fill</td>
<td>Below ground water level, continuous and widespread liquefaction possible within this layer.</td>
<td>Lateral Spread/cyclic displacement Settlement Reduced lateral support Reduced vertical support Buoyancy of buried vessels</td>
</tr>
<tr>
<td>Layer No.</td>
<td>Layer</td>
<td>Liquefaction potential and extent*</td>
<td>Potential Consequences</td>
</tr>
<tr>
<td>----------</td>
<td>---------------------</td>
<td>-----------------------------------</td>
<td>-------------------------------------</td>
</tr>
<tr>
<td>2</td>
<td>Beach Deposits</td>
<td>Localised liquefaction possible within occasional zones of loose materials within the beach deposit layer.</td>
<td>Settlement Reduced lateral support Reduced vertical support</td>
</tr>
<tr>
<td>3</td>
<td>Upper Alluvium</td>
<td>Possibility of Localised liquefaction cannot be discounted within occasional fine SAND lenses.</td>
<td>Reduced vertical support</td>
</tr>
<tr>
<td>4</td>
<td>Lower Alluvium</td>
<td>Liquefaction potential low</td>
<td>None</td>
</tr>
</tbody>
</table>

* The level of ground shaking that could trigger the liquefaction is assessed to be around 0.2 g to 0.3g M 7.5

**Options Identification**

When referring to the project team in this paper this includes the design team and the Client (WCC).

The project team participated in workshops to explore options for the re-development and seismic strengthening of the WTH. The workshops considered options for:

- Building use and associated requirements for development of space.
- Architectural and heritage preservation.
- Level of seismic performance.
- Structural engineering concepts.
- Geotechnical/foundation engineering concepts.

The main conclusions from the workshop influencing the geotechnical options evaluation and concept design included building use/architectural, structural engineering and Geotechnical/Foundation Engineering requirements. These are discussed in the subsequent sections.

**Building use/architectural**

The existing basement beneath the eastern portion of the existing building need not be preserved. A new basement is required beneath the auditorium (Western portion of the existing building).

** Structural Engineering**

The following two options were identified for evaluation:

- Base isolated satisfying 100% of new building standard (NBS).
- Non base isolated satisfying 67% of new building standard (NBS).
Geotechnical/Foundation Engineering

Table 3 summarizes the foundation options which were considered during the workshops.

The foundation options of; a) Screw piles and b) jet grouting, were selected for further evaluation in conjunction with both structural options of a) base isolated 100% NBS and b) non-base isolated 67% NBS.

Table 3. Foundation Options Considered.

<table>
<thead>
<tr>
<th>Option</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raft foundation on existing ground</td>
<td>WCC requires a minimum performance equivalent to 67% NBS but prefers 100% NBS. At the design level of earthquake shaking associated with 67% NBS, liquefaction of the underlying reclamation fill could be expected. Inadequate bearing capacity to support building loads. Discounted</td>
</tr>
<tr>
<td>Excavate weak ground and replace with basement</td>
<td>The project team brainstormed possible methods for construction of this option and considered its relative merits. The team concluded:</td>
</tr>
<tr>
<td></td>
<td>• To allow excavation and basement construction the existing building would require temporary support; underpinning. Jet grout or screw piles were identified as the most appropriate means of doing this. This temporary works construction was assessed to comprise a substantial portion of the construction of the permanent works for the alternative solutions, thus making the full basement option unlikely to have an overall lower cost.</td>
</tr>
<tr>
<td></td>
<td>• Basement over the western portion of the building is seen as valuable for the operation of the completed building. Basement over the eastern portion of the building would have no essential use for the operation of the building. Additional construction cost of an eastern basement could not be justified in terms of its utility.</td>
</tr>
<tr>
<td></td>
<td>• Basement construction compared to providing space above ground level is very expensive.</td>
</tr>
<tr>
<td></td>
<td>It was concluded that this option was unlikely to be lower cost than the alternatives being considered. Discounted</td>
</tr>
<tr>
<td>Reinforced concrete bored piles</td>
<td>Unlikely to be lower cost than screw piles. Previous costing information is available to check this conclusion. Screw pile costs was checked against bored pile costs. Discounted.</td>
</tr>
<tr>
<td>Screw Piles</td>
<td>Selected for evaluation.</td>
</tr>
<tr>
<td>Jet Piles (Steel tube in a grout body of 0.5m to 1m diameter)</td>
<td>Jet Piles can be expected to be more expensive than screw piles. Jet Piles can be constructed in a more confined space than screw piles. Jet piles were selected as an alternative to screw piles to be considered in locations where access constraints make this option more cost effective than screw piles. Not totally discounted; could be used in conjunction with screw piles.</td>
</tr>
</tbody>
</table>
Jet grout was selected for evaluation.

Compaction grouting was evaluated. Evaluation of site soils and their likely response to compaction grouting concluded that there was a moderate risk that compaction grouting would not be fully effective. Discounted.

Option Selection

Following the option identification exercise undertaken through multiple workshops, the following four options remained for further evaluation:

1. Base isolated 100% NBS, screw piles (100% NBS/SP)
2. Base isolated 100% NBS, jet grout (100% NBS/JG)
3. Non base isolated 67% NBS, screw piles (67% NBS/SP)
4. Non base isolated 67% NBS, jet grout (67% NBS/JG)

Pre-concept design and cost estimating work progressed for these four options. During the pre-concept design work it was assessed that it was not practical for the 67% NBS/JG to resist base shear loads. Subsequent costing work also indicated that this option was unlikely to be favored. 67% NBS/JG option was deleted.

The other three options were progressed to completion of pre-concept and cost estimating. The project team considered the conclusions of the pre-concept and cost estimating work and selected the 100% NBS/SP option to progress to concept design. Table 4 summarizes the evaluation of relative merits of the three options from a geotechnical engineering perspective, as assessed by the project team.

Table 4. Relative Merits of Foundation Options Evaluated.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>100% NBS/SP Base isolated screw piles</td>
<td>1.0</td>
<td>100% NBS Similar to jet grout.</td>
<td>Low to moderate. Main design challenges identified; lateral load capacity and ability of piles to penetrate dense ground.</td>
<td>Low to moderate. At least two established New Zealand contractors could do the work.</td>
<td>Low to moderate. Main risks identified; Access constraints, ability to penetrate dense ground, dewatering and ground contamination.</td>
</tr>
<tr>
<td>------------------------------</td>
<td>----------------</td>
<td>---------------------</td>
<td>------------------------------------</td>
<td>-----------------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>100% NBS/JG Base isolated jet grout</td>
<td>1.15</td>
<td>100% NBS Similar to screw pile.</td>
<td>Low to moderate. Main design challenges identified; lateral load capacity and variable vertical stiffness across the foundation system.</td>
<td>Moderate to high. Dependent on international tenderers. Limited New Zealand experience.</td>
<td>Low to moderate. Jet grout offers some benefits over screw piles with respect to dewatering risk.</td>
</tr>
<tr>
<td>67% NBS/SP Non base isolated screw piles</td>
<td>1.1</td>
<td>67% NBS The structural design unlikely to provide a higher % NBS rating.</td>
<td>Moderate. As per 100% NBS/SP but with lateral loads more than double in magnitude</td>
<td>Low to moderate. At least two established New Zealand contractors could do the work.</td>
<td>Moderate. As per 100 % NBS/SP but with almost double the number of piles.</td>
</tr>
</tbody>
</table>

* Relative Cost: Developed for the purpose of comparing options only.

The options were similar with respect to estimated cost; within the limits of the cost estimating accuracy.

Base isolation supported on screw piles was selected because it offered a similar performance to the alternative of base isolation on jet grout and the jet grout option presented a relatively high risk of tendered price.

**Main Residual Risks**

After selecting the 100% NBS base isolated screw pile option (Fig. 4), the project team compiled a register of identified risks. The main two risks identified for the screw piles were the ability to penetrate to recommended founding levels, and installation within the confines of the existing structure. Early screw pile trialing and specific geotechnical investigations were suggested for the next stage. More discussions with specialist contractors is also to be undertaken.
Conclusion

Often geotechnical engineering design input starts after the architectural form, and possibly the structural form, have been selected. As a consequence opportunities are missed to allow for site conditions in the selection of architectural and structural form. The consequences can include an unnecessarily expensive foundation form or a missed opportunity for structural or architectural form to benefit from aspects of potential foundation solutions.

This project included optioneering workshops involving the building owner, architect, structural engineer, geotechnical engineer and cost estimator, before even building use had been confirmed. This allowed a robust integrated evaluation of options. For example; a full basement offered benefits of removing potentially liquefiable soils and in establishing the base isolation plane across the building. The building owner and architects were able to consider the value of this space in terms of building use. Without this early input from geotechnical this opportunity would not have been considered.

Acknowledgment

The Authors would like to acknowledge the efforts of the Project Team which includes Wellington City Council (WCC), Holmes Consulting Group, Xigo, Beca, Dunning Thornton Consultants, Athfield Architects, Rider Levett Bucknall and Tonkin + Taylor. They appreciate having a client like WCC who value the importance of collaboration and knowledge sharing.

The Authors would also like to acknowledge the contribution to this paper and the efforts of the geotechnical team involved in the pre-concept and concept stage of WTH: James Munro, Emily Wright, Shirley Wang, Stefan Cook, Mike Jacka, Hayden Bowen, and Najla Kunhimon.

References

7. Boulanger RW, Moug DM, Munter SK, Price AB, dejong JT. Evaluating liquefaction and lateral spreading in interbedded sand, silt and clay deposits using the cone penetrometer. In5th International Conference on Geotechnical and Geophysical Site Characterisation 2016; 1 (pp. 5-9).