Providing resilience in a Wellington waterfront development

E Stocks
Tonkin & Taylor Ltd, Wellington
estocks@tonkintaylor.co.nz (Corresponding author)

A Riman
Tonkin & Taylor Ltd, Wellington
ariman@tonkintaylor.co.nz

S Palmer
Tonkin & Taylor Ltd, Wellington
spalmer@tonkintaylor.co.nz

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ABSTRACT

Wellington's waterfront is in demand for new developments due to its proximity to the central business district, infrastructure, and harbour. This area can be impacted on by several hazards like earthquake shaking, liquefaction and lateral spread, see level rise and tsunami (tsunami not considered in this paper). The waterfront was reclaimed in a number of stages beginning in the 1850s, mainly by end tipping of weathered gravels. Liquefaction and lateral spread of these gravels as a consequence of a strong earthquake shaking event is likely to occur as was observed in the recent M7.8 Kaikōura earthquake in November 2016. This paper presents a case study of a five-storey building currently being constructed on the Wellington waterfront. To mitigate the impact from some of the natural hazards and to provide a resilient structure various foundation and ground improvement systems were considered. An in-ground cellular foundation solution was selected. This solution offers: liquefaction mitigation, shear resistance to resist lateral spread beneath the building, foundations for the new structure, temporary basement walls and effective cut-off of ground-water flow during construction. The foundation selection process, features and associated risks are discussed in this paper.

1 INTRODUCTION

Since the 2011 Canterbury earthquakes and 2016 Kaikōura earthquake, 'resilience' is one of the main focuses while designing foundations and substructures on sites prone to liquefaction and lateral spread such as the Wellington waterfront.

A five storey building with a basement is currently being constructed on the Wellington waterfront. While the waterfront provides unique location and amenity advantages for developments, the ground conditions are very challenging and complex.

The building design includes base isolation to prevent the building's superstructure from absorbing earthquake energy, with ground improvement to support the base isolation system and provide a high level of resilience to earthquake damage.

This paper discusses the foundation options considered for this complex site, the basis of selecting the in-ground cellular foundation solution and its features and associated risks and hazards.

2 PROJECT INFORMATION

2.1 Proposed development

A five storey building with a basement beneath approximately 90% of the building footprint is currently being constructed on the Wellington waterfront (refer Figure 1 for site location). The building owner wanted to provide a high level of resilience under an extreme earthquake event, including a lower potential for damage in a severe event than a normal office building.

The building design includes base isolation to provide high seismic performance by reducing the extent to which the building's superstructure absorbs earthquake energy. To support the base isolation system and ensure this high seismic performance, ground improvement was needed to reduce liquefaction and lateral spread.

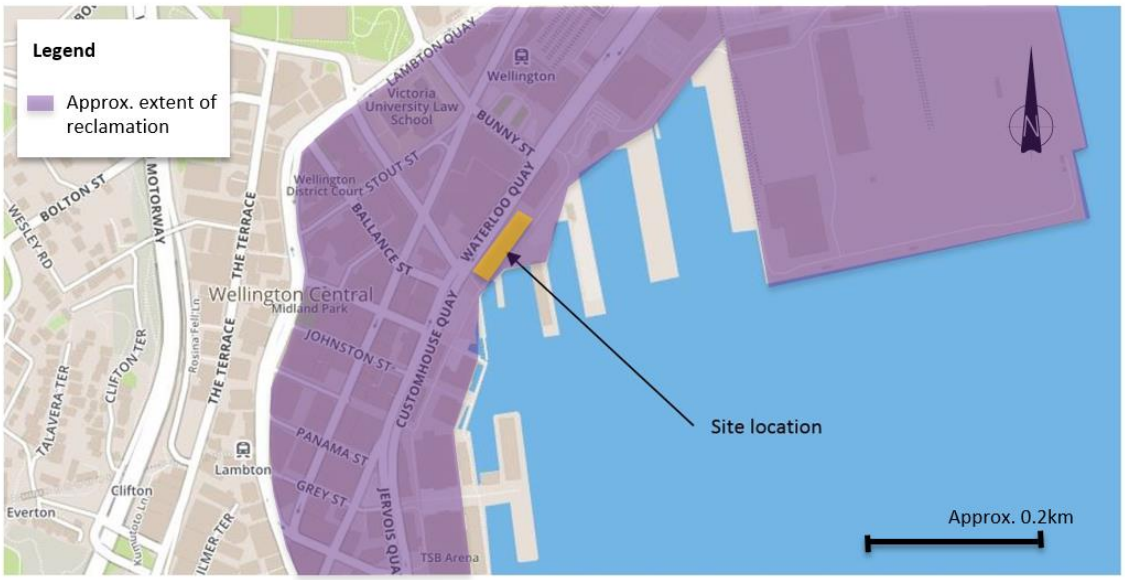

Figure 1. Site plan

2.2 Specific site conditions

The site is on reclaimed land on the Wellington Harbour foreshore. The reclamation fill is underlain by thin layer of recent marine deposits and Pleistocene alluvial deposits, which infilled the steeply graded valley (Begg & Johnston, 2000). Changes in sea level built up the alluvial deposits in layers of gravel, sand and silt. Greywacke bedrock underlies the alluvial deposits at more than 100m below existing ground level.

The original shoreline, which ran along Lambton Quay, is approximately 300m west of the site (refer Figure 1). Prior to 1876, the reclamation fills extended up to Waterloo Quay (Semmens et al. 2010). The land beneath the site was reclaimed in the early 1900s. As illustrated in Figure 2, a mass concrete seawall was constructed immediately to the south-east of the site which formed the edge of that reclamation. It is likely that the reclamation was formed by tipping materials excavated during roading and other construction work, predominately silty sandy gravels. In the early 1970s, a reclamation southeast of the site was constructed, supported by a sheet pile wall to the east and rock revetment to the south.

2.3 Seismic shaking hazard and liquefaction risk

The seismic subsoil class for the site is considered to be 'Class D – Deep or Soft Soil Sites' in accordance with NZS 1170.5:2004 (Standards New Zealand, 2004). An Ultimate Limit State (ULS) of 2500 year return period has been considered. Peak Ground Acceleration (PGA) of 0.62g and a corresponding earthquake magnitude of M_w 7.1 was derived from NZTA Bridge Manual (NZTA, 2016) following the recommendation in recent geotechnical guideline (NZGS, 2016).

Liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil (sands and non-plastic silts) during earthquake shaking. This causes the soil to undergo a partial to complete loss of shear strength. Such a loss of shear strength can result in settlement, bearing capacity failure and / or horizontal movement of the soil mass. Liquefaction of gravels within reclamation fills has also been observed following the earthquake in Kobe, Japan in 1995 (Cubrinovski & Ishihara, 2003; Hara et al. 2004; Hara et al. 2012), and the recent earthquake in Kaikōura in 2016 which greatly affected Wellington port land (Cubrinovski et al. 2017).

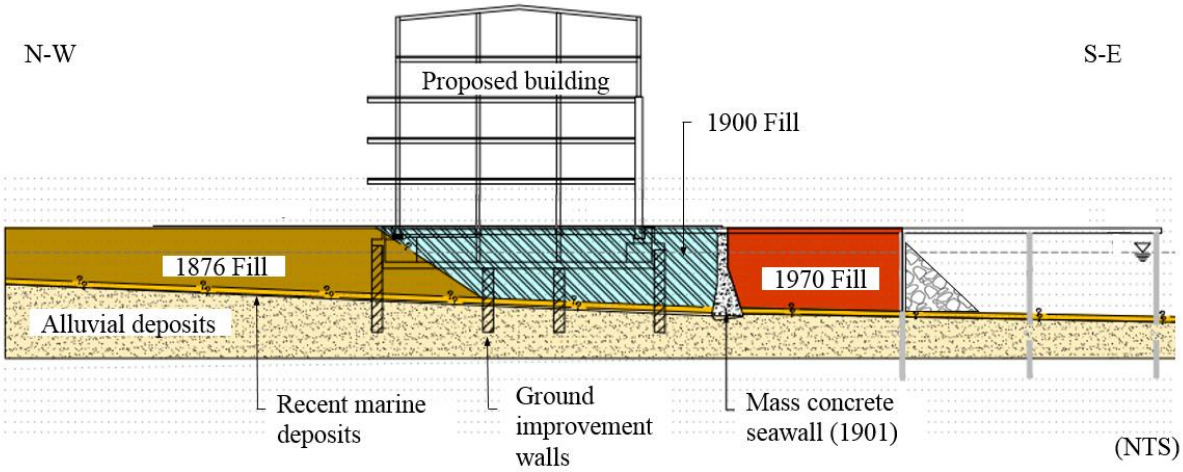

Figure 2. Cross section of reclamation fill and proposed development

Soils which are susceptible to liquefaction require a certain level of earthquake shaking (trigger) to cause them to liquefy. Analysis using the method proposed by Boulanger and Idriss (Boulanger & Idriss, 2014) concluded that as a consequence of a 150 year return period earthquake event (PGA 0.2g, magnitude M_w 7.1) liquefaction could occur within the reclamation fill and zones of liquefaction could occur within the marginal marine deposits. Due to the dense nature of the alluvium and the age of the deposit (Pleistocene >12000 years old) it was concluded that liquefaction of these soils is not likely but cannot be discounted as a consequence of severe earthquake shaking (>200 year return period).

Following the recent earthquake in Kaikōura, the site experienced greater level of shaking (PGA 0.23g, magnitude M_w 7.8). No liquefaction effects were observed at this level of shaking, however this cannot rule out the potential for liquefaction to occur under stronger shaking.

Liquefaction induced lateral spread is a movement of ground toward the free edge (i.e. sea) as a result of shearing of weak liquefied ground under seismic and / or gravity forces. It could be expected to occur in a series of scollops extending back from the reclamation edge with the magnitude of total displacement reducing with distance from the reclamation edge. As a consequence of a 500 year return period earthquake event (PGA 0.35g, magnitude M_w 7.1) lateral spread is expected.

2.4 Design objectives

The design objectives for the in-ground works were to:

- Retain life safety for a 2500 year event and limit building damage for a 1000 year event;
- Mitigate liquefaction beneath the building footprint and associated differential settlement of the building;
- Distribute building loads with depth and provide a relatively stiff base to found the building on;
- Provide adequate bearing capacity to support the building compression loads;
- Provide resistance to building seismic uplift loads;
- Mitigate basement buoyancy effects in the event of liquefaction and due to sea level rise over the lifespan of the building;
- Mitigate lateral spread potential beneath the building footprint and resist base shear from the structure;
- Provide lateral support to the basement excavation during construction; and
- Provide a cut off to groundwater to aid dewatering during construction.

2.5 Foundation options considered

At the concept design stage a broad range of foundation options were identified and discussed by the project team (the owner, the structural engineer and the geotechnical engineer).

- Option 1: Grid of gravel columns for ground improvement with bored belled piles to resist high, concentrated compression and tension loads;
- Option 2: Bored belled piles to resist tension, compression and lateral loads;
- Option 3: A deep foundation system comprising a grid of in-ground walls of secant deep soil mixing (DSM) or continuous flight auger (CFA). The in-ground walls extend through weak and potentially liquefiable reclamation fill and upper alluvium to support the building loads in the competent lower alluvium. Anchor piles at specific locations provide resistance to tension loads; and
- Option 4: Ground improvement comprising a cellular grid of in-ground walls to mitigate liquefaction of the reclamation fill and spread the building loads over the upper alluvial deposits. The in-ground walls created by secant DSM or CFA columns on which a concrete raft foundation is constructed. Anchor piles at specific locations provide resistance to tension loads.

The relative advantages and disadvantages of each option were considered. An option evaluation was undertaken in conjunction with the project team, and the robust system of CFA in-ground walls as a ground improvement system (Option 4) was identified as the preferred foundation option. A geotechnical risk register was developed and was updated throughout the design process.

The CFA construction method was selected over the DSM method due to the variable nature of the reclamation fill and the upper alluvium, which could compromise the effectiveness of the DSM method (i.e., DSM might not be able to achieve consistent cementation). The layout of the in-ground walls and a cross section is shown in Figures 3 and 4.

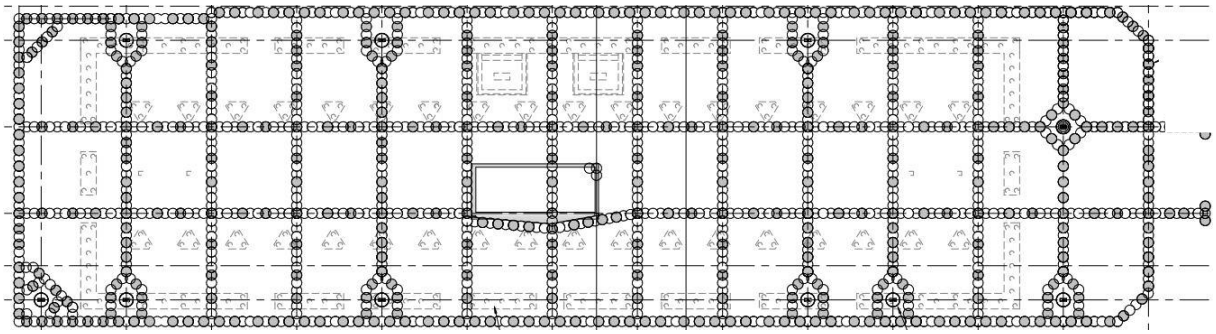

Figure 3. Plan of CFA in-ground walls foundation system

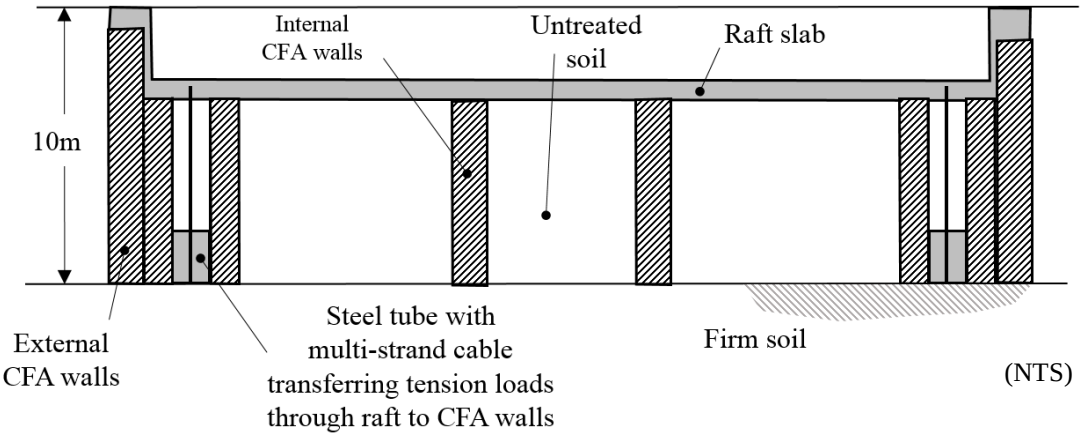

Figure 4. Cross section of CFA in-ground walls foundation system

3 CFA GROUND IMPROVEMENT

The CFA ground improvement meets all the design objectives listed in Section 2.4. In this section, we discuss in particular how this foundation system mitigates liquefaction and lateral spread, and resists uplift loads. An understanding of soil-structure interaction was achieved through collaborative approach between the project's structural and geotechnical engineers (Stocks et al. 2017).

3.1 Description of CFA ground improvement

At the Wellington waterfront site the ground improvement was constructed with secant CFA piles. Using hit one and miss one approach at the perimeter wall the unreinforced "soft" piles were formed first using lower strength concrete. The remaining piles were standard reinforced concrete "hard" piles constructed to overlap the "soft" pile by minimum 20% on both sides. The face of the piles following excavation is presented in Figure 5. The perimeter CFA piles created both a cut-off wall and a temporary basement retaining wall.

During construction the wall provided satisfactory performance of cutting of water flow. Ongoing groundwater monitoring outside the perimeter wall and movement monitoring at the top of the wall indicated little to no change during the construction period.

Figure 5. Hard and soft piles following basement excavation

Once the perimeter was completed, dewatering and bulk-excavation (to basement level) was carried out. The interior, cellular walls were then augered and cast from the excavated level, typically with a ratio of two "soft" piles to one "hard" pile.

At each tension pile location a ring of CFA piles was formed to help distribute the concentrated loads.

3.2 Mitigation of liquefaction

The cellular structure of in-ground walls mitigates liquefaction within the cells by:

- Resisting cyclic shear stresses from earthquake shaking to reduce cyclic shear strains in the soils within the cells to a level which mitigates their liquefaction; and
- Providing a barrier against the migration of high pore-water pressure from surrounding liquefied unimproved ground.

The layout of the in-ground walls to was developed using the method of Nguyen (Nguyen et al. 2013). The centre-to-centre spacing is typically 7.5m with maximum spacing up to 10.5m at selected locations.

The CFA ground improvement was designed to reduce the dynamic shear stresses and strains imposed on the enclosed saturated soils. This was expected to reduce the rate at which excess pore water pressures accumulate within the enclosed soil during severe earthquake shaking.
The lateral stability analysis did not rely on any shear resistance from the potentially-liquefiable marine deposits as the CFA piles were keyed into the underlying alluvial deposits, which are predominantly non-liquefiable.

Based on available research (e.g. Bradley et al. 2013), even though excess pore pressures develop to varying degrees within the enclosed soil, the lattice acts to limit loss of shear stiffness and significantly reduce the maximum shear strain, settlement and horizontal displacement.

3.3 Mitigation of lateral spread

Without ground improvement, lateral spread of the ground beneath building could be expected in a 500 year seismic event and is possible in lesser events. The improved ground is assessed to have adequate shear capacity to resist lateral spread beneath the building including kinematic / displacement loads from the ground landward of the proposed building, and including base shear from the building. The lateral spread assessment assumed the following:

- 65% of the peak building base shear;
- 100% of the lateral spreading load acting on the landward side; and
- Loss of support on the seaward side due to mass concrete seawall failure.

3.4 Buoyancy

The basement and structure has been designed for ground water pressure equivalent to water at ground level to account for possible sea level rise scenarios. The buoyancy forces were resisted by a combination of the following:

- Weight of the CFA walls;
- Weight of confined soils (friction on the CFA walls); and
- Weight of the superstructure.

3.5 Uplift resistance

Seismic uplift resistance is provided by connecting the secondary reinforced CFA piles with the substructure beams and basement walls. The seismic loads were resisted by the weight and shear resistance of ground improvement. Uplift resistance has been assessed considering:

- Buoyant weight of CFA piles;
- Lesser of shear resistance of soil against CFA piles and the buoyant weight of wedge of soil against CFA piles;
- Shear strength of potentially liquefiable soils is ignored. Soils outside the CFA system and over the bottom 1m of the CFA system were assumed to liquefy (The assumption that the Pleistocene Alluvial deposits will globally liquefy is rather conservative. In reality, due to the nature of these deposits and the reinforcement provided by the lattice, liquefaction is not expected except in some localised lenses, which is also unlikely); and
- Strength reduction factor of 0.5 on soil / CFA shear strength and 0.9 on soil / CFA pile weights.

3.6 Stiffness and bearing capacity

The gravity and serviceability limit state (SLS 1) loading cases did not require a detailed soil-structure interaction analysis. The improved ground was behaving as a relatively rigid block compared to the bearing pressures imposed by the substructure. The concentrated loads/ pressures were not high enough to unduly stress the improved CFA block. The loading stresses transferred to the bottom of the CFA block were less than 10% of the overburden pressure and resulted in minor differential settlements.

For the ultimate limit state (ULS) loading case, most of the soil-structure interaction analysis was carried out to design a substructure that would transfer large earthquake loads to the ground in a distribution that would not unduly stress the CFA improved block. Numerical analysis was undertaken in conjunction with the structural engineer to satisfy this design objective. Details of the soils-structure interaction analysis are provided in a separate paper (Stocks et al. 2017).

4 CONCLUSIONS

Nowadays, ‘resilience’ is one of the main focuses while designing foundations and substructures on sites prone to liquefaction and lateral spread such as the Wellington waterfront.

For the new development at the Wellington waterfront, the consideration of geotechnical hazards were taken into account from concept design stage through to construction. Understanding of the geotechnical hazards and their influence on seismic performance of foundations and the substructure was very important. For this project, it was achieved by collaboratively working together with the owner, the structural and geotechnical engineers and the contractor. The owner’s objectives for the building included a high level of resilience under an extreme earthquake event (2500 year seismic event) and a low level of damage in a severe event (1000 year seismic event).

A cellular ground improvement of CFA piles met these objectives and dealt with the geotechnical hazards by providing a stiff raft of soil-concrete composite. This solution offered: liquefaction mitigation, shear resistance to resist lateral spread beneath the building, foundations for the new structure, temporary basement walls, and effective cut-off of ground-water flow during construction.

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