Design of structural foundations on sites prone to liquefaction and lateral spreading

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ABSTRACT: Seismic behaviour of structures on liquefiable ground is affected by the type, strength and stiffness of the structural foundation, seismic response of the structure and soil, thickness and properties of liquefiable soil layers and non-liquefiable crust, intensity of ground motion and many other factors. Costly ground improvement is commonly carried out to stabilise liquefiable soils. In many cases seismic performance requirements for buildings located on liquefiable sites can be satisfied without ground improvement. Design of the foundation systems for the Rotorua Police Station and the Grey Base Hospital complex involved analyses of soil-structure interaction that took into account liquefaction and lateral spreading effects. It has been demonstrated that design of the foundation systems to cope with liquefaction and lateral spreading effects, in some cases, can provide more cost-efficient designs compared to ground improvement.

1 PERFORMANCE BASED DESIGN OF A STRUCTURAL FOUNDATION ON LIQUEFIABLE GROUND WITH A NATURAL SOIL CRUST

1.1 Design Issues

The mechanism of settlement associated with liquefaction is complex. The effect of structural inertia forces and shear deformation of the liquefied ground on the magnitude of settlement can be very substantial in many cases. In addition to the settlement associated with volumetric strain, buildings can experience settlement associated with shear deformation or deviatoric strain. According to Bray & Dashti (2010), liquefaction-induced settlement of buildings is affected by the following displacement mechanisms (Figure 1): (a) Volumetric strains caused by water flow in response to transient gradients; (b) Partial bearing failure due to soil softening; (c) Soil-structure-interaction-induced building ratcheting during earthquake loading. In many cases it is possible to completely avoid or substantially reduce the extent of ground improvement and include the benefits of non-liquefiable soil crust, lower foundation stiffness and greater foundation damping in the design of the structure. Recent research work by Karamitros et al. (2013) indicated that it is possible to ensure satisfactory foundation performance in terms of acceptable settlements and adequate post-seismic static bearing capacity, in the presence of a reasonably thick and shear-resistant non-liquefiable soil crust. The exceptions are sites that could be susceptible to lateral spreading or where there is a large contrast in ground stiffness or liquefaction potential across the site.

The early stage concept design for the Rotorua Police building required several million dollars worth of ground improvement to mitigate effects of liquefaction. This paper describes a design framework used for the design of the Rotorua Police building founded on liquefiable ground with non-liquefiable soil crust. The design framework utilises available empirical, analytical and numerical methods. The design procedures used by Opus International Consultants Limited (Opus) on the Rotorua Police project resulted in substantial cost savings due to avoidance of ground improvement.

1.2 Effect of natural soil crust

Based on observations after past earthquakes, Ishii and Tokimatsu (1988) concluded that for buildings where the width of the foundation is large (2-3 times thickness of the liquefiable soil layer), the settlement of the building is approximately equal to the free-field settlement of the ground surface (i.e.

settlement of the ground surface is not affected by structural loads).

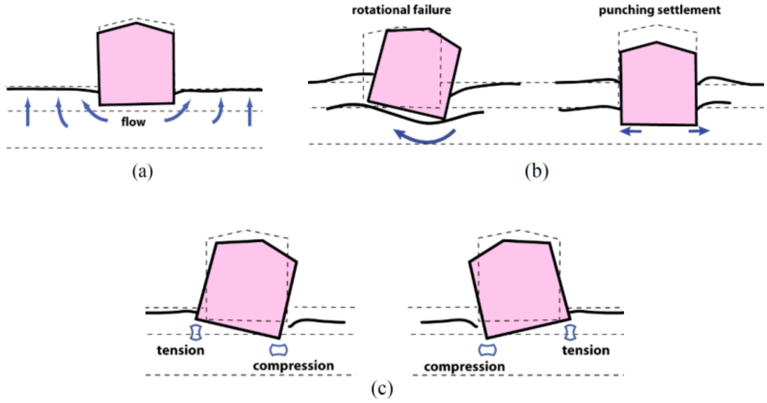

Figure 1 - Liquefaction-induced mechanisms affecting settlement (Bray & Dashti, 2010)

According to Seed et al. (2001), punching / bearing settlements can be expected to be large (many tens of centimetres or more) when post-liquefaction strength provide a factor of safety of less than 1.0 under gravity loading (without additional vertical loads associated with earthquake-induced rocking). These types of punching/bearing settlements can be expected to be small (less than 3 to 5 cm) when liquefaction occurs but the minimum factor of safety under the worst case combination of seismically-induced transient vertical loads plus static gravity loads and based on post-liquefaction strength is greater than about 2.0.

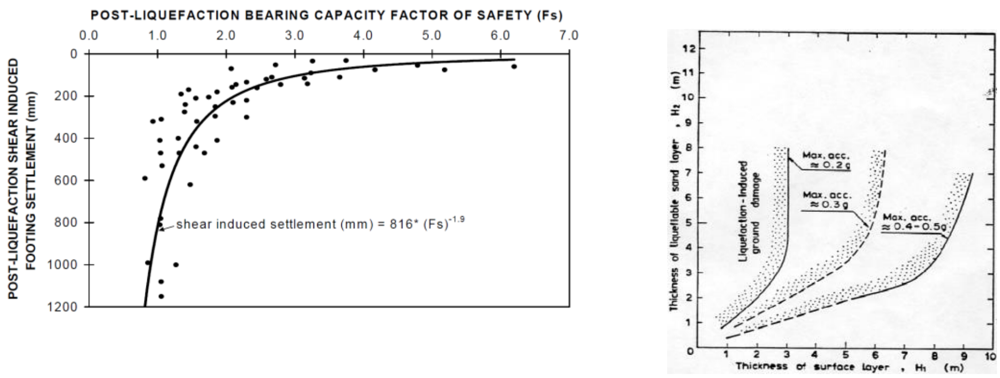

Figure 2 - Correlation between post-liquefaction shear-induced settlement and post-liquefaction bearing capacity factor of safety, Naesgaard et al., 1998 (left); correlations for the assessment of liquefaction-induced damage, Ishihara, 1985 (right)

For light structures (1 to 3 storey buildings) founded on a non-liquefiable cohesive crust over liquefied soil, settlement associated with the shear strain deformations can be estimated using Figure 2 developed by Naesgaard et al. (1998) based on limited total-stress dynamic numerical analyses. The shear strain induced settlement is additional to the post-liquefaction free field consolidation settlements. Ishihara (1985) developed correlations between surface manifestation of liquefaction (such as surface rupture and sand boils) and thicknesses of the liquefied layer and the overlying non-liquefied crust (Figure 2). The liquefaction manifestation correlations shown on Figure 2 can be considered for light (one to three storey) buildings.

Other simplified design procedures include (Andersen et al., 2007):

- Setting the foundation punching resistance such that the shear strength of the non-liquefiable crust around the perimeter of the foundation is greater than the foundation load, or
- Setting the foundation punching resistance such that the shear strength of the non-liquefiable crust around the perimeter of the foundation plus the bearing capacity of the underlying

liquefied soil using the residual strength is greater than the foundation load.

The important point of note that is borne out from the studies cited above is that the presence of a liquefiable layer overlain by a non-liquefiable crust of sufficient thickness and shear strength would not necessarily result in the manifestation of adverse effects at the ground surface even when the layer has undergone liquefaction after an earthquake. A possible explanation for this effect is that for the case of shallow foundation the majority of shear deformations due to the application of the foundation loads are expected to concentrate in the vicinity of the foundation. We note that this explanation is consistent with the results of our own analysis which are reported below (see for example Figures 3 and 4). Alternatively, numerical methods can also be used to design foundation systems for buildings founded on liquefiable ground. Numerical (e.g. finite element or finite difference) models can describe complex soil profiles and soil - structure interaction effects. Total stress and effective stress analysis procedures are available to assess triggering of liquefaction and the effects of liquefaction on the seismic response of the foundation-structure system. 2D and 3D numerical modelling has been carried out by a number of researchers and, while not free from limitations, has been proven to provide good quality results in many cases (Bray & Dashti, 2012).

Earthquake time histories are required as input ground motions in these analyses. Both simplified and numerical design procedures described above have been used by Opus to design a foundation system for the new Rotorua Police building.

1.3 **Site conditions and geotechnical design**

The recently completed three storey Rotorua Police Station building (plan size 26 m x 80 m) has been designed as an Importance Level 4 structure serving post-disaster function and remaining fully functional as critical infrastructure following a disaster such as a major seismic event. The structural design incorporates low damage self-centring PREcast Seismic Structural Systems technology adopted as an alternative to a more conventional reinforced concrete frame structure. Post-tensioned steel tendons were used to clamp rocking concrete shear walls to the foundations to resist earthquake forces, while remaining very stiff under serviceability loadings. The building is founded on a stiff reinforced concrete raft foundation over potentially liquefiable ground with non-liquefiable crust. Close interaction between the geotechnical designers (Opus) and the structural designers (Spiire) was required to develop a cost-effective design of the foundation system for the building. The design peak ground accelerations are as follows: SLS2 PGA = 0.27g; ULS PGA = 0.48g.

The following geotechnical analysis framework has been developed and used:

- Assessment of the liquefaction potential of the pumiceous soils based on borehole, CPT, shear wave velocity and laboratory test data
- Consideration of static and seismic performance requirements for the building based on the NZ Building Code
- Assessment of static building settlement and seismic (free field) subsidence
- Assessment of the static and seismic ultimate bearing capacity of the site soils
- Consideration of the effects of liquefaction on the shallow building foundation based on simplified methods described in Section 2 of this paper
- Static settlement (under gravity loads) and seismic pushover analyses of the raft foundation – soil system using numerical methods (finite element model of the raft – soil system was developed using computer program Plaxis 2D)
- Analysis of the stress-strain state of the raft foundation based on modelling of the raft on bi-linear Winkler springs allowing for differential ground displacements or loss of support to areas of the raft. A 3D model of the raft foundation was developed and analysed using computer program SAFE. Static and dynamic analyses of the raft were carried out.
- Dynamic time history finite element analysis of the soil-foundation -structure interaction using Plaxis 2D

The site is formed by undifferentiated pumiceous alluvium and is a geothermal site with low geothermal activity. The ground water table was 5 m below the ground surface level at the time of the investigations, but was conservatively assumed to be at 4 m depth for the design. Geotechnical investigations for the project comprised boreholes, seismic cone penetration tests (CPTs) with shear wave velocity measurements, grading and Atterberg Limit tests. The site ground conditions and extent of liquefaction based on the analysis of CPT data are summarised in Table 1.

Table 1. Summary of the site ground conditions

Unit No	**Depth range, m**	**Pumiceous soil type**	**Liquefaction for SLS2 & ULS events - depth range, m**
1	0 to 4	Sandy gravel and silty sand	No liquefaction (above GWT)
2	4 to 7	Silt	5.5-5.8; 6.5-6.8 (based on CPT)
3	7 to 11	Silty sand with minor gravel	No liquefaction
4	11 to 16.5	Sand	11 -16.5 (based on CPT)
5	16.5 to 25	Silt and silty sand	Localised liquefaction in up to 0.3 m think layers (based on CPT)

Available methods for the assessment of liquefaction potential are based on case studies for hard-grained alluvial or marine soils and their applicability to pumiceous soils has not been verified. Well recorded and analysed liquefaction case studies for pumiceous sands are very limited. Calibration chamber test results reported by Wesley et al (1998) indicate that end cone resistance recorded for dense pumice sands are similar to those measured for loose pumice sands, with relatively low values usually being measured. This means that CPT results may not necessarily provide an accurate representation of the relative density of a pumice sand. Recent studies by Orense et al. (2012) and Orense and Pender (2013) have further corroborated this finding, stating that the use of conventional methods for the assessment of liquefaction potential based on CPT data results in a conservative estimate of liquefaction potential for pumiceous soils and that liquefaction assessment based on Shear Wave Velocity test data may be more reliable. In our liquefaction potential assessment for this project, both the CPT and shear wave velocity results have been taken into account as per the methods described by Idriss and Boulanger (2008) and Kayen et al. (2013); nevertheless, as per the discussions above, we have greater confidence with the assessment using the shear wave velocity results due to the pumiceous nature of the materials found at the site. The measured shear wave velocities for our site range from 300m/s to 1300 m/s, which indicates that the site soils have low potential for liquefaction (Youd et al., 2001). Notwithstanding these findings however, it is assumed for our geotechnical design that the site soils will behave in accordance with the liquefaction assessment based on the CPT data due to the unreliability of available methods of assessment of liquefaction potential for pumiceous soils.

Under a SLS1 event the site soils are not susceptible to liquefaction, and therefore no subsidence associated with liquefaction is expected for the SLS1 event. Subsidence of liquefied ground (or free field settlement) associated with densification of liquefiable soils is expected to vary from 50 mm for the SLS2 event to 120 mm for the ULS event. Our assessment of the bearing capacity indicated that for the seismic load combinations with residual liquefied soil strength, the factor of safety against bearing capacity failure was 2.6. Therefore, according to Seed et al. (2001), punching / bearing settlements can be expected to be small (less than 3 to 5 cm). According to the graph developed by Naesgaard et al. (1998) and shown on Figure 2, the expected seismic settlement of the building in the ULS event is 130 mm. Also, the thickness of the non-liquefiable crust according to Ishihara (1985) is sufficient to prevent surface manifestation of liquefaction. Vertical displacements of soil from the 2D finite element analysis (seismic pushover analysis) are shown on Figure 3. The raft was assumed to be rigid in this analysis.

Our analysis indicates that, assuming that the site soils will liquefy, ratcheting may result in the maximum total settlement of approximately 200 mm and differential settlement of 70 mm over the 26 m width of the raft, which is acceptable for the ULS event (collapse avoidance). A more complex dynamic finite element time history analysis of the soil-foundation-structure interaction using Plaxis 2D was carried out to assess the effect of dynamic ratcheting (Figure 1c). The time histories from four historical earthquakes records have been used in the analysis. The superstructure was modelled as a two-storey portal frame constructed of plate elements. Node-to-node anchors were added in the form of diagonal cross bracings to provide the large lateral stiffness of the shear walls. The ground floor and foundation were modelled as a thick slab consisting of a soil cluster enclosed within four plate elements, where the plate element on the bottom represents the raft foundation. The mass assigned to the plate element on each floor was back-calculated from the equivalent static loads given by the structural engineers. This has been done to account for the inertia forces acting on the structure in the time-history analysis. Our analysis indicated that the RC raft foundation shown on Figure 4 would behave adequately under static and seismic load.

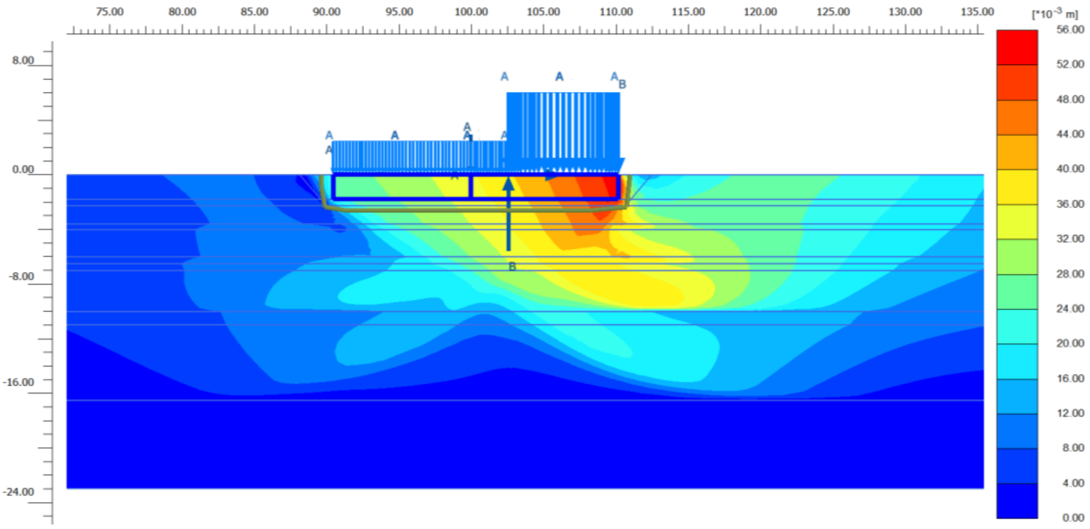

Figure 3 - Vertical displacements from the seismic pushover analysis case with liquefaction

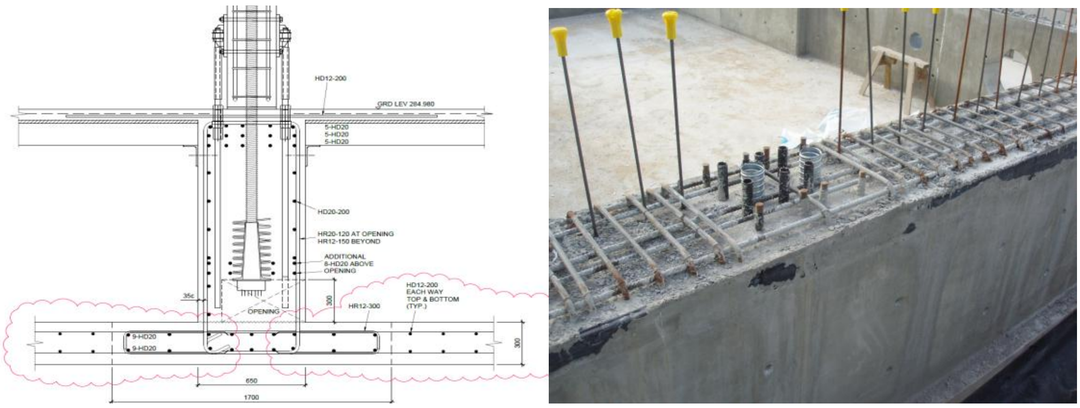

Figure 4 - Typical detail of the adopted RC raft foundation

2 PERFORMANCE BASED DESIGN OF A STRUCTURAL FOUNDATION ON GROUND PRONE TO LIQUEFACTION AND LATERAL SPREADING

2.1 Site conditions

Opus carried out geotechnical design and supplied other design services for the new Grey Base Hospital buildings in Greymouth The proposed development includes a three storey Hospital Building (Main Building), a new single storey Integrated Family Health Centre (IFHC) Building, a Store Building and other ancillary structures. The Hospital Building has been designed as an Importance Level 3 (IL3) structure and IFHC building as an Importance Level 4 (IL4) structure. The site consists of two approximately flat terraces (upper and lower) with an elevation difference of 5 m to 5.7 m. An estuary is adjoining the west side of the lower terrace. The estuary bed is 4 m to 5 m below the ground surface level of the lower terrace.

The site geotechnical investigations included boreholes, Cone Penetration Tests (CPTs), Multichannel Analysis of Surface Waves (MASW) surveys, trial pits, laboratory testing as well as a survey of the estuary bed levels. The site geology can be described as post glacial alluvium including river gravel, and swamp deposits. Groundwater table at the upper terrace is approximately 6 m below the ground surface level. Groundwater table at the lower terrace is approximately 2 m below the ground surface level. Our assessment of the available geotechnical investigation data indicated that a sand layer located between 8.2 m and 9.2 m depth below the ground surface level of the upper terrace has a potential for liquefaction. An approximately 1 m to 2 m thick layer of soft silt was encountered immediately beneath the sand layer. Where the silt’s plasticity is low, the silt is prone to liquefaction. Where the silt’s plasticity is high, the silt is prone to cyclic softening, i.e. substantial loss of strength in cyclic loading associated with earthquake shaking. Both, the sand and the silt layers were encountered in the lower terrace at approximately the same elevation, with the top of the liquefiable sand layer being at the depth of 2.5 m below the ground surface level at the lower terrace. If pile foundations are used, the top non-liquefiable soil crust located above the liquefiable soil layers, will apply negative skin friction forces to the piles. Given the proximity of the estuary and a 4 m to 5 m elevation difference between the lower terrace and the estuary bed, the site is also prone to lateral spreading. Our assessment based on Jibson (2007) indicated that the magnitude of lateral spreading for the ULS earthquake can vary from 150 mm to 600 mm at the building sites, with larger lateral displacements being closer to the estuary.

2.2 Geotechnical design

Ground improvement in the form of stone columns, deep mixing, high pressure jet grouting and CFA piles to stabilise the entire buildings’ footprints have been considered. However, the cost of ground improvement to fully stabilise the site and construct the building on shallow foundation would be high. Also, the shallow foundation option with improved ground would have inferior performance compared to a pile foundation. An option of improving a strip of ground between the estuary and the buildings was also considered. With this option, lateral spreading would be substantially reduced, however the buildings would need to be piled to reduce settlement. Pile foundations without ground improvement were adopted for the Hospital and IFHC Building, with a number of different options considered. The expected large lateral spread loads would generate substantial bending moments and shear forces in the piles. Construction of driven piles would generate vibration and noise and therefore driven piles were considered to be unacceptable for the operational hospital site. Therefore RC piles in the form of either bored or Continuous Flight Auger (CFA) piles have been adopted. The piles were designed to resist ULS lateral spread loads with some limited plastic hinging. The design of the piles was carried out in a conventional manner (using bi-linear soil springs) and also by using a more complex soil-structure interaction analysis.

In the conventional design approach, the full free field lateral spread displacement of the top non-liquefiable crust is applied to the base of the soil springs connected to the piles (Murashev et al., 2014). The actual lateral spread displacement, i.e. the displacement assessed for the site with structural piles in place, would be expected to be smaller than the free field displacement due to the reinforcing effect of the piles. The soil - structure interaction effects cannot be properly assessed using conventional design methods. Therefore, a more complex design methodology based on finite element analysis of soils and structural foundations was used.

A geotechnical model of the site soils (using Mohr - Coulomb constitutive model) and pile foundations was developed using the finite element (FE) computer program PLAXIS 2D. Initially, the expected free field distribution of the lateral spread ground displacements was generated by applying pseudo-static lateral volume forces representing horizontal seismic forces. After the free-field displacements were satisfactorily modelled, the FE model was amended to include the piles and pile caps. The reinforcing effect of the piles resulted in reduced lateral displacements of the foundation piles and reduced bending moments and shear forces in the piles (Figure 5 and 6).

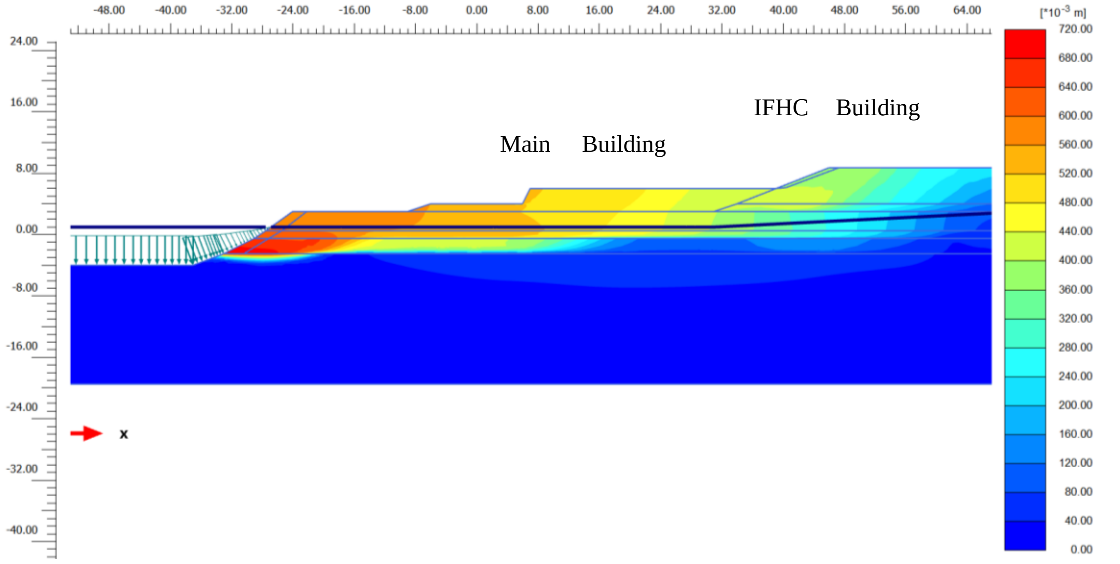

Figure 5 - PLAXIS free-field displacement for IL3 ULS event (displacements are 400 mm to 510 mm at the Main Building platform)

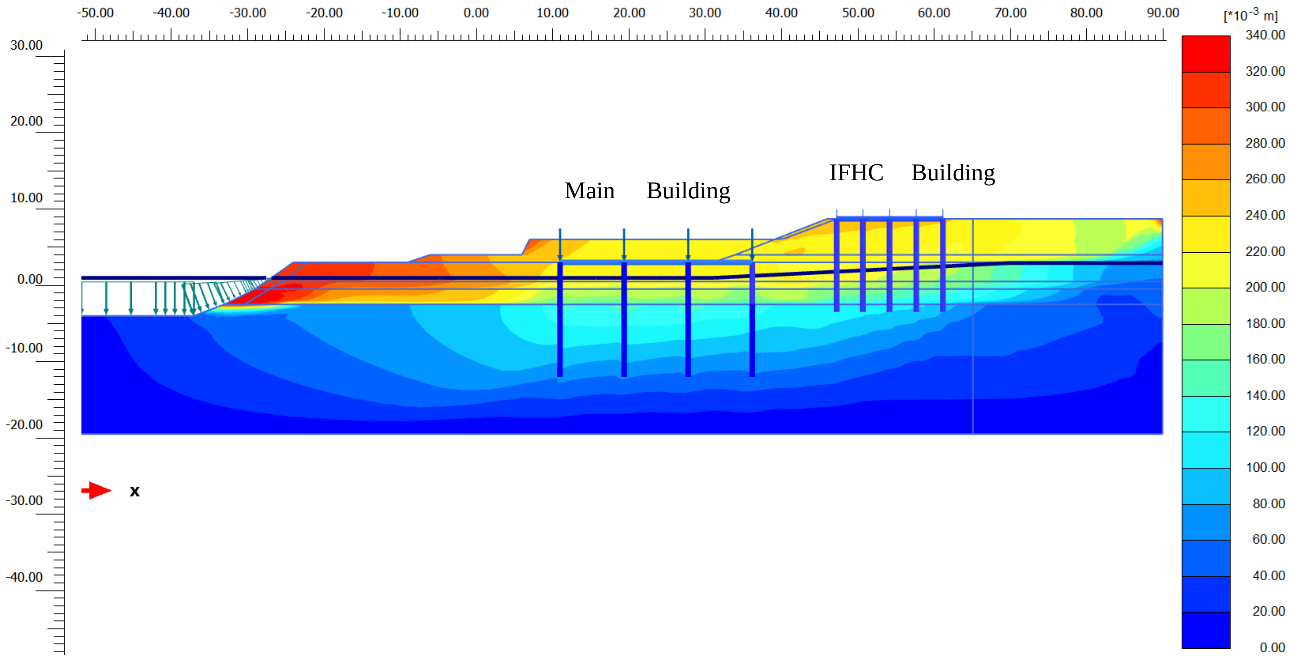

Figure 6 - PLAXIS displacements for IL4 ULS event (displacements IFHC building platform are 235 mm to 245 mm)

Pile design for the IFHC Building was carried out based on the reduced ground displacement, i.e. taking into account the reinforcing effect of the Hospital Building piles. Based on consideration of a number of pile foundation options with and without ground improvement, we concluded that the most cost-effective foundation option was to avoid ground improvement and support the Hospital Building on 1.0 m diameter, 15 m deep CFA or bored RC piles. The most cost effective foundation option for the IFHC building is to support the building on 0.6 m diameter 12 m deep CFA or bored RC piles.

3 CONCLUSIONS

Foundation systems comprising a reinforced concrete raft foundation over liquefiable soils with non-liquefiable crust and a RC pile foundation on a site prone to liquefaction and lateral spreading were designed to support the Rotorua Police Station and Grey Base Hospital buildings. The adopted design framework included conservative assumptions with respect to the site soils' potential for liquefaction and lateral spreading, and finite element analysis of soil-structure interaction. Performance based design utilised in the analysis resulted in substantial cost savings due to avoidance of ground improvement.

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