In-ground cellular structure as a foundation system

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ABSTRACT: In-ground cellular structure has been used as a foundation system for a high profile new building on Wellington's waterfront. The owner of the development sought a design that provides a high level of resilience to earthquake damage, over and above typical requirements for Wellington and New Zealand. The design team evaluated various foundation systems. Cellular ground improvement was selected, because it provides a robust and resilient foundation that meets the owner's objectives for a high-performance structure. The grid of walls provided multiple functions: mitigation of liquefaction, shear capacity to resist lateral spread beneath the building, foundations for the new structure, and temporary basement walls and effective cut-off of ground-water flow during construction. This paper discusses the selection of the in-ground cellular structure solution, the features of this solution, the soil-structure interaction and associated structural / geotechnical collaboration.

1 INTRODUCTION

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 challenging and complex.

A key objective for the new building was to design in-ground works that provided both ground improvement and the foundation for the building superstructure. This was achieved through close cooperation and coordination between geotechnical and structural engineers.

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 selection of the in-ground cellular structure solution and its features, the soil-structure interaction, and the role of effective structural / geotechnical collaboration.

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 plan). 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.

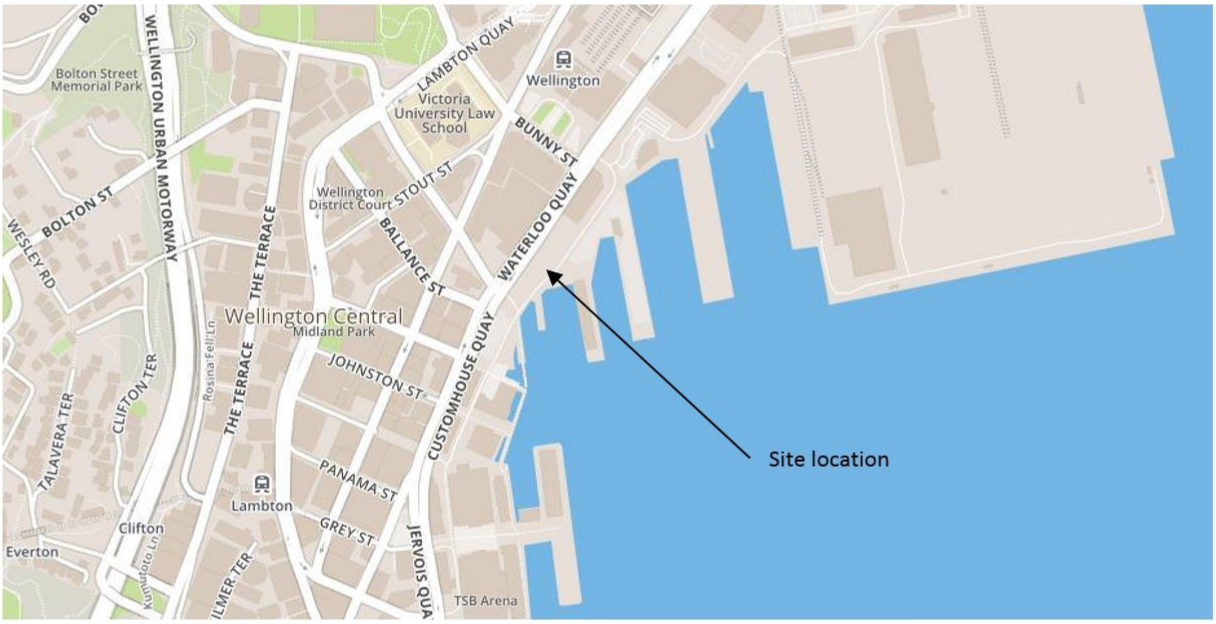

Figure 1. Site plan.

2.2 Specific site conditions

The site is on reclaimed land on the Wellington Harbour foreshore, underlain by alluvial deposits (Begg & Johnston 2000). Alluvial deposition processes transported material from the surrounding hills, and in-filled the steeply graded valleys of the inner Wellington Harbour during periods of elevated sea level. A thin layer of marginal marine deposits built up on top of the alluvium in very recent geological time. Greywacke bedrock underlies the alluvium at more than 100m below existing ground level.

The original shoreline, which ran along Lambton Quay, is approximately 300m west of the site. 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.

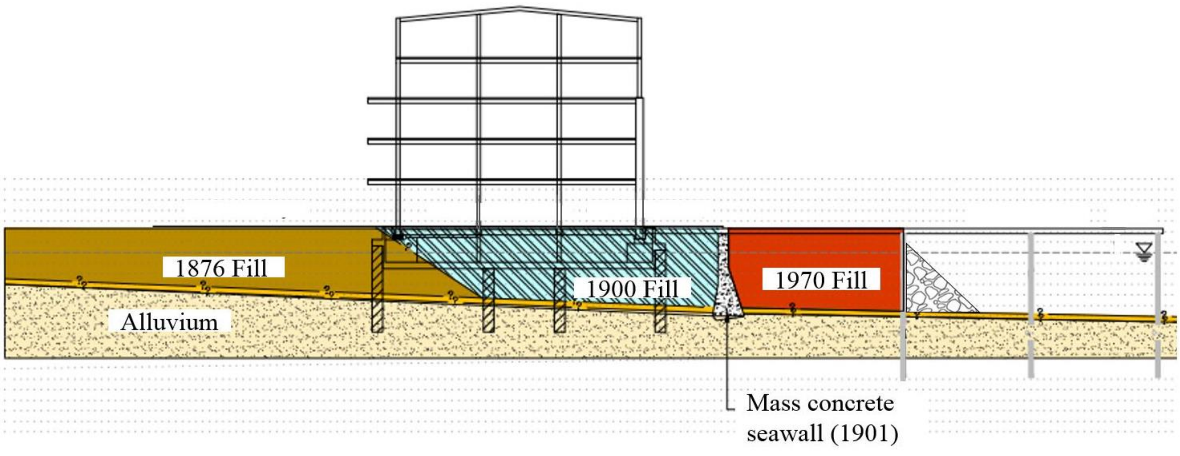

Figure 2. Cross section of reclamation fill and proposed development.

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 which equates to 0.62g Peak Ground Acceleration (PGA) and a corresponding earthquake magnitude of 7.1.

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. Liquefaction of gravels has also been observed (e.g. Kobe, Japan in 1995; Hara 2004). Such a loss of shear strength can result in settlement, bearing capacity failure and / or horizontal movement of the soil mass.

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 (2014) concluded that as a consequence of a 150 year return period earthquake event (PGA 0.2g, magnitude 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 >12,000 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).

2.4 **Design objectives**

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

- Mitigate liquefaction beneath the building footprint and associated differential settlement of the building;
- Distribute building loads with depth and provide a 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;
- Mitigate lateral spread potential beneath the building footprint and resist base shear from the structure;
- Provide lateral support to the basement excavation during construction;
- Provide a cut off to groundwater to aid dewatering during construction; and
- Retain life safety for a 2500 year event and limit building damage for a 1000 year event.

2.5 **Foundation options considered**

At the concept design stage a broad range of foundation options were identified 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: Grid of in-ground walls of deep soil mixing (DSM) or continuous flight auger (CFA) columns acting as underpinning. These in-ground walls provide a base for the concrete raft foundation. Anchor piles at specific locations provide resistance to tension loads; and
- Option 4: Cellular ground improvement created by 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. The layout of the in-ground walls and a cross sections is shown in Figures 3 and 4. The CFA construction method was selected 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).

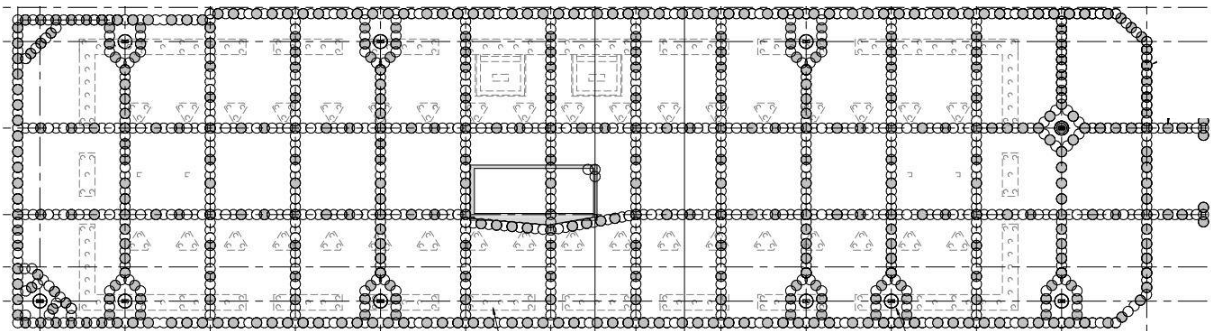

Figure 3. Plan of CFA in-ground walls foundation system.

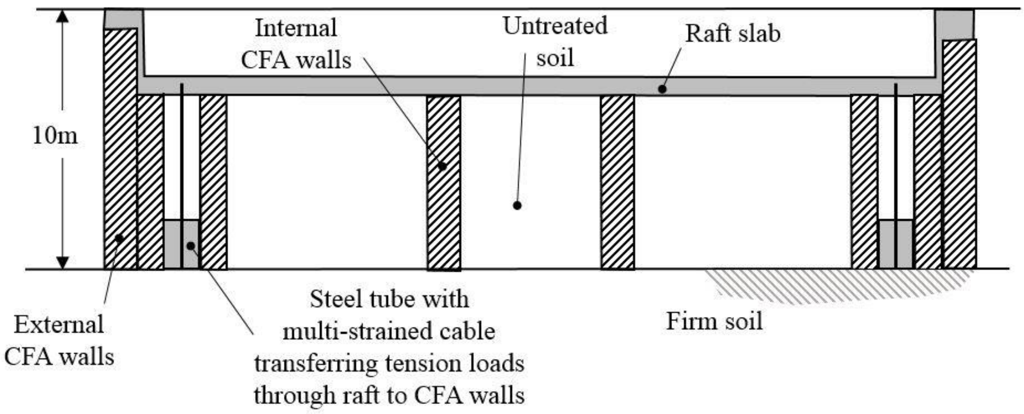

Figure 4. Cross section of CFA in-ground walls foundation system.

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. The collaborative approach to understanding of soil-structure interaction between the project's structural and geotechnical engineers is discussed.

3.1 Description of CFA Cellular Structure

A below ground cellular wall structure was constructed by using an overlapping/interlocking Continuous Flight Augur (CFA) technique. The perimeter was constructed first using a hit-and-miss methodology with unreinforced, low strength (soft) piles augered and cast initially and then a few days later the infill (hard) piles, with normal strength concrete and standard reinforcing cages, were cut into the soft piles. The perimeter CFA created both a cut-off wall and a temporary basement retaining wall.

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 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 et al. (2013). The centre-to-centre spacing is typically 7.5m with maximum spacing up to 10.5m at selected locations.

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.

3.4 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 1 m of the CFA system were assumed to liquefy; and
- Strength reduction factor of 0.5 on soil / CFA shear strength and 0.9 on soil / CFA pile weights.

3.5 Stiffness and bearing capacity

The gravity and serviceability limit state (SLS) 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 loads to the ground in a distribution that would not unduly stress the CFA improved block. The most critical parts of the CFA block are the corners, and this is where the value of collaborative work between structural and geotechnical engineers was realised.

The finite element model (FEM) analysis (refer Figure 5) used an idealised soil profile, in which the soil between the walls down to 2m above the base of the walls was modelled as not liquefied under the ULS. Soils outside the walls and down to 3m below the base of the walls was modelled as liquefied. Soils at greater depth were modelled as not liquefied. This assumption that the Pleistocene Alluvium deposits will globally liquefy is rather conservative. In reality, knowing the nature of these deposits, liquefaction is not expected except in some localised lenses, which is also unlikely. Refer to Table 1 for a comparison between the liquefaction assumptions in the FEM versus the actual conditions of the ground.

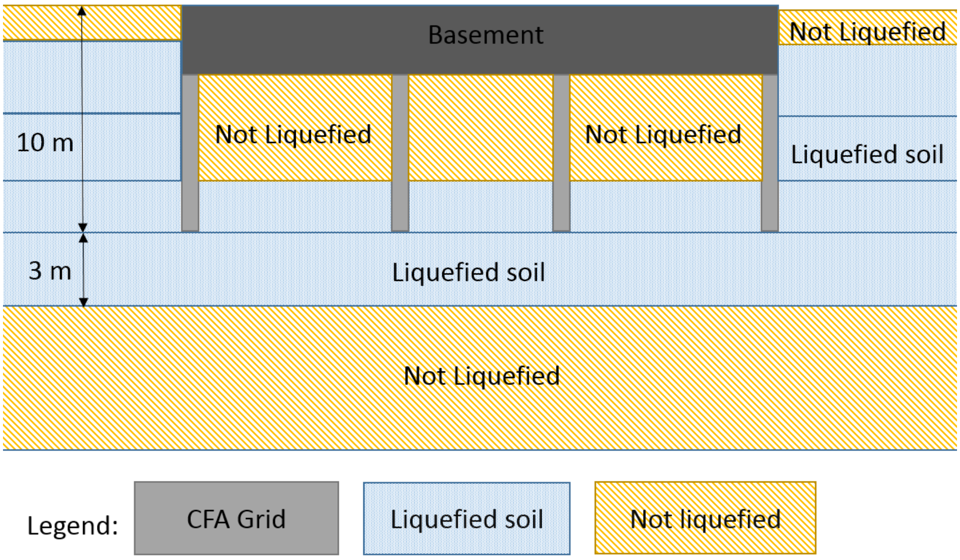

Figure 5. Assumed FEM ground liquefaction profile at ULS Shaking

Table 1. Actual Ground Conditions versus Model Assumptions

Description	**Actual conditions (real world) inferred from investigations**	**FEM assumptions relative to liquefaction (idealised)**
Reclamation Fill	The fill comprises mainly gravels. Below groundwater level, liquefaction could occur as a consequence of a 150 year return period earthquake event (0.2g, magnitude 7.1)	Liquefied soil
Marginal Marine Deposits	As a consequence of a 150 year return period earthquake event (0.2g, magnitude 7.1) extended zones of liquefaction could occur	Liquefied soil
Upper Pleistocene Alluvium	This layer generally comprises dense and very dense gravel and sand. Because of its dense nature liquefaction of the gravel and sand is not expected in a 2500 year ULS design earthquake event. Within the upper alluvium, lenses of medium dense silt of typically 0.5m thickness were encountered. This soil is marginal with respect to susceptibility to liquefaction. The alluvial deposit is relatively old (Pleistocene >12,000 years old). Liquefaction of these soils is not likely but cannot be discounted as a consequence of severe earthquake shaking (>200 year return period).	Liquefied soil
Lower Pleistocene Alluvium	Dense and very dense soil which are unlikely to be subject to liquefaction effects. Localised liquefaction of the occasional silt lenses as a consequences of severe earthquake shaking (>200 year event) is not likely.	Not liquefied

The stiffness and bearing capacity of the CFA ground improvement system was assessed as follows:

STEP 1: Initial geotechnical numerical analysis

Numerical modelling (PLAXIS 3D) of the soils and CFA ground improvement system was undertaken by the geotechnical engineer to provide an initial estimate of the vertical stiffness and limiting bearing pressure of the CFA system, i.e. stiffness of the support to the building's substructure (basement floor, ground beams and perimeter basement wall). Features of the model included point loads from the structure applied to beams in the 3D model. The bearing pressures and deflections beneath the beams and slabs were used to calculate stiffness values to be applied to the structural analysis (Step 2)

STEP 2: Structural analysis

Structural engineering models were created for both the superstructure and the in-ground works. The in-ground model considered the CFA walls together with the in situ concrete basement structure (walls, beams and raft slab) acting compositely. The structural engineer applied the vertical stiffness and limiting bearing pressure values from Step 1 to the structural analysis and design of the substructure. Structural engineering analysis included a sensitivity check on the stiffness and limiting bearing pressure values. Values of ½ and 2 times the values from Step 1 were considered. The structural engineer concluded this range of values did not make a material change to their structural design.

STEP 3: Geotechnical review of Step 2

The geotechnical engineer compared the structural analysis output of Step 2 (substructure bearing pressure and vertical displacement) with the output of the geotechnical model. The vertical displacements from the structural model (Step 2) were similar (within 20%) to those determined by the geotechnical model (Step 1). The ULS bearing pressures had a factor of safety against a bearing failure of greater than 3.

4 CONCLUSIONS

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 by providing a stiff raft of soil-concrete composite.

All building projects can benefit from effective collaboration between geotechnical and structural engineers. For this building, effective collaboration enabled a more complete understanding of the soil-structure interaction for a particular project and its impact on design. A collaborative approach offers project owners the opportunity to make well-informed decisions on a broader range of options to meet building performance objectives.

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