Push-over analysis of a 3D architectural wharf structure including soil-structure interaction

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ABSTRACT

A new wharf structure was proposed for Te Wānanga in central Auckland. Due to the complex geometry of the proposed wharf, which included plan irregularity, open apertures, deck-suspended planter boxes, suspended nets and a step down in level, a 3D finite element model was developed for the structure to undertake design. Push-over analysis that incorporated both modelling of non-linear hinges and the effect of non-linear soil-structure interaction was undertaken. This paper highlights the collaborative approach undertaken between the geotechnical and structural designers to achieve a practical and efficient design solution by detailing the design methodology and the technical challenges in the modelling approach.

The proposed Te Wānanga structure consists of bored reinforced concrete filled steel tube piles of varying founding depths supporting a reinforced concrete deck slab. The spatially varying founding depths result in significant variation in pile stiffness. To capture the non-linearity of the stress-strain relation of the soil as well as the effect of the spatially varying pile stiffness, a series of non-linear load-deformation (p-y) curves were developed for each soil unit along the depth of the piles. These p-y curves were then converted into uncoupled lateral non-linear springs to be input into the 3D structural model using multi-linear elastic link elements. By modelling the p-y curves in the 3D model, programme efficiencies were gained by reducing the number of design iterations of soil spring stiffnesses required between the structural and geotechnical engineers.

1 INTRODUCTION

As part of the Downtown Infrastructure Redevelopment Programme in Auckland, a new wharf structure, called Te Wānanga was proposed to provide new public space. Te Wānanga, which was designed to be a welcoming and aesthetically pleasing space for people, is located in Quay Street, Auckland Central. Te Wānanga consists of reinforced concrete-filled steel tube piles supporting a reinforced concrete deck which has both an irregular seaward edge and numerous irregularly shaped apertures within the extent of the deck (refer to Figure 1). These apertures create space for a number of architectural features; including deck-mounted suspended steel planters which hold large Pōhutukawa trees, woven suspended nets (Kupenga) and an open aperture with a sculpted weathering steel balustrade. The steel planters consist of flat plate elements welded together with both base and wall stiffeners and are cast into the deck using headed steel studs. The concrete slab was nominally a flat slab, but in fact consisted of a constant soffit level with a varying topping level which resulted in a deck thickness varying from a nominal 500 mm to 1000 mm with a 330 mm step down towards the western end.

The uneven distribution of mass due to the irregular geometry leads to an uneven distribution of load to the supporting piles. The length of the piles also varies as a result of the variation in geology, which contributes to an uneven distribution of lateral pile stiffness. Furthermore, slope stability analysis of the site, including the marine deposits and the proposed rock buttress closer to the Quay Street end of Te Wānanga, determined that there is potential for displacements under a design level seismic event, which would impart additional load onto the piles.

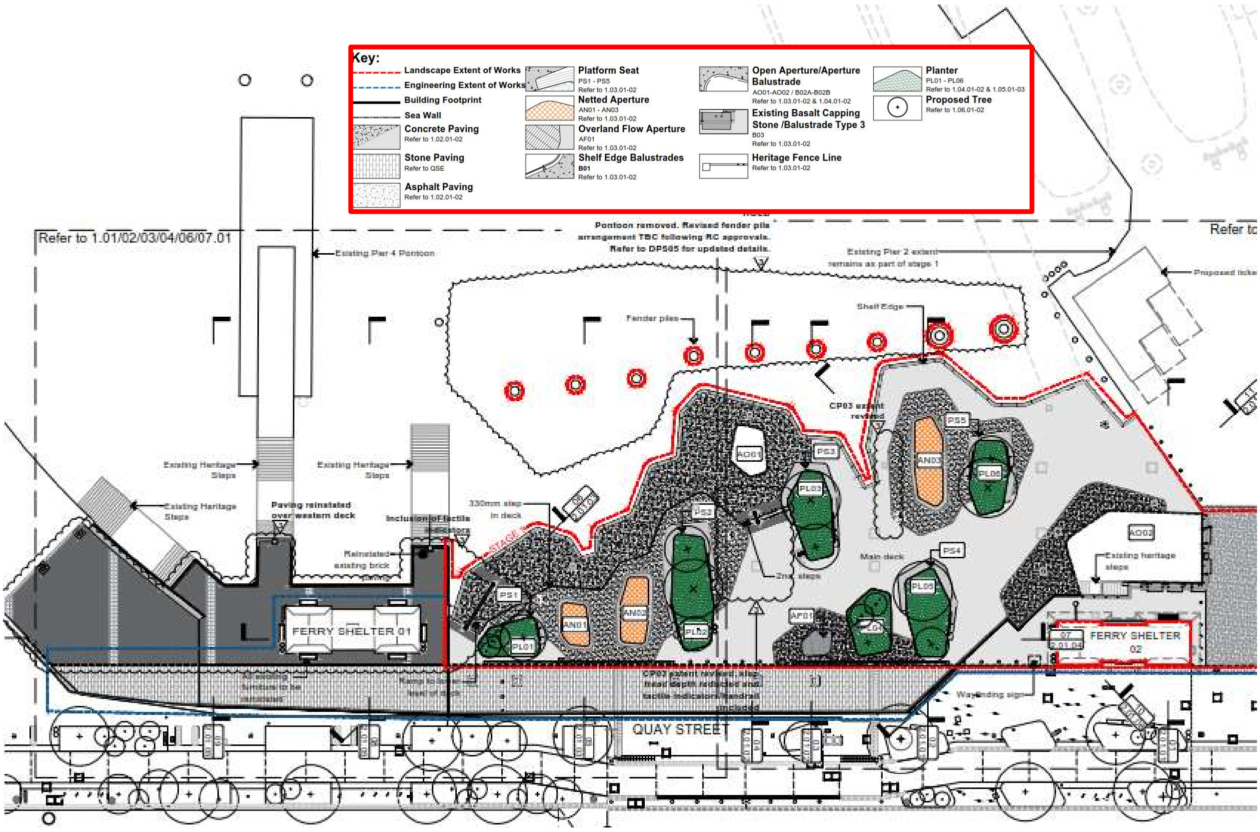

Figure 1: Te Wānanga Plan (Layout courtesy of Isthmus Group Ltd)

Due to these interconnected issues between structural and geotechnical design, a collaborative design approach between the structures team and the geotechnical team was essential to ensure an efficient delivery of the project. The team decided to develop a 3D finite element structural model in the software SAP2000 by CSi, and allow for soil-structure interaction (SSI) by modelling the soil using non-linear springs generated using the p-y method. To capture the non-linearity of the stress-strain relation of the soil, a series of non-linear p-y curves were developed in the software LPile by Ensoft and converted into non-linear lateral springs, which could be modelled as multi-linear elastic link elements in the 3D structural model.

The collaborative approach to the design reduced the number of iterations required between structural and geotechnical teams and allowed for an efficient delivery. Sensitivity analyses allowing for variations in geology and uncertainties in the ground model were able to be easily carried out using the 3D structural model where SSI was integrated. In the final design oversize pile casings were adopted to provide a seismic gap and decouple seismic slope movements from the seismic loading of the structure. The oversize casings were allowed to displace during a seismic event to avoid the additional lateral inertia load from soil displacement to be applied to the piles. Adopting the oversize casings also contributed to a more even distribution of lateral stiffness for the piles across the site.

2 METHODOLOGY

The structural analysis was carried out using the software package SAP2000 for which a 3D non-linear static model was created. The key structural elements were modelled (refer to Figure 2) with the piles and steel planters modelled as frame elements, and the reinforced concrete flat slab modelled as shell elements (each section property is shown in a different colour). Rigid frame elements were used to connect the planters to the deck slab at stiffener locations.

A series of non-linear springs were required for the structural model in SAP2000 to model the soil. The p-y curves of the soil were generated using the software LPile and were subsequently converted into non-linear lateral springs for input. These non-linear springs representing the soil were applied as compression only springs at the quarter points around the circumference of each pile and at 1 m intervals along the pile length. The structural and geotechnical design methodologies are described in more detail in Section 2.1 and Section 2.2.

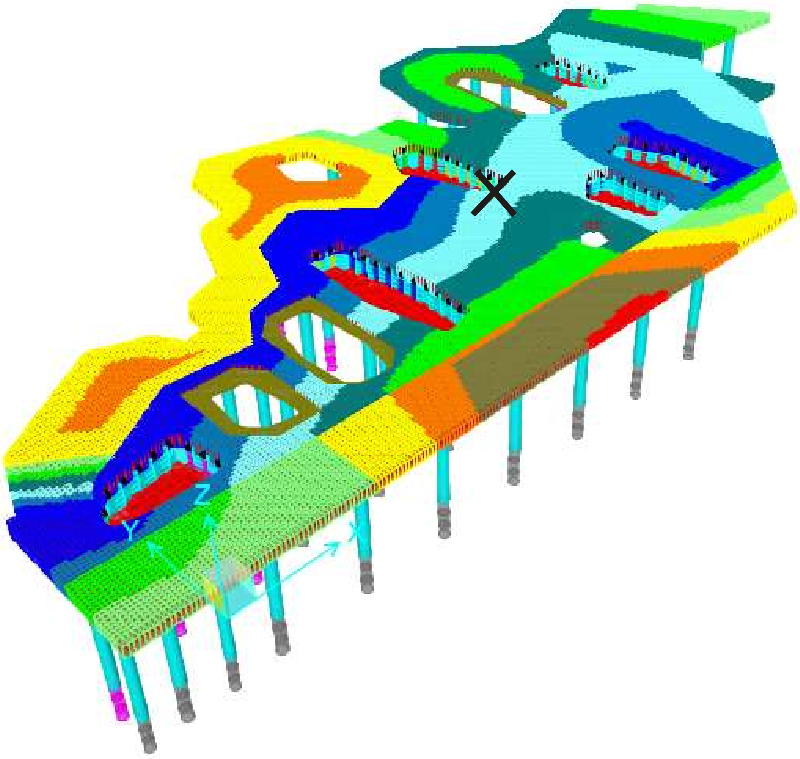

Figure 2: Te Wānanga SAP2000 model isometric view

2.1 Structural design methodology

2.1.1 Structural design philosophy

Te Wānanga was designed as an importance level 3 structure with a 50 year design life in accordance with AS/NZS 1170. All concrete elements were detailed to achieve a 100 year durability design life. Load combinations from AS/NZS 1170 and AS 4997 were adopted for design. All reinforced concrete elements were designed and detailed in accordance with NZS 3101 and all steel elements were designed in accordance with NZS 3404 with appropriate allowances from corrosion losses in accordance with SNZ TS 3404.

2.1.2 Pushover analysis

Seismic analysis was carried out using the non-linear static procedure (more commonly referred to as pushover analysis) in accordance with the guidance set out on ASCE 41-17 and ASCE 61-14, but adopting the material properties and load regimes in the relevant New Zealand standards. Pushover analysis broadly involves the creation of an analysis model which is representative of the structure. The relevant non-linear elements (in this case plastic hinges and soil stiffness) are also modelled. Horizontal forces of increasing magnitude (in this case proportional to the mass lumped at each node) are applied to the model until the plastic hinges reach predetermined rotation limit. The displacement of a target node (in this case the centroid of the deck which is marked by a cross in Figure 2) at a rotation limit is then compared to the displacement demand of the structure for a design level earthquake. The boundary conditions in the horizontal direction were modelled by non-linear p-y curves which represent the supporting ground, and the vertical support was modelled by a vertical restraint at the pile toe. The structure was displaced in both positive and negative directions, both parallel and perpendicular to the shoreline.

Column hinge properties were modelled in accordance with the relevant sections of ASCE 41-17 and ASCE 61-14. ASCE guidelines for the seismic design of wharves suggest allowing for a "weak column – strong deck" hierarchy. Accordingly, plastic hinges were allowed to form at both the head of the column and in the ground within the prescribed hinge rotation limits. ASCE 61-14 guidance gives allowable material strains and hence plastic hinge rotations for a given earthquake event. These rotations relate to a level of damage rather than an Ultimate Limit State (ULS) or a Maximum Credible Earthquake (MCE). The hinge rotations for "Controlled and Repairable Damage" were treated as the maximum rotations for the ULS earthquake. Similarly, the hinge rotations for "Life Safety Protection" were treated as the maximum rotations for the MCE earthquake.

The displacement demand for the ULS earthquake at the target node, ranged from approximately 125mm to 225mm. The displacement ductility demand on the structure at ULS ranged between 1.75 and 2.0. The reasons for the range in displacement and ductility demands result from the sensitivity cases that were carried out to assess how certain assumptions affected overall seismic performance.

For most analysis scenarios, for the purposes of calculating stiffness, the steel tube pile casings were treated as being composite with the reinforced concrete piles, and the steel tube pile casings were ignored for strength considerations. Under this assumption, the structure was found to be relatively stiff with fundamental elastic periods in the order of 0.8s, and therefore displacements were towards the lower end of the displacement range in the preceding paragraph.

As part of the sensitivity analyses, the casing was ignored for the calculation of stiffness. This scenario represented long term corrosion loss of the casing or a total debond of the casing from the reinforced concrete piles and gave a longer period of approximately 1.2s and displacements towards the upper end of the range above.

2.2 Geotechnical design methodology

2.2.1 Development of ground profile

There are four main geological units in the area of Te Wānanga, which are summarised in Table 1. A typical cross section of the geology is presented in Figure 3. Most of the piles were designed to be drilled through the soft marine sediment (Upper Tauranga Group, UTG) then embedded into the East Coast Bays Formation (ECBF) rock. The piles near the landward side of Te Wānanga were designed to be embedded through the existing and proposed rock buttress material before being socketed into ECBF.

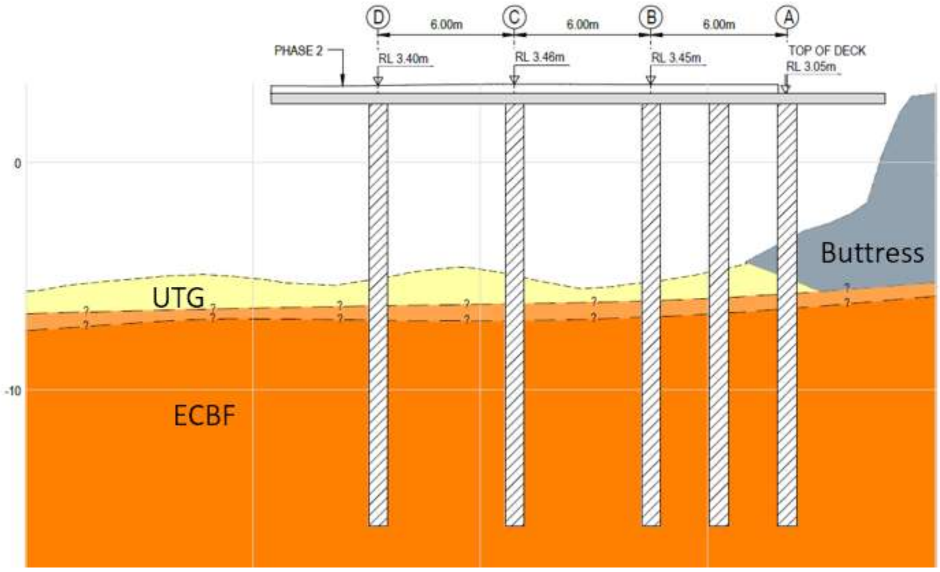

Figure 3: Typical cross section (north south) through Te Wānanga .

Table 1: Summary of geological units.

Geological units	Undrained shear strength (kPa)	Effective cohesion, c' (kPa)	Effective friction angle (°)
Fill – buttress	N/a	0	45
Upper Tauranga Group (UTG) – silts and clays	24	2	26
Weathered East Coast Bays Formation (WECBF)	N/a	100	36
Competent East Coast Bays Formation (ECBF)	N/a	250	36

In the development of representative geological profiles, it was important to identify the critical factors affecting the lateral stiffness of the piles, to ensure similar behaviour of the piles within zones of similar geology. Typical factors affecting the lateral stiffness of the piles include pile structural properties, pile embedment depth, strength of the soil/ rock and the length of the piles.

The pile structural properties and pile embedment lengths modelled in SAP2000 were independent from the generation of p-y curves. Since the UTG contributed very little resistance compared to the ECBF rock and the buttress fill material, the contribution in lateral stiffness due to the UTG could almost be considered negligible at small displacements. As the piles terminate at roughly the same level (the soffit of the wharf deck), the pile lateral stiffness could be considered to be governed by two factors: the elevation of ECBF rock and whether the pile was embedded through the buttress. As a result, the variation in rock level and the extent of the buttress caused the lateral stiffness of the piles to vary significantly across the site. When subjected to the same lateral displacement, the shorter piles attract more load than the longer piles. Therefore, the site was divided into 3 zones based on the ECBF rock contour (see Figure 4) for design, assuming the piles within the same zones would have similar load-displacement behaviours.

The row of piles embedded in the buttress material were expected to be significantly stiffer. Oversize casings were adopted for the piles embedded in the buttress in the final design, to allow for the potential movement of the buttress in a seismic event, as well as to reduce the lateral stiffness of the piles embedded in the buttress and ensure a more even spatial distribution of lateral stiffness. The soil within the oversize casings was designed to be removed, so the force from the material around the oversize casing would not be transferred to the pile. The process of designing the oversize casings is explained in more detail in Section 3.2.

Unforeseen ground conditions were considered to be a substantial risk during the construction phase. To manage this risk, a number of varying ground conditions were considered during the design to minimise the risk of redesign and delays on site. This required several additional analysis models with varying ground conditions. These sensitivity analyses included changing the weathered ECBF mantle parameters to ECBF rock both across the site and locally which allowed for variations in depth to competent rock on site.

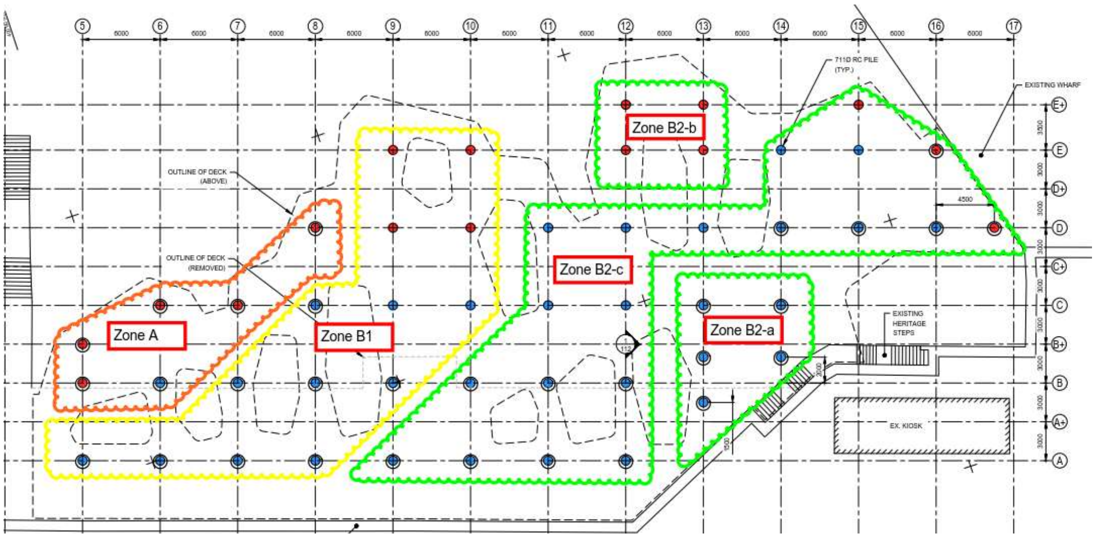

Figure 4: Representative pile zones based on rock elevations.

2.2.2 Generation and application of p-y curves

The software LPile by Ensoft was used to generate p-y curves. P-y curves are non-linear load-deformation curves used to model the lateral response of a pile which are specific for both the dimensions of the piles and the geological conditions. Appropriate models were selected to represent each geological unit, presented in Table 2.

Table 2: Summary of soil and rock strength parameters in LPile.

Geological units	**Soil type**	**Effective unit weight (kNm^{-3})**	**Uniaxial compressive strength/ Undrained shear strength (kPa)**	**Rock quality designator, RQD (%)**	**Soil/Rock strain parameter, E_{50}**	**Rock mass modulus (kPa)**
Upper Tauranga Group – silts and clays	Soft clay (Matlock)	16 6*	24	N/A	0.02	N/A
Weathered East Coast Bays Formation	Weak rock	19 9*	800 – 1500	5 – 40	0.004 – 0.0005	50000 – 500000
Competent East Coast Bays Formation	Weak rock	19 9*	1500	40	0.0005	500000

*Submerged unit weight.

The piles were divided into segments at 1 m intervals, and p-y curves were generated along the depth of the piles in the middle of each segment. Once the p-y curves were generated, they could be input directly into SAP2000 as multi-linear elastic link elements for structural modelling.

2.2.3 Grouping effects

The lateral efficiency of closely spaced piles reduces due to grouping effects. The grouping effect of the piles was evaluated using the methodology described in the GROUP 2016 Technical Manual (Reese et al. 2016) and was applied as p-multipliers to the p-y curves. Three types of grouping factors were considered, leading, trailing and side by side; skewing effects were not included due to relatively large spacings. The grouping factors of the piles around the edges of the site (end piles) may change depending on the direction of loading, since the end piles can either be leading piles or trailing piles depending on the direction of loading. To account for this effect, grouping factors of the end piles were calculated and applied specifically depending on the direction of loading on a pile by pile basis.

3 TECHNICAL CHALLENGES

3.1 Structural challenges

The irregular plan arrangement of Te Wānanga presented a number of challenges for both analysis and construction. Due to these complexities, and the additional challenging programme requirements, Te Wānanga project had Early Contractor Involvement (ECI). Invaluable feedback from the constructors included that a structure with a flat soffit would offer significant programme savings and end up being cheaper than a conventional "beam and slab" structure. Aesthetic requirements prevented any internal joints in the deck. This unconventional structural form presented a number of technical challenges. The irregular outer edge shape lead to a number of bespoke post shapes and fixing details for the balustrade.

The flat slab arrangement introduced a number of design complications. The minimum thickness of the deck was dictated both by overstrength considerations of the columns and by anchorage of the pile bars. Due to the required surface levels and step in the deck, the deck thickness was required to be up to 1000mm in parts. This increased thickness led to both an increased seismic mass and much larger shrinkage and temperature effects. Due to the increased seismic mass, as well as both the irregular mass distribution resulting from the plan layout, and the irregular stiffness distribution due to variable soil levels and profiles, it was decided to carry out pushover analysis in an attempt to minimise pile size and hence deck thickness to minimise temperature effects.

The positive differential temperature case (where the top of the concrete deck is hotter than the bottom due to solar radiation), combined with shrinkage led to large tension and bending forces in the concrete deck. This resulted in a large reinforcing content requirement when considering the crack width maximum recommended limits in NZS 3101 for a coastal environment. The risk of cracking was exacerbated by the constructor's preferred construction sequence, whereby the deck is poured sequentially in six pours of two spans width from east to west. The differential shrinkage effects due to restraint between the pours results in large tension forces on the newer pour, which are effectively "locked-in" at the start of the structure's design life.

Having a 3D structural model that accounted for SSI allowed for easy integration of these challenges during design and allowed the design team to work dynamically as these challenges arose.

3.2 Geotechnical challenges

Due to the variation in ECBF rock elevation, the slope stability of the existing buttress was found to be an issue for loading the piles in the seismic case. Under the critical design case, the UTG marine mud was susceptible to lateral spreading; the proposed rock buttress was also found to be unstable under a design level seismic event.

Under a design level seismic event, the factor of safety against slope failure of the slope near the western end of Te Wānanga in the north/ south direction was less than 1, indicating that the slope may displace under seismic loading. This displacement can exert extra horizontal load to the piles, the p-y curves generated were also not applicable, over the range of depth where soil displacements were expected.

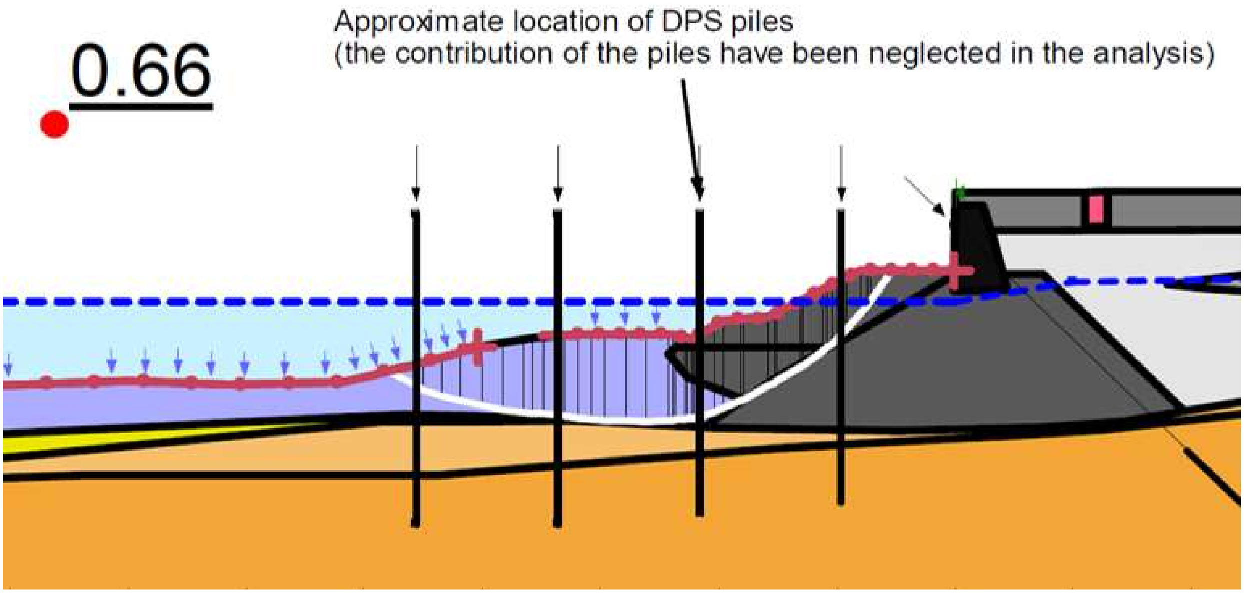

Figure 5: Slope failure under design level seismic event in Zone A.

As mentioned before, the pile lateral stiffness was found to be mainly governed by the rock elevation and the buttress. The most problematic piles were the row of piles closest to the existing seawall, as they have the shallowest rock elevation, and were directly in contact with the existing and proposed buttress material. As a result, the lateral stiffness of the piles embedded through the rock buttress was expected to be considerably higher than the rest of the piles. When the structure is subjected to a design level seismic event, these piles will attract more load for a given lateral displacement.

To address both the slope instability of the UTG as well as to ensure a more even spatial distribution of lateral stiffness of the piles, oversize casings with seismic gaps were proposed, embedded through the buttress material, for those piles affected by the potential soil movement under a seismic event.

The Newmark Sliding Block approach allows the horizontal acceleration to be reduced as a function of soil block movement, and was adopted to estimate the lateral displacement of the soil block. It was found that a soil block displacement of 30 - 90 mm could be expected in the north/south direction on the western side of Te Wānanga . A 250 mm seismic gap was adopted to ensure that the movement of the soil did not exert any extra inertia load to the piles. During a seismic event, the oversize casings were designed to allow for displacement and rotation without colliding with the piles.

The addition of the oversize casings addressed the potential issues of an increase in lateral load on the piles due to seismic displacement of the UTG, and also reduced the stiffness of the piles embedded in the buttress, reducing the proportion of the load these piles would attract. While lateral soil movement would displace and rotate the oversize casing, the seismic gap allowed for in the design ensures that the soil movement does not impose extra horizontal loading to the piles during a design seismic event. The oversize casings also reduced the stiffness of the piles embedded in the buttress, since the soil within the oversize casings will be removed;

these piles were no longer in direct contact with the buttress material, resulting in a more even distribution of lateral stiffness across Te Wānanga .

4 ADDITIONAL DESIGN CONSIDERATIONS

Although the design process described in this paper is considered appropriate for the design of Te Wānanga, the following items could be considered in other design scenarios.

Vertical springs were not included in this project, since the most critical design actions were horizontal seismic inertia loads. For structures where a large vertical load and displacement are expected, vertical deformation of the structure (e.g. the piles) required to develop the ultimate geotechnical bearing strength/ pull out strength could also be considered.

The change in non-linear reaction of the soil under cyclic loading was not captured in the springs. In reality, the stress-strain relationship of the soil under repeated loading will change with time, which could affect the damping in the structure.

A non-linear dynamic model, using time history analysis, would be able to capture both the cyclical stiffness degradation of both the soils and also the structural hinges. These effects were considered likely to have a relatively small effect on the final design of Te Wānanga but could be important in other scenarios.

5 CONCLUSION

This paper highlights the collaborative approach undertaken between geotechnical and structural designers to achieve a practical and efficient design of Te Wānanga. The method presented shows an effective way for structural engineers and geotechnical engineers to collaborate efficiently by developing a 3D finite element model of the structure and foundations that allowed for SSI. P-y curves were developed from the typical geological cross sections of a range of design zones that represented varying ground profiles and allowed for the non-linearity of the stress-strain relationship of the soil to be captured in structural modelling. From a geotechnical perspective, instead of analysing typical 2D sections and determining the maximum allowed lateral load of the critical piles, representative p-y curves were converted into lateral springs and provided to the structural engineers for direct input into the 3D finite element structural model produced in SAP2000, which greatly increased the efficiency of the analysis and allowed geotechnical and structural challenges to be addressed easily. Oversize casings were adopted in the design to cater for the potential movement of the UTG and the buttress, preventing the additional load applied to the piles, but also contributes to a more even distribution of lateral stiffness spatially across the site.

The lateral springs generated from p-y curves were modelled as multi-linear links elements in SAP2000. Compared to modelling them as elastic springs, this method greatly reduced the number of iterations required in the design process which again increases efficiency.

The structural team and the geotechnical teams worked together closely during the design process, which was a key element in the efficient and effective delivery of Te Wānanga project.

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