South British House – A collaborative approach for the seismic upgrade of a building

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ABSTRACT

This paper describes the geotechnical aspects of a collaborative seismic assessment and foundation strengthening design for an existing building in the Tauranga CBD. The reinforced concrete 6 storey high structure is supported on concrete piles founded on potentially liquefiable volcanic soils. Initial assessments concluded that the building seismic rating was governed by the foundations, limited by piles bearing on liquefiable soils. Both CPT based and shear wave velocity based methods were used to assess the liquefaction potential at the site. The data indicated that the site was susceptible to liquefaction at a moderate level of shaking, and that the zones of liquefaction differed across the building footprint. The detailed seismic assessment concluded that ground effects were a limiting factor for the building's seismic performance and foundation strengthening would be required. The assessment and design of the foundation strengthening scheme required numerical analyses to model the soil-structure interaction effects. Two independent numerical models, one structural the other geotechnical, were run in parallel. Close collaboration between the geotechnical and structural engineers enabled the independent models to converge allowing the strengthening design to be based on a robust analysis. A key feature of the foundation strengthening design was an emphasis on controlling building (and therefore soil) deformation under seismic loading. This meant that pile settlement tolerances under the ULS seismic conditions could be relaxed from that normally associated with conventional designs.

1 INTRODUCTION

The seismic assessment of buildings in New Zealand requires collaboration between structural and geotechnical engineers to ensure that both the building superstructure and local site conditions are adequately considered. Where the site conditions are difficult, such as liquefiable ground, or the superstructure unusual, such as an asymmetric structure, numerical modelling can be a useful tool to determine the effects of soil-structure interaction. The resulting soil-structure interaction effects can then be communicated between geotechnical and structural engineers to produce the best possible outcome for the building owner.

Such a process has been undertaken for South British House in Tauranga. South British House is a six storey high rectangular shaped building located on a near-level site at 35 Grey Street in the Tauranga CBD. The building was constructed circa 1978, and consists of a reinforced concrete-framed and shear walled dual system supported on concrete piles. The building is founded on potentially liquefiable volcanic soils and was classified as earthquake prone by an Initial Evaluation Procedure (IEP) completed in 2012. Design records and limited pile testing suggested that many of the piles supporting the structure were founded within the potentially liquefiable soils.

Figure 1: Location and façade of South British House

The building owner wished to strengthen the building to a minimum of 67% of the New Building Standard (67%NBS). A low percentage NBS rating based on the IEP combined with liquefiable ground conditions suggested extensive strengthening of both the substructure and superstructure would be required. In addition there were a number of challenges, including:

- Uncertainty surrounding the existing pile depths (there were no as-built records), although limited testing indicated there was variability in the lengths;
- An asymmetric structural system;
- The building is located within a constrained site (see Figure 1) surrounded by other buildings, an alley way and Grey Street. The majority of any foundation works would need to be carried out from within the building, i.e. with limited headroom;
- The presence of a significant underground service (11kV cable) running under the rear of the building which presented a significant constraint to investigation, design and construction;
- The Client's preference that the existing tenants at the front of the building on the ground floor should be able to remain during the works.

2 GEOTECHNICAL CONSIDERATIONS

The initial geotechnical investigations were conducted in 2013 and indicated significant risks of liquefaction to depths of at least 20m below ground level (bgl). These initial investigations consisted of Cone Penetration Testing (CPT) and machine boreholes. The soils identified during this testing included the Matua Subgroup, a material that can have a large amount of pumiceous content. More recently, the geotechnical community has become aware of the limitations of CPT based liquefaction assessments in the Matua Subgroup due to the potential for crushing of this pumice content (for example Orense et al, 2012). Therefore, additional geotechnical investigations were undertaken to better understand the liquefaction hazard by considering the low strain properties the materials being investigated.

This additional testing was undertaken in 2015 during the detailed seismic assessment process and consisted of seismic CPT (sCPT), hand auger boreholes and laboratory testing. These investigations were carried out in the alley to the south of the building and also internally.

Internal investigations by sCPT proved to be challenging owing to the electromagnetic 'noise' from the 11kV cable and difficulty with generating a signal through the ground floor slab. The latter was overcome by cutting slots through the floor slab to allow the surface wave generator to be in direct contact with the ground. The former was overcome by performing the testing at night when the cable could be turned off.

2.1 Soil Profile

The soil profile was collated from both series of investigations and was found to comprise fill and undifferentiated volcanic ashes overlying silty sands and sandy silts of the Matua Subgroup, with groundwater at around 2.5m bgl (refer Figure 2). The Matua Subgroup soils are of Pleistocene age (c. 1.8 Ma to 10,000 years) and can be highly variable in their distribution. The Matua Subgroup soils encountered included a succession of loose to medium dense silty sands and sandy silts, and medium dense to very dense sands.

An infilled valley / channel feature was encountered under Grey Street that consisted of mostly cohesive Holocene Alluvium. Some differences in the soil profile in the transverse direction were also found with the infilled valley feature appearing to narrow towards the north.

2.2 Liquefaction Assessment

Liquefaction analyses utilised conventional CPT methods (Boulanger & Idriss, 2014) combined with shear wave velocity methods (Kayen et al., 2013, 2015). The assessment of the Matua Subgroup included consideration of the shear wave velocity measurements in relation to the theoretical limit of the onset of liquefaction due to the age of the materials. The overall assessment indicated that a thin 0.5m layer just beneath the water table is likely to liquefy, along with a 3m thick layer from between 5 and 8m bgl across the site. The denser Matua Subgroup was regarded as less likely to liquefy at the seismic shaking levels being considered.

This initial liquefaction assessment was conducted with the magnitude and Peak Ground Acceleration (PGA) determined following NZS1170.5:2004. Following the updated recommendations from MBIE/NZGS Module 3 (2016), the PGA and magnitude were amended to that outlined in the NZTA Bridge Manual (2016), and resulted in the thin 0.5m layer as not being susceptible to liquefaction. Therefore the crust thickness across the site was able to be increased.

Regardless, extensive liquefaction is indicated at a PGA of 0.12-0.14g in the deeper 5-8m bgl layer, which would lead to a loss of bearing support to the piles founded within or close to the liquefied soils, resulting in step-change behaviour as outlined by Clayton et al (2014).

2.3 Pile Investigations

The original building drawings indicate that the structure was founded on piles formed as 2 pile or 3 pile groups, with some individual piles. The drawings do not indicate a required depth of founding for the piles and as-built information is not available. Low strain pile integrity testing was initially undertaken in 2013 and focussed on the building piles located along the alley and the building frontage on Grey Street. This initial testing suggested that the piles tested along Grey Street were founded at approximately 15-16m bgl and those along the alley at 5-6m bgl.

Other methods to confirm the pile depths were considered destructive, and as the piles were to be retained, it was decided to complete the low strain pile integrity testing on the remainder of the piles. This concluded that with the exception of the Grey Street piles, all piles were founded at approximately 5-6m bgl, and thus founded on or within a liquefiable layer (Figure 2). The testing was not conclusive for two piles located on the northern side, possibly shorter than anticipated, and this was considered during our analysis.

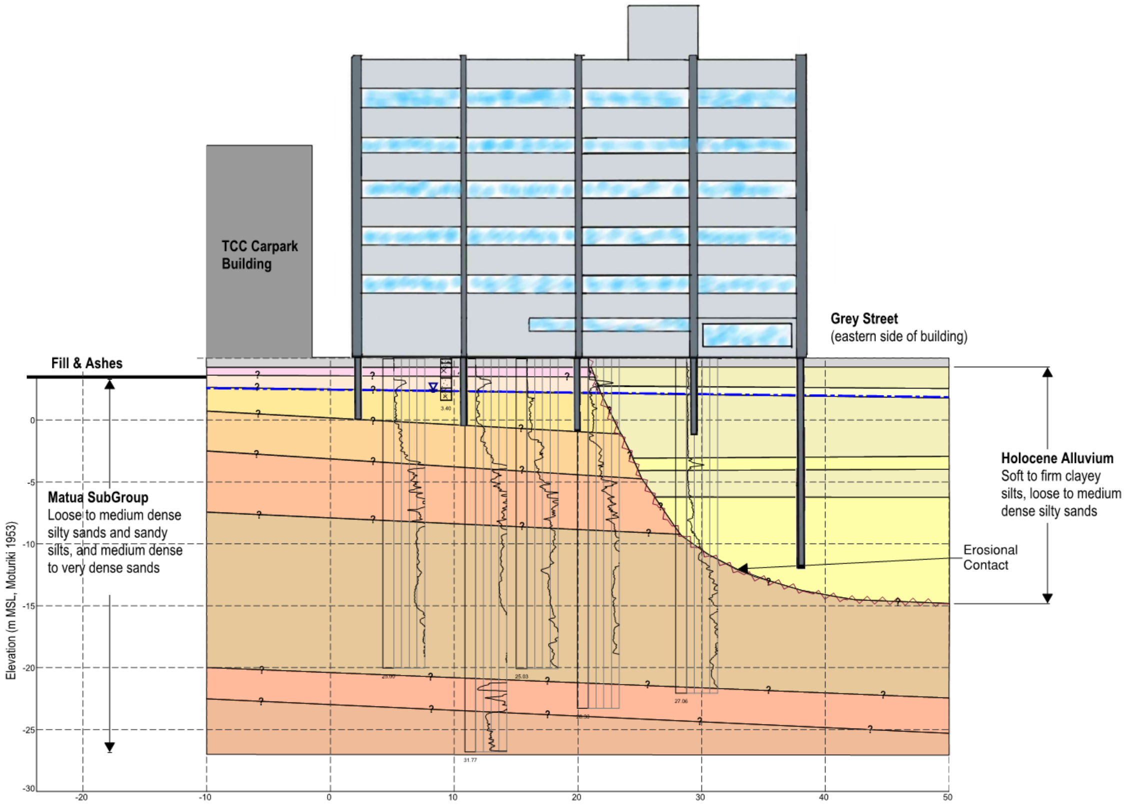

Figure 2: Soil profile and pile lengths southern side (Lavin et al, 2016)

2.4 Detailed Seismic Assessment

A detailed seismic assessment of the existing building was completed following the recommendations provided in the New Zealand Society for Earthquake Engineering (NZSEE) guideline document (NZSEE, 2006 including amendments 1, 2, 3 and 4).

The assessment concluded a seismic rating for the building of 55%NBS (IL2), as determined using the NZSEE guidelines. The rating was limited by the potential loss of pile bearing support resulting from the anticipated onset of liquefaction in the founding soils at 55% of the Ultimate Limit State (ULS) seismic loading (approximately 0.14g).

3 NUMERICAL MODELLING OF STRENGTHENING OPTIONS

A number of potential foundation strengthening solutions were considered, including:

- Reinforced concrete raft foundation tied into existing ground beams and pile caps.
- Formation of a soil raft by jet grouting under the building.
- Bored piles around the perimeter of the building with ground beams tied into the existing pile caps.
- Multiple screw or micro-piles each side of existing piles tied into the existing foundations by way of reinforced concrete pile caps.

Given that the liquefiable layers are influential in the movement of the building during seismic shaking, it was identified that there was a need for geotechnical and structural collaboration in order to appropriately consider the effects of the soil-structure interaction.

Preliminary geotechnical finite element models (FEM) of the problem were analysed using Plaxis2D Classic (2012). The initial results of soil behaviour were then provided to the structural engineers in the form of moduli of subgrade reaction or soil springs, to consider the effects of the various strengthening options on the building superstructure in eTABs. Results from eTABs were then incorporated back into the Plaxis2D model to update the soil response. In this manner, several iterations of modelling were performed.

3.1 Plaxis2D Model

Plaxis2D modelling was undertaken in order to understand the effects of the liquefiable layers on the existing foundations and superstructure, and the proposed new foundations. The variability in the soil profile across the building indicated less liquefiable material was present on the northern side of the building with a variable crust thickness above this. The axisymmetric design of the building also caused variability in the stiffness of the building from the north to the south and therefore the seismic loads that would be incurred across the building foundations. Thus both soil profiles were modelled to consider the three-dimensional effects on the building.

The modelling was not concerned with the response of the ground due to seismic shaking, rather the effects of loss of bearing, tilting and settlement on the superstructure. From the initial geotechnical model, structural engineers returned likely loading scenarios under static and seismic conditions across the footprint of the building and through the piles (refer to Lavin et al (2016)). These loads were included in the Plaxis2D model as point and uniformly distributed loads (UDL) at piles and across ground beams, respectively. The sensitivity of the depth of embedment of the existing piles, from the pile integrity testing, was considered during the modelling with variations in depths for the piles on the north-east side of the building and toward Grey Street.

The liquefied materials within the soil profiles on the northern and southern side were represented by a Mohr-Coulomb material with a reduced Young's Modulus and undrained shear strength. The stiffness of the liquefied soils was varied from 3-5% of the original values as sensitivity runs, following Cubrinovski et al (2006), and the undrained shear strength amended to a liquefied shear strength following Olson & Stark (2002).

3.2 Modelling Outcomes

The Plaxis2D modelling considered the proposed foundation options; including the existing foundation, a raft foundation, a raft with additional piles and a raft with ground improvement. All cases were considered for both the northern and southern sides of the building to understand the impact of the change in the soil profile beneath the building.

As anticipated, both the northern and southern sides of the building showed tilting toward the west under the existing piles when liquefied layers were included. The raft foundation with and without ground improvement also failed to reduce the differential settlements.

The inclusion of additional piles, in the centre and western areas of the building, which are founded well beneath the liquefiable layers, was found to provide a robust solution with resilience of the foundations to liquefaction (Figures 3 and 4). The soil movements resulting from this solution were considered manageable for the superstructure when included in the eTABs structural model.

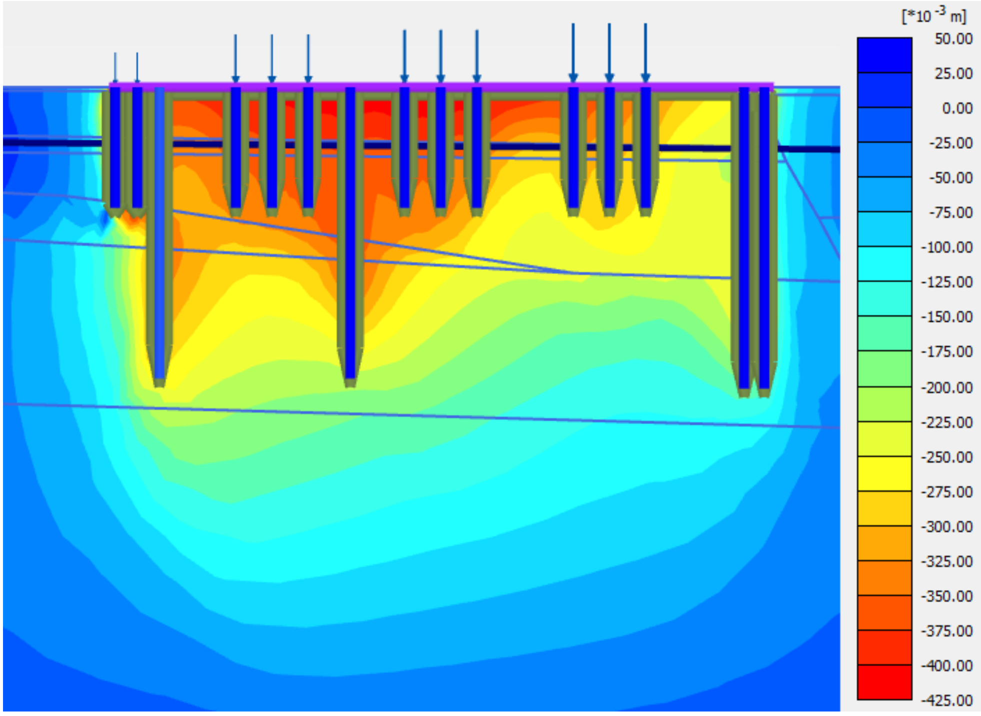

Figure 3: Plaxis2D analysis vertical settlements on northern side

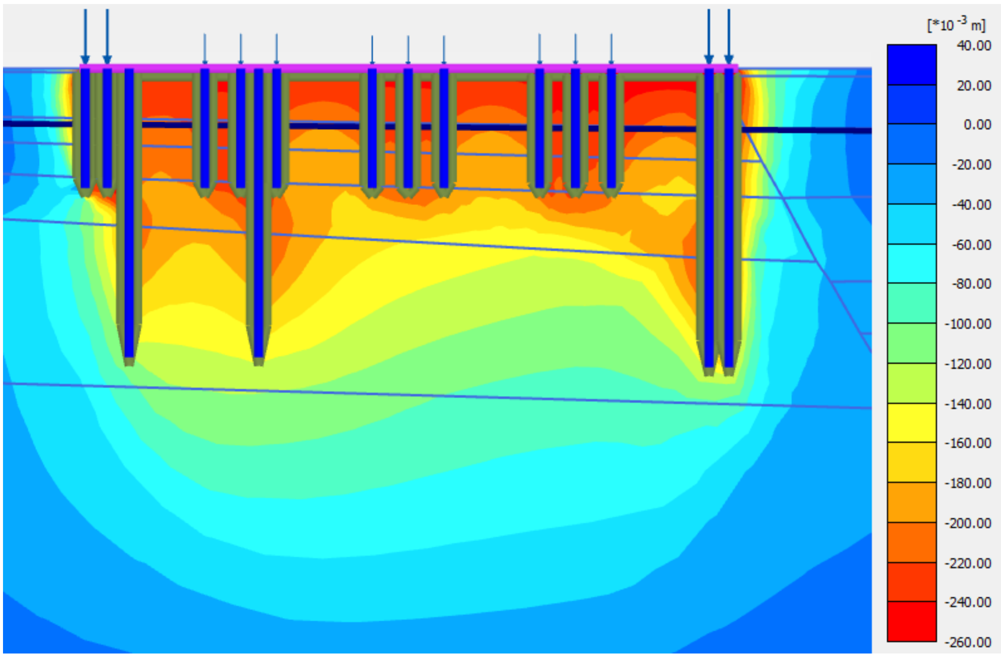

Figure 4: Plaxis2D analysis vertical settlements on southern side

4 FOUNDATION DESIGN

The results of the joint numerical modelling indicated that additional piles would be required under seismic conditions to support the building. This result, in combination with the constrained space around and within the building, lead to a foundation strengthening design using screw piles, with a substantial structural raft foundation beside the shear wall on the northern side of the building.

A key feature of the foundation strengthening design was an emphasis on controlling the building and associated soil deformations under seismic loading. Due to this, pile settlement tolerances under the ULS seismic conditions could be relaxed from that normally associated with conventional designs, as building serviceability was not the governing factor. This allowed the screw piles to be designed for greater deflections.

As Tauranga soils are inherently variable, the pile designer installed three screw piles to the south of the building to undertake pile testing. This testing allowed the pile designer to be confident that the depths of embedment and pile sizing would be adequate to provide the required support to the building during seismic events. The ultimate load and deflection results were incorporated back into the eTABs model to further improve the foundation and superstructure design.

5 CONSTRUCTION

Safety in design reviews of the construction process outlined difficulties in opening multiple sections of ground around the columns, with likely reduction in support for the pile caps and therefore the building, if a seismic event happened during construction. Due to this and the site constraints, screw piling has been undertaken in stages.

Initial screw piling began in late 2016 (Figure 6) with the raft foundation adjacent to the shear wall completed in June 2017. It is anticipated that construction of the foundations will be completed by the end of August 2017.

Figure 6: Stage 1 installation of screw piles (Lavin et al, 2016)

6 CONCLUSIONS

The seismic assessment of a six storey building in the Tauranga CBD has resulted in an increased %NBS and reduced cost to the building owner for retrofit, due to a collaborative approach between geotechnical and structural engineers, and the pile designer.

Initial evaluations of the South British House building suggested that it was earthquake prone and would require substantial foundation and superstructure remedial works. Additional investigations to characterise the behaviour of the Matua Subgroup led to a reduction in the likely depths of material susceptible to liquefaction, however pile bearing was still affected.

The collaborative approach between the structural and geotechnical engineers, and later the pile designer, involved the sharing of numerical modelling results and pile testing results. The incorporation of the various results into the numerical models resulted in a robust retrofit design that could be carried out within a restricted site.

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