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Performance Based Design of a Structural Foundation on Liquefiable Ground with a Natural Soil Crust

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ABSTRACT

Seismic behaviour of buildings on liquefiable ground is affected by the size and stiffness of the structural foundation, level of contact pressure, 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 and structures located on liquefiable sites with a non-liquefiable crust can be satisfied without ground improvement. A foundation system comprising a reinforced concrete raft foundation over liquefiable soils with non-liquefiable crust was designed to support the recently completed Rotorua Police Station building. The adopted design framework included dynamic time-history finite element analysis of soil-foundation-superstructure interaction. Performance based design utilised in the analysis of dynamic soil-foundation-superstructure interaction resulted in substantial cost savings due to avoidance of ground improvement.

Introduction

Seismic behaviour of buildings on liquefiable ground is affected by the size and stiffness of the structural foundation, magnitude of contact pressure, 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. While it is common for geotechnical engineers to assess liquefaction-induced settlement of buildings based on free field settlement due to liquefaction (when only volumetric deformation is considered), this approach is deficient and can lead to deficient designs. The actual mechanism of settlement associated with liquefaction is much more 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. This effect is more prominent for tall buildings with high contact pressures as such structures generate high shear stresses in the liquefied soil during seismic shaking. 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

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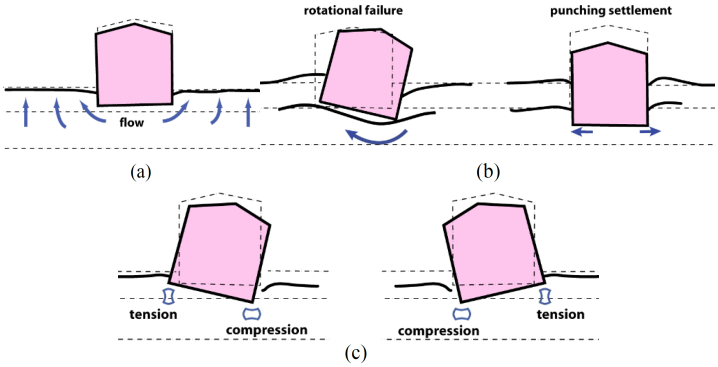

Figure 1 Liquefaction-induced mechanisms affecting settlement (Bray & Dashti, 2010)

While the mechanism of seismic settlement of structures on liquefiable ground is still a subject of extensive research, the available case studies from previous earthquakes as well as the existing empirical and analytical design procedures can be used to design foundation systems at potentially liquefiable sites. 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 (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.

Structures on Liquefiable Ground

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). 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.

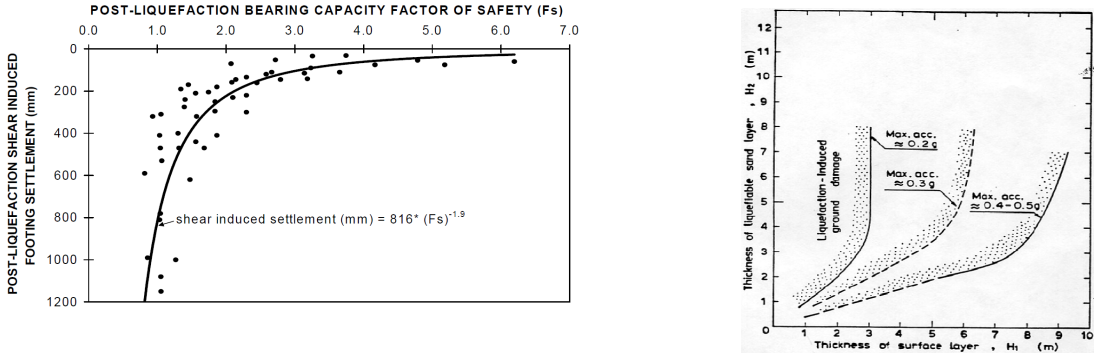

Figure 2 Correlation between post-liquefaction shear-induced settlement and post-liquefaction bearing capacity factor of safety, Naesgaard et al., 1998 (left); correlations for the assessment of liquefaction-induced damage, Ishihara, 1985 (right)

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.

Site Conditions

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 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**	**Soil description**	**Liquefaction for SLS2 & ULS events - depth range, m**
1	0 to 4	Interbedded pumiceous sandy gravel and silty sand	No liquefaction (above GWT)
2	4 to 7	Pumiceous silt	5.5-5.8; 6.5-6.8 (based on CPT)
3	7 to 11	Pumiceous silty sand with minor gravel	No liquefaction
4	11 to 16.5	Pumiceous sand	11 -16.5 (based on CPT)
5	16.5 to 25	Pumiceous 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, and that its use for assessing the liquefaction potential on pumice sands could thus lead to overly conservative results. 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 Bolanger (2008) and Kayen (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 & Idriss, 2001).

Results of cyclic triaxial tests reported by Orense et al (2012) show that alluvial pumice deposits, similar to those found at the Rotorua Police Station site, tend to possess cyclic resistance to liquefaction that is greater than other sands of comparable relative density. We note that Orense et al's test results are consistent with the results of our recent cyclic triaxial tests conducted for the Waikato Expressway Project, which indicate that cyclic resistance ratio of pumiceous sands and silts is 50-100% higher than that for hard-grained (quartz) sands (Jacobs & Dennison, 2015). 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. This assumption is conservative as the measured high shear wave velocities indicate that the site soils are non-liquefiable.

Geotechnical Analysis Framework

The following geotechnical analysis framework has been developed and used:

- Conservative 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
- Assessment of resistance to sliding of the structure under seismic load
- 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

Geotechnical Considerations

The expected static settlement of the raft foundation founded 1.8m below ground surface level is expected to be in the order of 10 mm. 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. The expected differential settlement associated with the subsidence (based on the analysis of soil profiles encountered by different CPTs) is in the order of 10 to 30 mm. According to Ishii and Tokimatsu (1988), for the large size of the foundation footprint (26 m x 80 m), the seismic settlement of the building should be approximately equal to the free-field settlement of the ground surface (90 – 120 mm for a 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 (static and seismic pushover analyses) are shown on Figures 3 and 4. The raft was assumed to be rigid in this analysis.

A foundation system comprising of a RC raft (with a grid of RC walls), similar to a cellular raft, as shown on Figure 5 was adopted by the geotechnical and structural designers. The structural designers analysed the stress – strain state of the raft using a Winkler model with soft by-linear soil springs reflecting behaviour of the liquefiable soils. Also, in this analysis, soil springs where removed within 4x4 m zones at various locations beneath the raft to model the effect of differential settlement of soils.

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.

Interface elements were used to simulate the soil-foundation interaction on the base of the raft foundation. Two material models were used for the dynamic analysis: the Mohr-Coulomb model (M-C) and the Hardening Soil model with small-strain stiffness (HSsmall).The stiffness values of the soils used in the model have been derived from CPT results. Liquefied soils were modelled as M-C materials where the stiffness values have been greatly (15 times) reduced from their pre-liquefied values and the shear strength has been reduced to the residual shear strength. The drainage type for all the materials used in this Plaxis model was assumed to be undrained, where calculations for the non-liquefiable and liquefied soil layers

were based on effective and total stress values respectively. While it is acknowledged that the modelling methodologies described hitherto constitute a relatively simplified approach, we believe there are sufficient details present in the model to provide for a satisfactory simulation on how the foundation system would likely behave when part of the underlying soils have suddenly undergone a drastic reduction in stiffness and strength, as is the case when liquefaction occurs in the soil.

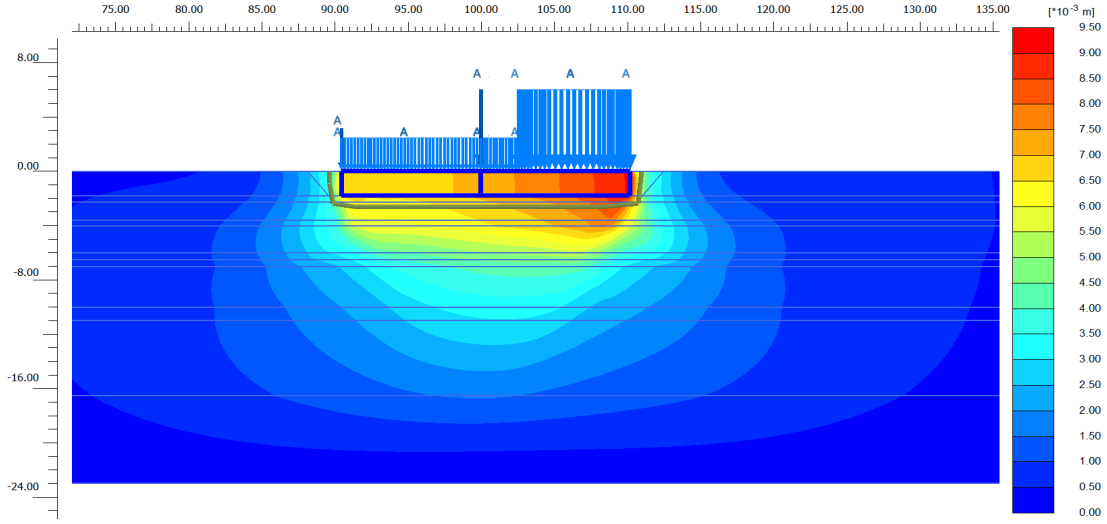

Figure 3 Vertical displacements for the static load case

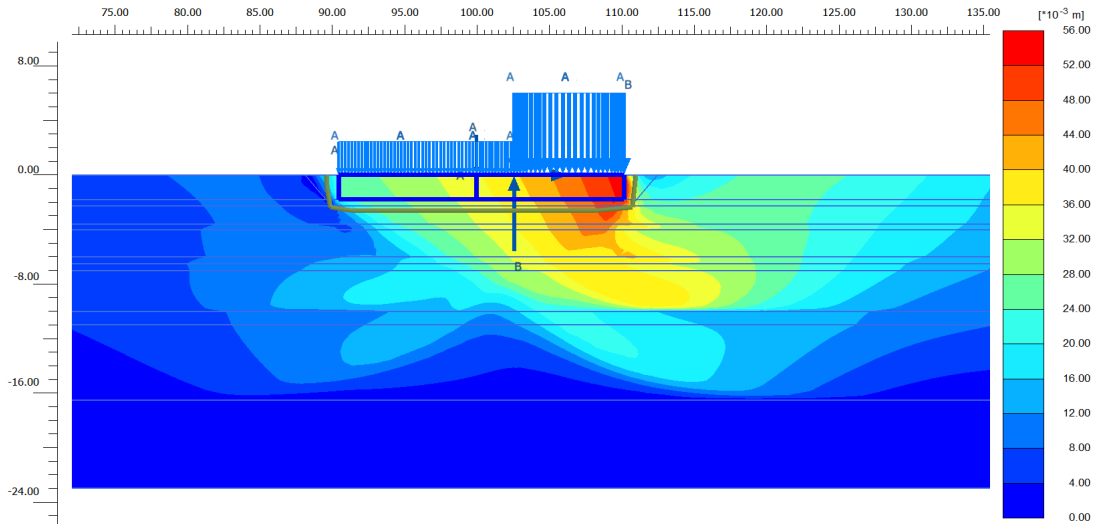

Figure 4 Vertical displacements from the seismic pushover analysis case with liquefaction

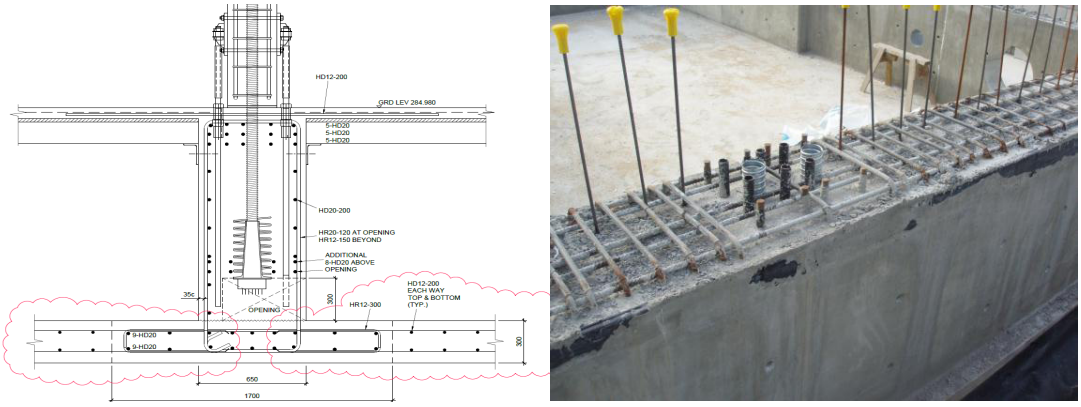

Figure 5 Typical detail of the adopted RC raft foundation

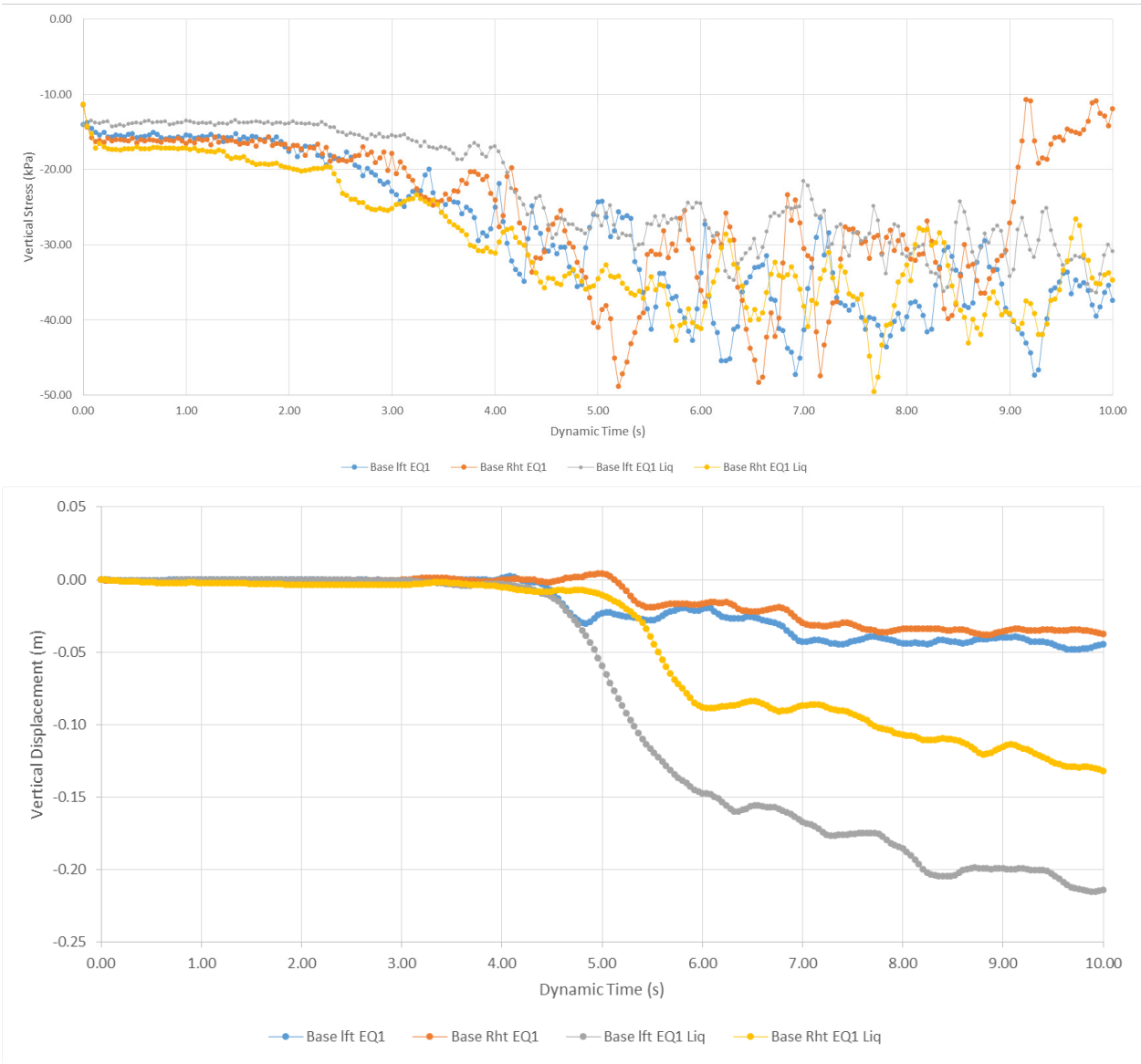

Figure 6 Vertical stress and settlement at the corners of the raft foundation for 1989 Loma Prieta earthquake time history: orange & blue – no liquefaction; yellow & pale – with liquefaction

Typical results of the time-history analysis are shown in Figure 6. 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).

Conclusions

A foundation system comprising a reinforced concrete raft foundation over liquefiable soils with non-liquefiable crust was designed to support the recently constructed Rotorua Police Station building. The adopted design framework included conservative assumptions with respect to the site soils' potential for liquefaction and dynamic time-history finite element analysis of soil-foundation-superstructure interaction. Performance based design utilised in the analysis of dynamic soil-foundation-superstructure interaction resulted in substantial cost savings due to avoidance of ground improvement.

Acknowledgments

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