NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING
2019 Pacific Conference on Earthquake Engineering
TURNING HAZARD AWARENESS INTO RISK MITIGATION
4 – 6 April | SkyCity, Auckland | New Zealand

Lateral design of screw piles using a displacement-based design approach

E.C. Wright, A.K. Riman & S. J. Palmer
Tonkin & Taylor Ltd, Wellington.

ABSTRACT

This paper discusses the seismic lateral design of screw piles using a displacement-based design approach that develops some of the ideas presented in NZGS Module 4 (MBIE, 2016) related to performance-based design. This is illustrated in the context of a real case project to seismically strengthen Wellington Town Hall. The design approach is based on uniform displacement across the foundation system, which allows the assessment of potential actions in the foundation elements allowing for a range of ground stiffness scenarios. The analysis considers the combined lateral stiffnesses of foundations and other embedded elements.

The paper presents the design approach and describes some key considerations for geotechnical and structural engineers when adopting this approach.

1 INTRODUCTION

This paper sets out a displacement-based approach used in the lateral design of screw piles for the Wellington Town Hall seismic strengthening project. The Town Hall is an unreinforced masonry (URM) structure and is to be base isolated with the isolators supported on a raft slab suspended on 450 screw piles. The design approach can be summarised as:

- Simplifying the ground conditions into a series of uniform "ground models" (i.e. Model A, assuming no liquefaction. Model B, assuming liquefaction of the upper soils. Model C, assuming liquefaction of upper soils and some underlying weaker soils)
- Assessing load/displacement behaviour of individual piles in each of these "ground models"
- Developing "design cases" to model the variability of the ground across the site, and to model the potential development of liquefaction across the site. Each "design case" is represented by selected "ground models" applied over portions of the total building footprint area. (Portions of the total number of piles)

Paper 104

- Combining the contributions of the various piles to assess the total load displacement behaviour of the foundation system, for each "design case." Applying this to assess the displacements of the system for the buildings total base shear load for each "design case"
- Assessing the actions (bending moments and shear forces) in a pile for each "ground model" with the calculated system displacement

This culminates in an understanding of the foundation performance as a whole under lateral loading during the critical stages of an earthquake and in assessing the potential range of lateral actions in the piles.

The project referenced in this paper, still ongoing at time of writing and referred to herein as Town Hall, is a project to upgrade and seismically strengthen the Wellington Town Hall, including a base isolated founding system that meets 100% of the New Building Standard (NBS). Opened in 1904, the building has Heritage New Zealand Category One rating, but was closed to the public in 2013 because it was assessed as earthquake prone.

The substructure for the proposed strengthening includes a raft slab at ground level (the "upper slab") supported on Rubber-Lead Bearing (RLBs) base isolators, and a local area of basement with its floor (the "lower slab") extending below the base isolation plane. The rattle space around the base isolated structure constrains displacements of the base isolator and foundations to 500mm. Approximately 450 pinned and fixed head screw piles are installed beneath the upper and lower slabs. The foundations are tied together, which means that all the elements of the foundations will move in unison, in spite of their varying stiffnesses. The design provides sufficient base shear resistance from the screw piles within the displacement limits by combining the relative stiffnesses of each element.

The approach described for the Town Hall could be applied when designing or assessing pile foundations for other new or existing structures in a seismic setting. Projects that would be suitable would ideally involve early and close collaboration between structural and geotechnical engineers, and substructure systems that will behave as one (i.e. tied).

The paper is written from a geotechnical perspective; however, the design approach discussed relies heavily on geotechnical and structural engineers working closely together.

2 GROUND CONDITIONS

The ground conditions were first assessed, along with a high level understanding of the proposed structure, in order to identify potential hazards and critical design elements early in the design. This section presents a brief outline of the Town Hall ground model development, which informed the critical cases used for lateral design.

2.1 Overview

The site is located approximately 100m from the current shoreline of the Wellington Harbour. 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). The Wellington Fault is located approximately 1.4km northwest of the site. (Begg & Mazengarb, 1996) (Semmens, 2010)

Potential liquefaction could result in reduced soil strength and thus reduced lateral restraint that these soils can provide to the piles and building. The liquefaction analysis for Town Hall below identifies the extent of liquefaction expected to impact design. This extent of liquefaction could be expected to be triggered by a 200 year and more intense seismic events. The soil profile and liquefaction extent is summarised in Table 1 and the figures below.

Table 1 Liquefaction Potential and Extent

Layer	Liquefaction potential and extent
Reclamation Fill	Liquefaction not expected above groundwater level. Below ground water level, **continuous and widespread liquefaction** possible within this layer. (The base of the proposed 'upper slab" is at ground water level.)
Beach Deposits	**Localised liquefaction possible within occasional zones** of loose materials within the beach deposit layer. (The base of the proposed "lower slab" is at approximately the top of this layer.)
Upper Alluvium	**Possibility of Localised liquefaction cannot be discounted within occasional fine SAND lenses.** These Lenses were encountered during investigations.
Lower Alluvium	**Liquefaction potential low / nil**

The liquefaction analysis uses (Bray, J.D. and Sancio, R.B., 2006) and (Boulanger & Idriss, 2006) to assess the susceptibility of the soil and (Boulanger & Idriss, 2014) and (Boulanger, et al., 2016) for triggering.

2.2 Design ground models

The ground models developed for individual pile assessment are simple vertical (1D) "ground models", with no lateral variation. These "ground models" were analysed in different combinations to account for the expected lateral variability of the underlying ground conditions at the site as well as the effects of different stages of liquefaction triggered during an ultimate limit state event. All cases considered apply to 100% ULS loading conditions. The models are summarised below and shown in Figure 1 to 3.

Ground Model A is an extreme model with no liquefaction triggered and the full expected strength of the underlying Alluvium.

Ground Model B is a model where the full depth of the Reclamation Fill has liquefied. Underlying soils are not affected by liquefaction.

Ground Model C is an extreme model representing liquefaction to depth, where the full extent of the Reclamation Fill and Beach Deposits have liquefied as well as lenses in the Alluvium. Underlying soils are not affected by liquefaction. A reduced strength of the Alluvium also applied to represent variability in this layer.

The proposed piles to support the strengthened Town Hall include fixed head screw piles beneath the "upper slab" and pinned head screw piles beneath the "lower slab". These piles are shown as green and red respectively in Figure 1 to 3. These pile types were considered in combination with each "ground model" described in Table 2.

The lateral load displacement behaviour of both pile types in each "ground model" was assessed by applying LPile laterally loaded pile analysis.

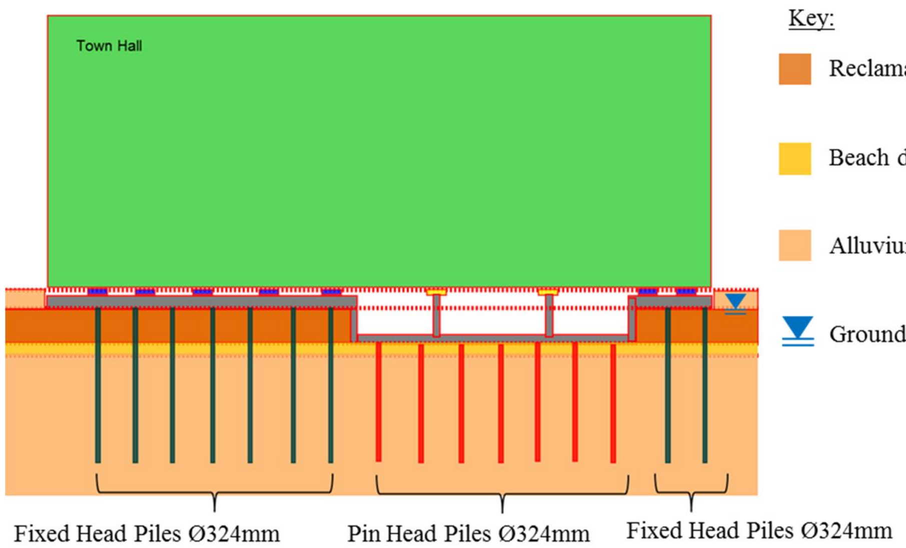

Figure 1 Ground model A

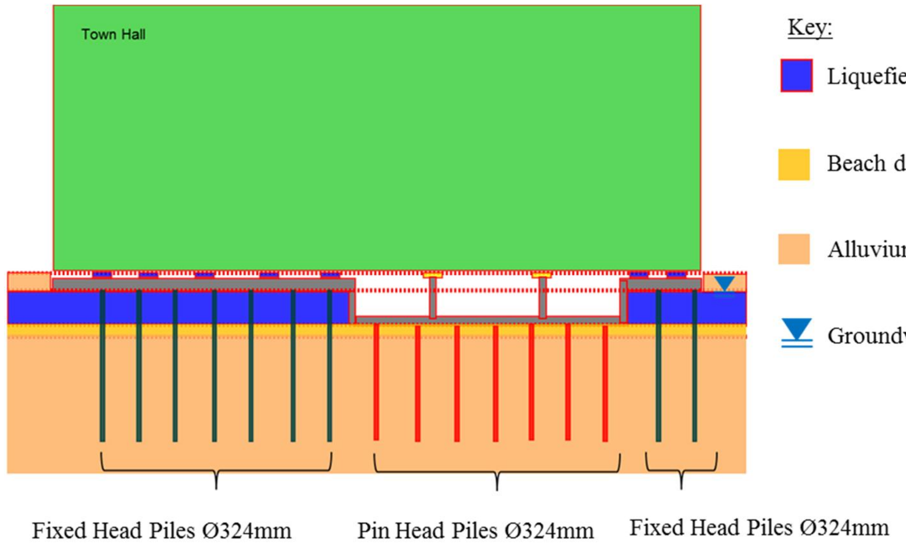

Figure 2 Ground model A

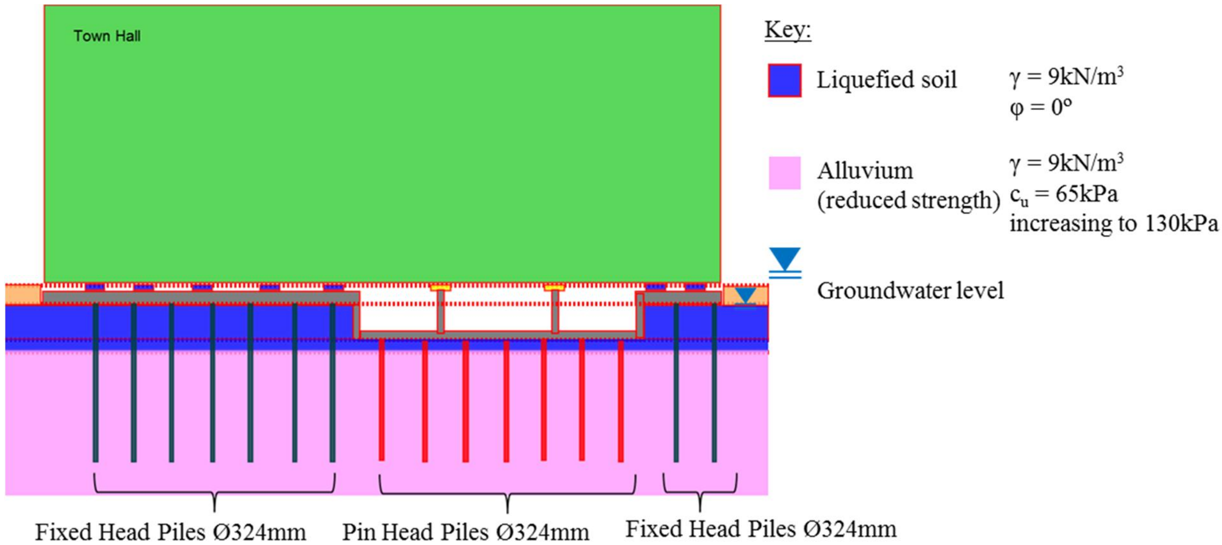

Figure 3 Ground model C

2.3 Design cases

A number of "design cases" were nominated to represent the variability of ground conditions across the site and the stages of liquefaction triggering and extent. "Design cases" were represented by selected "ground models" applied over portions of the total building footprint area (portions of the total number of piles). This, rather than adopting one site wide "typical" model, provides a more realistic understanding of lateral pile response, given the non-linear nature of soil and uncertainty in earthquake loading.

The "design cases" are described in Table 2 with references to the "ground models" presented in Section 2.2.

Table 2 Design cases

	Design Case 1	**Design Case 2**	**Design Case 3**
Case Description: Liquefaction Assumption (see below for NL, L and DL terminology)	An extreme case, i.e. no liquefaction. Liquefaction not triggered (NL 100% of site coverage) ULS shaking early in earthquake	An intermediate case, which may be critical for pile and basement wall design. Partial liquefaction (NL 30% L 70% of site coverage) ULS shaking stage: mid	An extreme case, i.e. widespread liquefaction. Maximum plausible extent of liquefaction (L 70% DL 30% of site coverage) ULS shaking late in earthquake
Ground Model	Model A (NL)	30% Model A (NL) 70% Model B (L)	70% Model B (L) 30% Model C (DL)
Pile Lateral Design Assumptions	100% of piles in NL	30% piles in NL 70% piles in L	70% of piles in L 30% of piles in DL
Basement Wall Resistance	100% of passive from NL soil	30% of passive resistance from NL soil 70% of passive resistance from L soil	100% of passive resistance from L soil

Terminology: NL – No Liquefaction (Ground Model A), L – Liquefaction (Ground Model B), DL – Deeper Liquefaction (Ground Model C)

Note: Case 3 and Case 2 also consider the likely inherent variability in the anticipated ground conditions.

As it is not possible to reasonably predict the lateral extent of liquefaction, it is recommended that the structural design consider all three cases, i.e.:

- Case 1: For understanding, but not critical
- Case 2: Likely critical with respect to lateral loads on basement walls and bending moments in piles
- Case 3: Likely critical with respect to total lateral displacement of the building

Case 2 is likely critical overall for pile design as elements in the system will be experiencing high displacement simultaneously with stiff ground response.

3 SYSTEM ANALYSIS

Analyses were undertaken to assess base shear resistance provided by individual foundation elements in response to substructure (pile head) lateral displacement. The following foundation elements were assessed:

- Piles – for each of the three ground models described in Section 2.2 in combination with each of the pile types; upper slab fixed head piles and lower slab pinhead piles
- Basement walls – for each of the ground models described in Section 2.2
- The total base shear resistance with substructure displacement was then calculated for each design case listed in Table 2 by summing the contributions from each foundation element. This comprised summing the resistance provided by; the number of piles of each type in each ground model and the portion of basement wall in each ground model; e.g. where a design case included 30% of Ground Model A, it was assumed that 30% of the number of piles of each type (pinned and fixed) were in Ground Model A. Similarly for other Ground Models and portions of basement wall. The calculated total resistance versus displacement relationships for each of the three design cases are plotted as solid lines in Figure 4. Figure 4 also presents as dashed lines the total resistance for each design case split into two components; that provided by piles and that provided by basement walls.

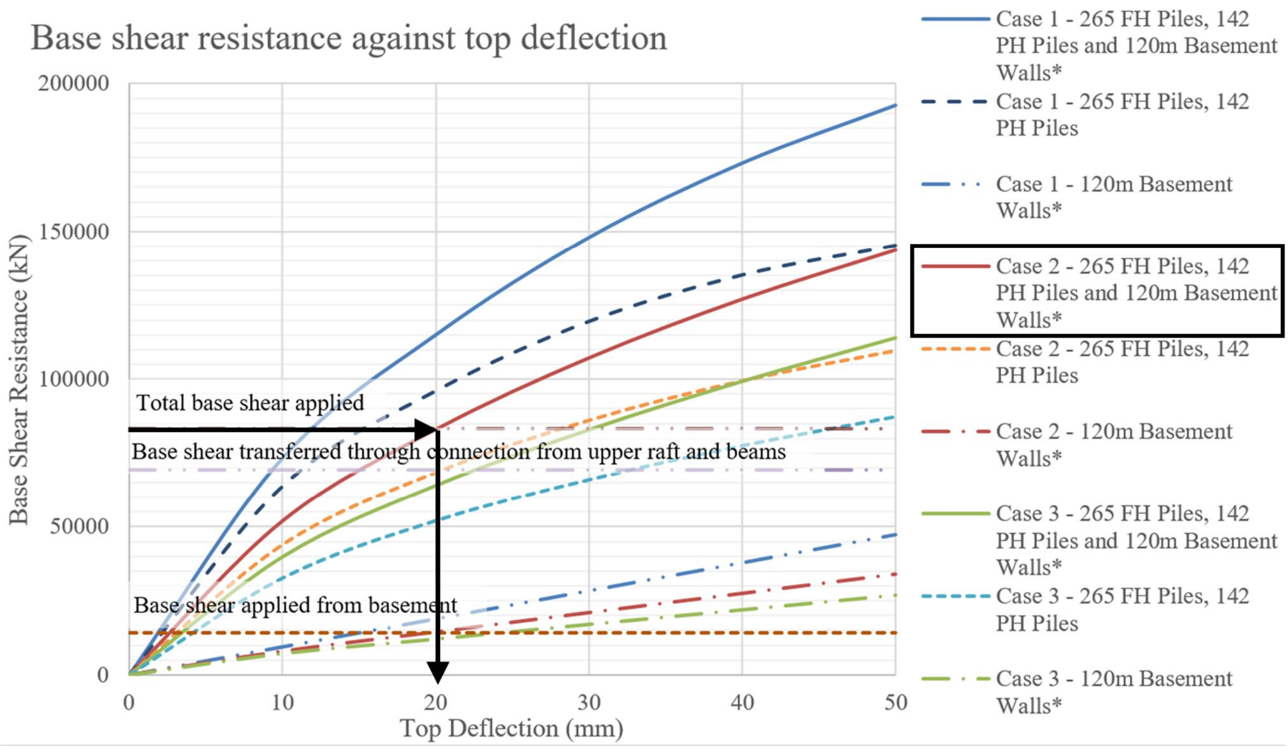

Figure 4 Base shear resistance against top deflection

The total base shear demand from the superstructure to the substructure, as assessed by the structural engineer, is 83,400kN. From Figure 4, to mobilise sufficient resistance across the substructure, the system (including pile heads and basement walls) must move 20mm with design case 2 conditions, and 12mm and 30mm for design cases 1 and 3 respectively. Design bending moments and shear forces down individual pile lengths were taken from the LPile outputs for each ground model and the assessed substructure deflection plus 10mm. This extra 10mm was to provide some redundancy.

4 KEY CONSIDERATIONS

Some key considerations for adopting this approach are briefly outlined below:

- Stress concentration within the substructure – in addition to displacement, the capacity of the beams connecting the upper and lower portions of the foundation system was a key consideration. Applying the calculated total system displacement to the combined stiffness of the lower portions of the foundation system allowed the load transferred through these beams to be calculated.

- Durability – sacrificial thickness of steel was included to allow for corrosion. A sensitivity of the lateral pile analysis was undertaken including and excluding the sacrificial thickness. The analysis conclusions were not sensitive to this.
- Group effects – The effect of pile spacing on lateral resistance was considered. The minimum shaft centre to centre spacing is 4.5 shaft diameters (D). It was concluded that with this pile spacing group effects did not need to be allowed for. This was based on (Brown, et al., 2001), (Mostafa & El Naggar, 2002) and (Rollins, et al., 2003) where it has been assessed that for dynamically loaded piles the minimum spacing can be assumed as 3 to 4 D for ignoring lateral pile group effects.
- Pile head fixity – pinned head piles were preferred for the basement because of floor waterproofing details. The displacement based approach allowed for variations in the number of fixed and pinned piles during the design process to be analysed efficiently.
- Other elements in the foundation system – while the Town Hall lateral design was for the proposed screw piles, additional elements can easily be incorporated when using this approach. The basement wall contribution was considered (shown on Figure 4), but conservatively excluded from the final calculation of total lateral restraint. The basement wall was a small lateral restraint component compared to the piles.

5 CONCLUSION

The design approach presented was an efficient way of analysing a complex design problem. The foundation system for Town Hall comprises screw piles that are all tied together, but will undergo different dynamic responses due to the ground conditions that vary laterally, vertically and with time during an earthquake. By focussing on displacement, we were able to successfully combine the relative stiffnesses of the individual elements to assess the performance of the foundation system as a whole, and assess the range of potential actions in piles.

6 ACKNOWLEDGMENT

The Authors would like to acknowledge the efforts of the project's structural design team from Holmes Consulting Group and the geotechnical peer review team from Beca. They appreciate having a client like Build Wellington (Wellington City Council) who value the importance of collaboration.

The Authors would also like to acknowledge the contribution to this paper and the efforts of the geotechnical team involved in the design development of Wellington Town Hall: Shirley Wang, Stefan Cook, Mike Jacka, Hayden Bowen, EngLiang Chin and Najla Kunhimon.

7 REFERENCES

Begg, J. G. & Mazengarb, C., 1996. *Geology of the Wellington Area: Sheets R27, R28 and Part Q27, Scale 1:50,000.* s.l.:s.n.

Boulanger, R. W. & Idriss, I. M., 2006. Liqufaction Susceptibility Criteria for SIlts and Clays. *Journey of Geotechnical and Geoenvironmental Engineering,* 132(11), pp. 1412-1426.

Boulanger, R. W. & Idriss, I. M., 2014. *CPT and SPT based liquefaction triggering procedures,* Davis, California: s.n.

Boulanger, R. W. et al., 2016. *Evaluating liquefaction and lateral spreading in interbedded sand, silt and clay deposits using the cone penetrometer.* s.l., s.n., pp. 5-9.

Bray, J.D. and Sancio, R.B., 2006. Assessment of the liquefaction susceptibility of fine-graded soils. *Journal of Geotechnical and Geoenvironmental Engineering,* 132(9), pp. 1165-1177.

Brown, D. A. et al., 2001. *Report 461 - Static and Dynamic Lateral Loading of Pile Groups,* s.l.: NCHRP.

MBIE, 2016. Module 4: Earthquake resistance foundation design. *Earthquake geotechnical engineering practice*, November.

Mostafa, Y. E. & El Naggar, M. H., 2002. Dynamic analysis of laterally loaded pile groups in sand and clay. *Canadian Geotechnical Journal,* 39(6), pp. 1358-1383.

Rollins, K. et al., 2003. *Response, Analysis, and Design of Pile Groups Subjected to Static & Dynamic Lateral Loads,* s.l.: Utah Department of Transportation Research Division.

Semmens, S. P. N. a. D. G., 2010. *It's Our Fault - Geological and Geotechnical Characterisation of the Wellington Central Business District,* s.l.: GNS Science Consultancy Report.