South British House – seismic upgrade of a building founded on liquefiable soils

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ABSTRACT: This paper discusses the benefits of collaborative work between geotechnical and structural engineers in consideration of soil-structure interaction for the seismic assessment of an existing building in Tauranga's CBD. The structure consists of a concrete-framed and shear walled dual system supported on concrete piles, founded on potentially liquefiable volcanic soils. Initial assessment concluded that the building seismic rating was governed by the foundations, limited by pile bearing on liquefiable soils. Structural seismic assessment and design was carried out using ETABS Nonlinear. Geotechnical analysis of the soil was undertaken using Plaxis 2D. Nonlinear soil-structure analysis was undertaken to utilise the benefits of energy dissipation resulting from displacement of the building. A collaborative and iterative process was required in order to converge geotechnical and structural analysis outputs to an acceptable tolerance.

INTRODUCTION

Building background

South British House was constructed circa 1978. The six storey high rectangular shaped building is located on a near-level site at 35 Grey Street in the Tauranga CBD. The building is approximately 35m long and 17.5m wide.

The building structure consists of a reinforced concrete-framed and shear walled dual system supported on concrete piles, founded on potentially liquefiable volcanic soils. The building was classified as earthquake prone by an Initial Evaluation Procedure (IEP) completed in 2012. Previous geotechnical investigation and assessment had also indicated that the soils at the site were susceptible to liquefaction. Design records and limited pile testing suggested that many of the piles supporting the structure were founded within the potentially liquefiable soils.

Figure 1 shows the existing building frontage on Grey Street.

Figure 1 Existing building street frontage (eastern side)

Strengthening challenges

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

- Uncertainty surrounding the actual 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) and 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 was that the existing tenants at the front of the building on the ground floor should be able to remain during the works

GEOTECHNICAL CONSIDERATIONS

Geotechnical investigations

Initial geotechnical investigations were conducted in 2013 with further investigations carried out in 2015 during detailed seismic assessment. The investigations consisted of Cone Penetration Testing (CPT), seismic CPT (sCPT), machine boreholes, hand augers and laboratory testing. The investigations were carried out in the alley and road adjacent to the building, and also internally.

Internal investigations by sCPT proved to be challenging owing to 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.

Soil conditions

The soil profile was found to comprise fill and undifferentiated volcanic ashes overlying silty sands and sandy silts of the Matua Subgroup. The Matua Subgroup soils are of Pleistocene age (c. 1.8Ma 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.

The groundwater level was measured at around 2.5m below ground level (bgl). The inferred ground model is shown in Figure 2.

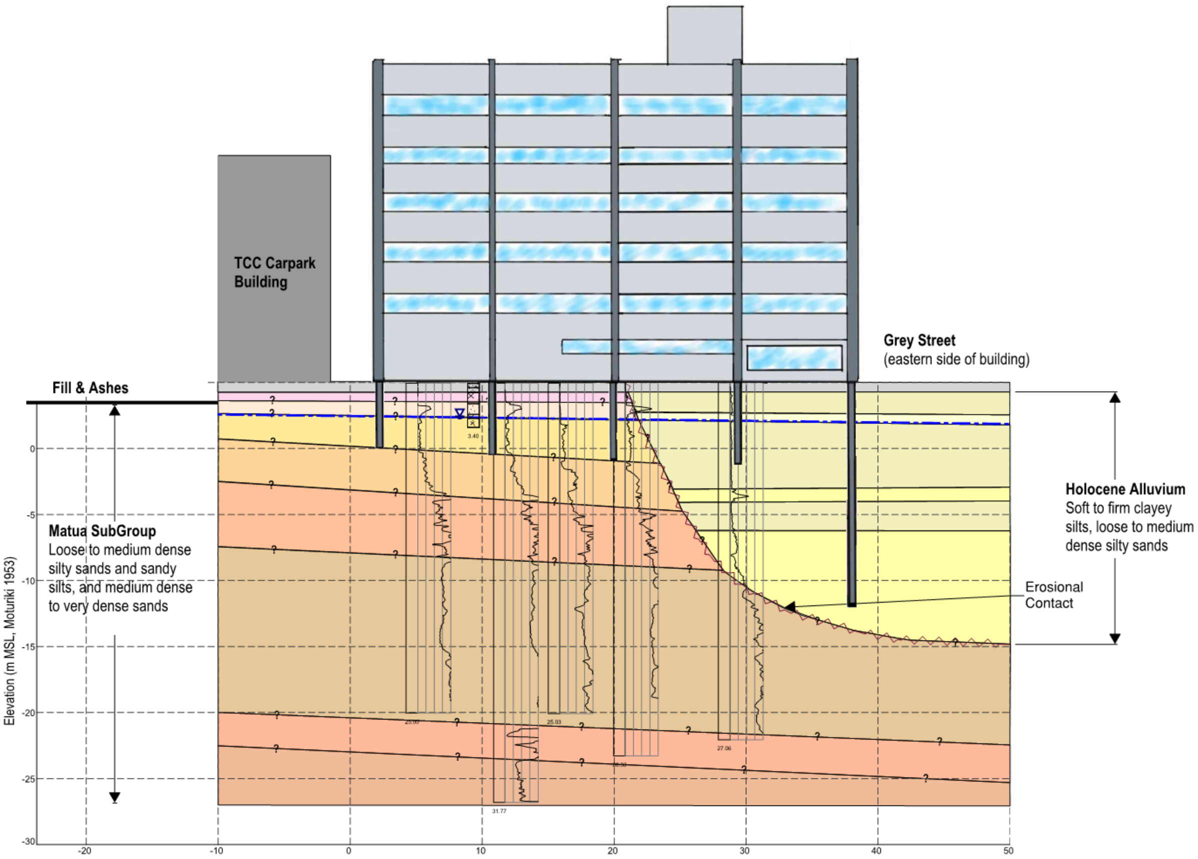

Figure 2 Typical soil profile in building longitudinal direction

The site subsoil classification was determined to be Class D (deep soil site) in accordance with NZS 1170.5:2004, based on the shear wave velocity measurements and anticipated depth to rock.

Existing foundations

The original building drawings indicated 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 carried out in two stages to provide an indication of the likely pile lengths. This testing indicated that the piles along at the eastern side of the building (Grey Street frontage) were mainly founded at approximately 15-16m bgl. The remaining piles were considered to be founded at approximately 5-6m bgl.

Ground effects

Liquefaction analyses utilised conventional CPT methods (Boulanger & Idriss, 2014) combined with shear wave velocity methods (Kayen, et al., 2013). The liquefaction assessment concluded that extensive liquefaction was indicated at a Peak Ground Acceleration (PGA) of 0.12-0.14g at a depth of between 5 and 10m bgl across much of the site. Extensive liquefaction within this zone would lead to a loss of bearing support to piles founded within or close to the liquefied soils, and this would result in a step-change behaviour (Clayton, et al., 2014).

SEISMIC ASSESSMENT FINDINGS

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).

Further assessment was carried out to challenge the likely behaviour of the building following potential loss of pile bearing support, considering shallow soil bearing support on pile caps and ground beams. Initial geotechnical and structural modelling and analysis considered estimated ground settlements from liquefaction and the resulting deformations reflected in the building superstructure. This exercise concluded that building deformations from this scenario were likely to be too significant to justify post-earthquake stability of the structure with respect to life-safety, without strengthening.

The assessment also identified other deficiencies in the building superstructure which are not discussed in this paper, including:

- Lack of floor slab ties for columns along the south elevation.
- Inadequate seating of concrete floor units near perimeter frames.
- Inadequate gravity support to steel roof trusses when subjected to seismic deformations.

SOIL-STRUCTURE INTERACTION

Foundation Strengthening Solution

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

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

The raft and micro-pile solutions were tested through a coordinated geotechnical and structural engineering approach for modelling soil supports and analysis. The seismic rating of the building was reassessed for each scheme considering the resulting soil deformations which were reflected in the superstructure.

The process identified that controlling soil deformation was the key factor in determining the most suitable solution. Instead of a capacity based approach, a deformation driven design approach was adopted with a target of controlling differential foundation settlements with minimal invasive works.

A raft foundation, whilst smoothing out some differential movement, was unable to provide suitable reduction. Significant differential settlements were found to be onerous on the superstructure.

The introduction of additional piles at the back (western end) of the building, founded well beneath the liquefiable layers, was found to reduce the likely settlements under seismic demands to tolerable levels.

Option 4 was selected as the preferred foundation strengthening solution. The concept consisted of installing four micro-piles each side of existing reinforced concrete pile caps. The new pile caps connect the new piles and existing foundations using high-strength Macalloy tie bars. Confirming the number of locations needed to adequately control soil/building settlement required further analysis and design. Figure 3 shows the final adopted configuration.

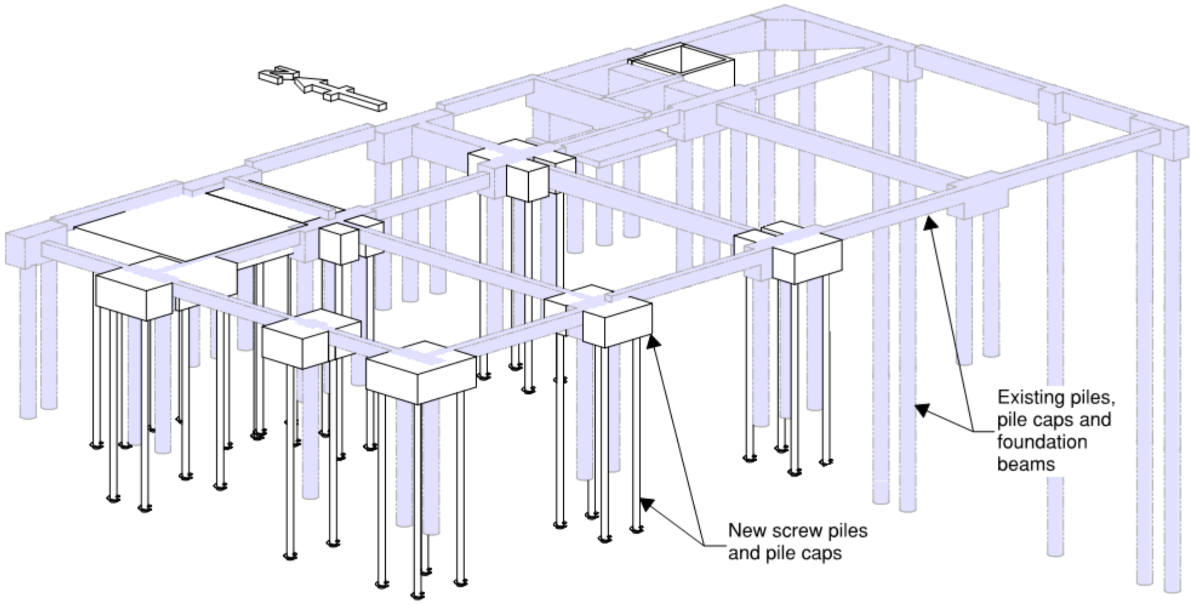

Figure 3 Perspective view of adopted foundation strengthening scheme

Modelling iteration

An iterative process was required to determine pile loads and deformations, and the extent of the retrofit foundation scheme. Geotechnical and structural models and analysis were undertaken in parallel.

Finite element modelling (FEM) for the ground was undertaken using Plaxis 2D Classic, with the soil modelled using the Mohr-Coulomb constitutive model. Liquefied soils were represented in the Plaxis model by reducing the soil stiffness of the liquefied units. Modelling considered the effects of the liquefied layers on the existing slab and piles and the proposed new foundations under earthquake loads. Variability exists within the soil profile beneath the property, with the soil profile on the western side indicating less liquefiable material than the soil profile on the eastern side (refer to Figure 3). In addition, the structural form and loading conditions differ between the north and south of the building. Due to this, cross-sections including liquefied material were evaluated in Plaxis, one for each side of the building.

Structural non-linear analysis of the building was completed using three-dimensional ETABS models. Two models were used in to consider a potential upper and lower bound soil stiffness;

'Upper-bound' Model: Consisting of a static nonlinear pushover model with plastic hinge properties for potential yield regions, and pin-ended supports with no soil spring allowance to represent a stiff soil with no vertical displacement under loading (refer Figure 4). This model was used to:

Consider the potential impact on the superstructure of a greater seismic base shear resulting from a shorter fundamental building period.

- Provide the base reactions for an initial estimate of the modulus of subgrade reactions by the geotechnical engineer.

- Identify upper-bound pile reactions.

'Lower-bound' Model: Consisting of bi-linear vertical soil spring supports to represent compressible soils with greater vertical displacement under loading. The spring properties were provided by the geotechnical engineer for the new and existing piles, based on the outputs from the Plaxis modelling. The softening effect of soil deformations beneath the foundations resulted in a lengthening of the fundamental building period and a reduction in the seismic base shear, compared to pin-based supports with no soil springs. This model was used to:

- Estimate vertical displacement of the new and existing piles.
- Re-assess the seismic rating of the building superstructure for the deformations resulting from differential soil settlements.
- Determine the extent of retrofit piles required for the adopted foundation strengthening scheme.

An initial estimate of soil spring values was used for structural analysis. Base reactions and displacements from the structural ETABS model were then used as inputs in a geotechnical Plaxis model. The modulus of subgrade reaction was updated by considering the deformations from the Plaxis model in response to the latest applied loads. This modulus was then provided to the structural team, who subsequently incorporated this into the ETABS model. In this way the geotechnical and structural models were improved and updated through an iterative process until reasonable agreement of deformation and reactions was reached. This collaborative process involved both geotechnical and structural engineers interrogating the results and inputs in each of the models.

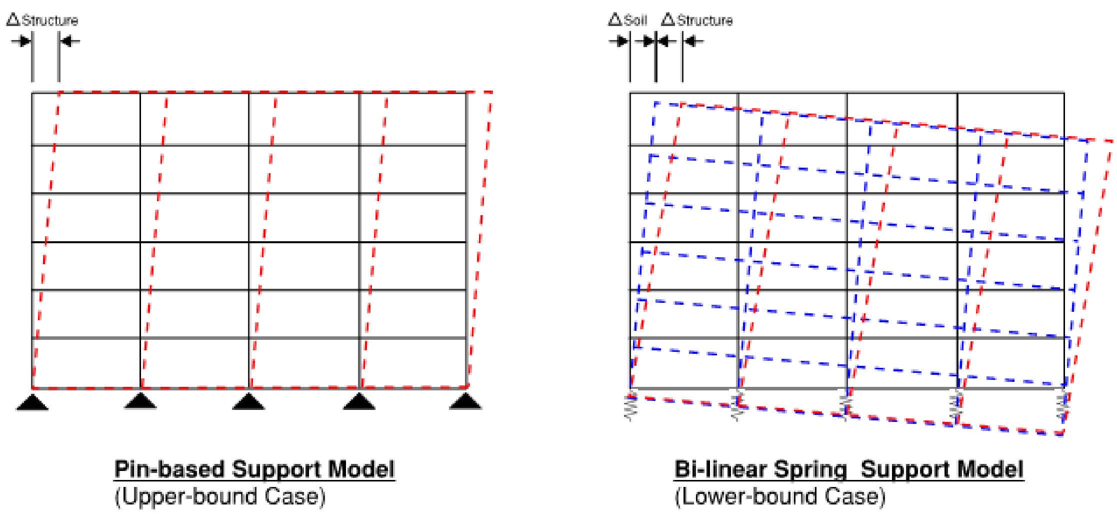

Figure 4 Upper and lower-bound analysis model support conditions

Screw Piles

Screw piles were adopted as the most suitable solution for the site challenges presented. As well as being a cost effective solution compared to the alternatives, screw piles had the added benefit of reducing the construction noise and vibration effects, and potential for disruption to the existing tenants (refer Figure 5).

A pile load test was carried out by the screw pile contractor in advance of production piling to confirm the pile design. In addition to confirming the geotechnical ultimate capacity that could be achieved at the site, the pile load test provided data for the load-displacement behaviour that could be expected. With the focus on deformation control adopted for the strengthening design, pile settlement tolerances under the ULS seismic conditions could be relaxed from that normally associated with conventional designs.

Initially, reinforced concrete and steel beam pile cap options were considered to span between the new screw piles beneath the existing pile caps to support the building column loads. Subsequently, following a safety-in-design review (which included both the client and constructor), an option to core high-strength Macalloy through the existing pile caps was adopted to reduce the depth of the new pile caps and therefore reduce the depth of excavation during construction.

Figure 5 Screw pile installation

Outcome

With strengthening of the foundations and retrofit work to remediate deficiencies in the superstructure, assessment work indicated that the building seismic rating could be increased to 90*%NBS* (IL2).

CONCLUSIONS

The design of seismic strengthening for the existing building at 35 Grey St, Tauranga has been achieved by allowing controlled foundation displacement under seismic loading. By considering tolerable building deformations resulting from settlement of new and existing piles founded on liquefiable soils, a pragmatic strengthening design solution has been developed to achieve minimal disruption.

The challenges presented with this seismic strengthening project required a collaborative and iterative approach between geotechnical and structural engineers, following the principles set out in the NZSEE guidelines for soil-structure interaction.

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