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# Design of pump station foundations by numerical modelling – Part 2

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## ABSTRACT

The Jeffreys water supply pump station comprises replacement of the earthquake-damaged 200m$^3$ concrete suction tank with a new 500m$^3$ tank. Ground conditions at the site predominately comprise interbedded alluvial silt and sand in the surficial 8.75m, underlain by dense sandy gravel. Surficial soils are susceptible to liquefaction triggering during earthquake shaking and the site is bounded to the south by a small stream. Subsequent to the quasi-static advanced design incorporating residual properties for the liquefied materials (using PLAXIS 2D), a more advanced numerical modelling approach is undertaken with FLAC 2D and the PM4Sand & PM4Silt constitutive models. The plasticity models are calibrated at a soil element level using data from the existing CPT and boreholes. Time-history analyses considering a Christchurch CBD Class D strong motion event from 22 February 2011, deconvoluted to the site stratigraphy is undertaken with and without ground improvement/ support elements in place, and results of soil and structural performance are presented. The more advanced analysis concludes that the proposed interlocking lattice-wall ground improvement is compromised during a similar to the February 2011 earthquake event by lateral displacements of the soils at the site. Further foundation options are considered and conclude that large-diameter pile elements provide superior resilience for water supply infrastructure at the site.

## 1 INTRODUCTION

Providing resilient foundations and robust geotechnical design has become a key aspect of water supply infrastructure in the Canterbury area. This paper reviews the geotechnical assessment and foundation design for a rectangular 500m$^3$ suction tank and associated infrastructure located in Fendalton, Christchurch. Various design scenarios are considered during the assessment, the critical case emerges that the tank must remain in service after any significant future seismic events. As such, this paper will focus on the assessment of seismic ground performance at the site, and the subsequent design methodologies that are applied to ensure a resilient foundation design.

For clarity, this review is divided into two papers. The intent is to present results and conclusions of two industry accepted design solutions as follows:

1. An assessment based on a simplified liquefaction triggering analysis and pusedo-static ground conditions utilising the finite-element software PLAXIS 2D (Bentley Systems, 2019), and;
2. An assessment based on calibrated plasticity soil constitutive models that account for liquefaction and strain-softening, and an appropriate time-history earthquake record utilising the finite-difference software FLAC (Itasca Consulting Group, 2019).

Part 2 of this assessment is presented herein.

### 1.1 Project Details

The Jeffreys Pump Station is located at the southern end of Jeffreys reserve, an existing 200m$^3$ capacity suction tank was damaged during the February 2011 earthquake and is currently offline. The purpose of the project is to replace the suction tank with a rectangular (15.2m x 11.5m) reinforced concrete 500m$^3$ capacity suction tank and carry out all necessary works to ensure secure water supply from the pump station during the design life.

A preliminary geotechnical assessment identified that soil strata at the site are susceptible to liquefaction triggering, and a foundation solution comprising ground improvement by installation of cement grout columns in a rectangular lattice beneath the suction tank is preferred.

### 1.2 Site and Ground Conditions

The pump station is located at the southern end of Jeffreys Reserve and bounded by residential property to the west and east, and the Wairarapa Stream to the south, as shown in Figure 1.


*Figure 1. Site location (Google Earth, 2019)*

An investigation programme comprising six cone penetration tests (CPT) and one machine borehole, located at various locations across the site consistently shows interbedded alluvial silt and sand in the upper 8.75m, underlain by dense sandy gravel greater than 10m in thickness. The ground water table is set at 1.0m below ground surface for the assessment. A generalised soil model and basic soil parameters based on CPT assessment and interpretation by commercially available software CPeT-IT (Geologismiki, 2006) are presented in Table 1. The $q_c$ values are related to $(N_1)_{60}$ using the relationship suggested by Robertson (2012). The $(N_1)_{60}$ values are preferably used to calibrate and set up the plasticity models for sand and silt discussed in the following section.

*Table 1: Generalised ground conditions*

| Layer No. | Depth to Base (m) | Thickness (m) | Description | Representative $(N_1)_{60}$ |
|---|---|---|---|---|
| 1 | 1.25 | 1.25 | Silt and clay | 5 |
| 2 | 3.40 | 2.15 | Dense sand and gravel | 49 |
| 3 | 4.70 | 1.30 | Silt and clay | 4 |
| 4 | 6.05 | 1.35 | Loose sand | 19 |
| 5 | 6.35 | 0.30 | Silt and clay | 6 |
| 6 | 7.05 | 0.70 | Loose sand | 18 |
| 7 | 7.45 | 0.40 | Silt and clay | 6 |
| 8 | 8.10 | 0.65 | Loose sand | 15 |
| 9 | 20.00 | 11.90 | Dense sand and gravel | 50 |

# 2 METHODOLOGY

## 2.1 2D Time History Modelling

Fast Lagrangian Analysis of Continua (FLAC) is a finite difference formulaic model that can model complex soil, rock, groundwater, and ground support in 2D. The most informative site cross-section has been selected, in terms of structures of interest, and comprises a section approximately North-South through the location of the suction tank and adjacent Wairarapa Stream. The soil layers that were found as prone to liquefaction and strain softening have been modelled using the PM4Sand and PM4Silt constitutive models respectively (version 3.1 and 1.0, Boulanger & Ziotopoulou, 2018) as outlined in the below methodology.

- Calibration of the PM4Sand and PM4Silt constitutive models for all soil elements to their appropriate stiffness and strength to allow for liquefaction and strain softening respectively;
- Application of the calibrated constitutive model per soil layer;
- Application of the tank loads;
- Application of the Christchurch Botanic Gardens (CBGS) February 2011 earthquake Class D record as a realistic time history at the base of the model;
- Run the model without the proposed ground improvement/foundation solution;
- Assessment of the kinematic effects against the proposed ground improvement solution (CFA lattice);
- Selection of suitability reinforced pile element to withstand the assessed kinematic demand; and
- Run the model with a reinforced pile arrangement.

The non-liquefiable layers (layer 1, 2, and 9) were modelled with the Mohr-Coulomb constitutive model in terms of shear strength and hysteretic damping to simulate the cyclic modulus – strain changes and damping.

FLAC was selected in lieu of PLAXIS because the latter does not yet have the capability to allow for a user defined hysteretic damping. PLAXIS manual stipulates that as the Hardening Soil model with small-strain stiffness (HSsmall) incorporates the loading history of the soil and a strain-dependent stiffness, it can, to some extent, be used to model cyclic loading. However, it does not incorporate a gradual softening during cyclic loading, so is not suitable for cyclic loading problems in which softening plays a role. In fact, just as in the Hardening Soil model, softening due to soil dilatancy and debonding effects are not taken into account. Moreover, the HSsmall does not incorporate the accumulation of irreversible volumetric straining nor liquefaction behaviour with cyclic loading.

The model runs produce ground deformations and the soil-structure-interaction is monitored in terms of deformations, strains, forces, bending moments, etc.

## 2.2 Soil Parameters - Calibration

The plasticity models for sand and silt require a series of parameters. These are grouped in two categories; a primary set that requires proper calibration, and a secondary set that are usually not modified. The sand and silt layers were assessed, and special calibrations were carried out to populate the first set of parameters using the guidelines and the driver programs provided by the model authors. These primary properties are:

- Apparent relative density, $D_R$: Primary variable controlling dilatancy and stress-strain response characteristics.
- Shear modulus coefficient, $G_0$: Primary variable controlling the small strain shear modulus, $G_{max}$.
- Contraction rate parameter, $h_{p0}$: Primary variable that adjusts contraction rates and hence can be adjusted to obtain a target cyclic resistance ratio, as commonly estimated based on CPT or SPT penetration resistances and liquefaction correlations.
- Undrained shear strength ratio, $s_{u,cs,eq/\sigma'vc}$ that is used to compute $s_{u,cs,eq}$ from the $\sigma'_{vc}$ at "consolidation" (i.e., at the time of model initialization). The undrained shear strength, $s_{u,cs,eq}$, is specified undrained shear strength at critical state for the current void ratio.

The calibrated primary set of plasticity model properties per soil layer where they were applied are presented in the below Table 2.

*Table 2: Primary soil parameters for the PM4Sand and PM4Silt plasticity models*

| Layer No. | Apparent Relative Density $D_R$ | Shear Modulus Coefficient $G_0$ | Contraction Rate Parameter $h_{p0}$ | Undrained Shear Strength Ratio $s_{u,cs,eq/\sigma'vc}$ | Plasticity Model |
|---|---|---|---|---|---|
| 3 | | 404 | 60 | 0.73 | PM4Silt |
| 4 | 0.44 | 530 | 0.65 | | PM4Sand |
| 5 | | 453 | 60 | 0.71 | PM4Silt |
| 6 | 0.43 | 515 | 0.65 | | PM4Sand |
| 7 | | 455 | 50 | 0.63 | PM4Silt |
| 8 | 0.43 | 506 | 0.60 | | PM4Sand |

### 2.3 Strong Motion Record

The Christchurch Botanic Gardens (CBGS) February 2011 earthquake Class D record was selected as a realistic time history to be applied at the base of the model. The record was deconvoluted as an outcrop motion to the base of the model and baseline corrected. A 1.25 numerical scaling factor was applied. The unscaled record's plot is shown in the below Figure 2.

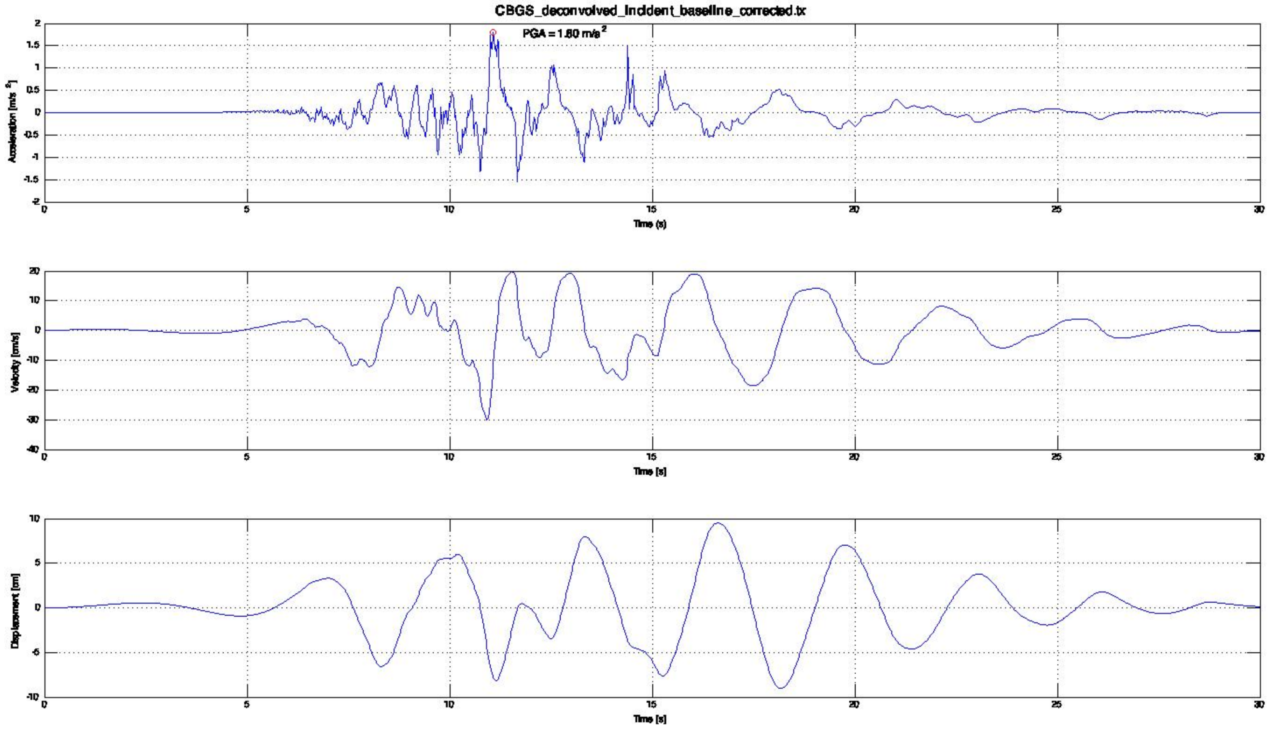



*Figure 2: Christchurch Botanic Gardens (CBGS) February 2011 earthquake Class D record, deconvoluted and baseline corrected*

## 3 RESULTS

Running of the calibrated constitutive model with the February 2011 earthquake CBGS time history record indicates liquefaction being triggered in loose sandy soil strata from 4.70m to 8.75m below existing ground surface and partial strain softening of the in-between silt layers. The phenomenon is present under all loading cases, i.e. free field, with imposed structural loads with and without piles.

The kinematic effects (soil displacement) for a no ground improvement option are such that a horizontal soil displacement component is developed in the order of over 0.30m (Figure 3). With the addition of the reinforced concrete shaft piles this behaviour is damped but it concentrates to the lower sand layers 6 and 8 (Figure 4).

It is important to notice that in all cases examined a permanent differential horizontal displacement develops within the liquefied/strain softened zone which is about 0.20m. This is not expected to be visible on ground surface as it is capped by a dense sand layer (approximately 4m thick). This displacement pushes any non-reinforced concrete element to failure. Hence, piles with adequate shear and bending capacity are selected as the updated foundation solution as a result of the numerical analysis. These are expected to tilt within tolerance under a similar event and to be functional in the future.

We have also undertaken an additional FLAC model simulating a minor earthquake time history (by applying a numerical scaling of 0.8 to mimic ground response close to a SLS event. While lateral ground displacements were reduced, the magnitude of the displacement and corresponding kinematic soil load on unreinforced vertical elements pushes them to failure.

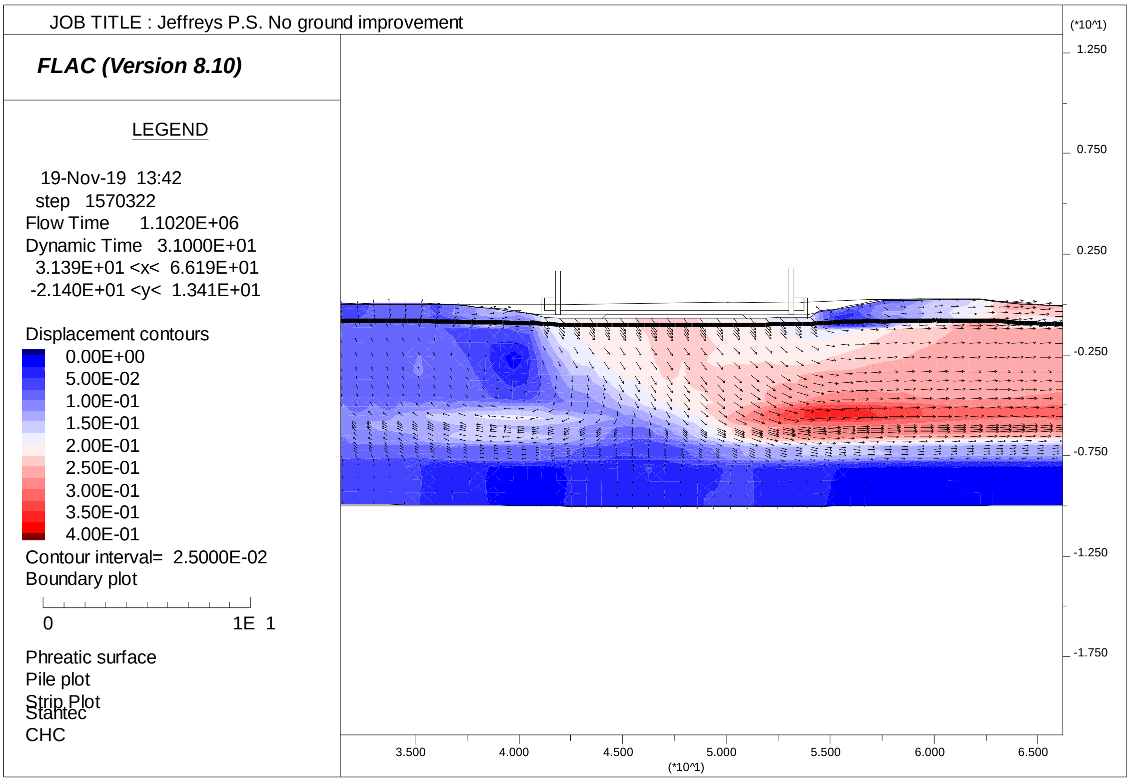



*Figure 3: Finite-difference model representing the end of the time history analysis without ground improvement over the North-South section (FLAC, 2019)*

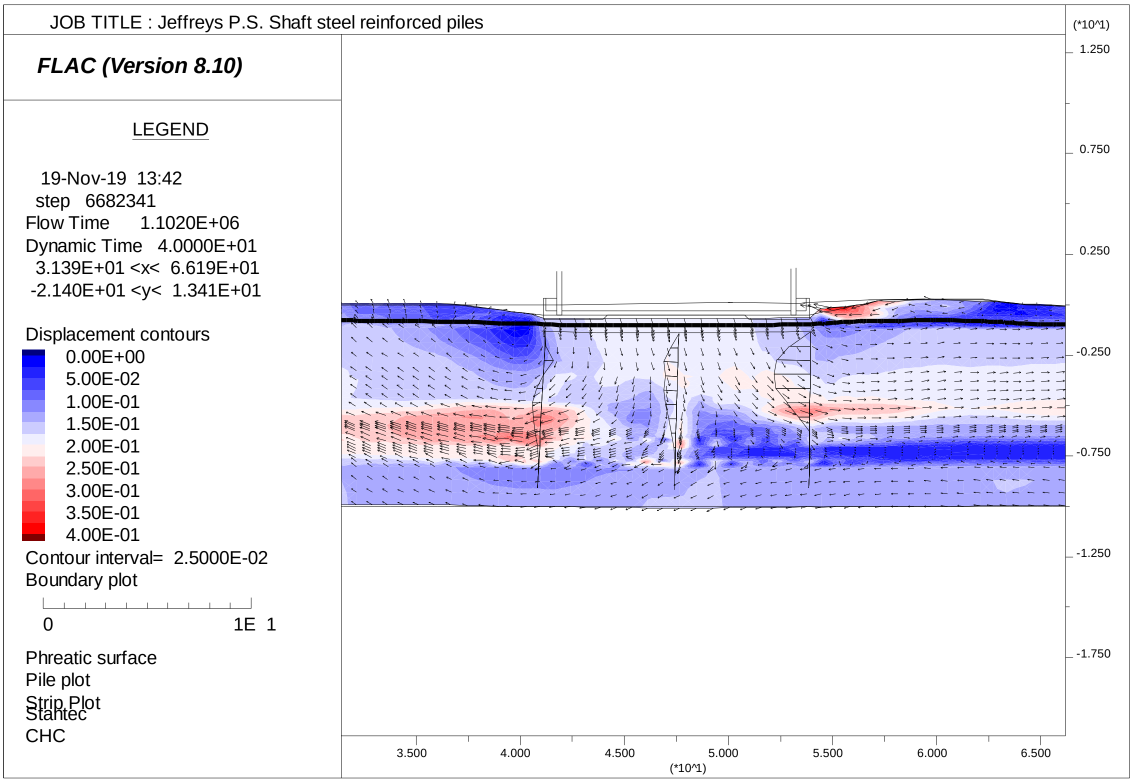
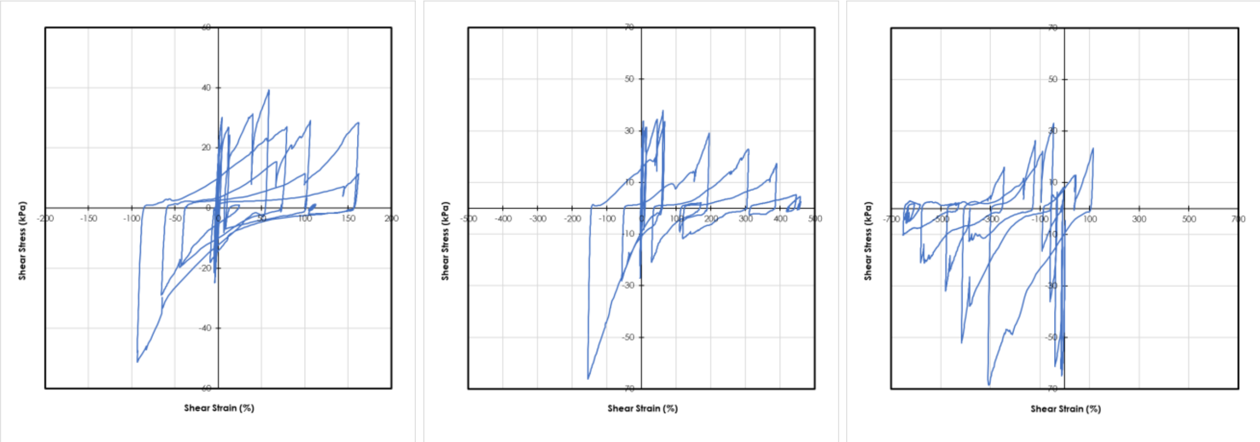


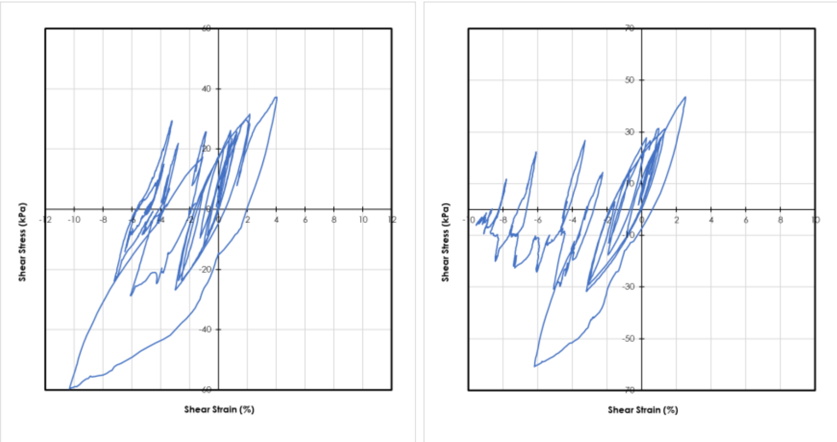

*Figure 4: Finite-difference model representing the end of the time history analysis with load bearing shaft piles over the North-South section (FLAC, 2019)*

The layers that liquefied during the time history simulation were Layer 4, Layer 6, and Layer 8. The layers that exhibited strain-softening effects were Layer 5 and Layer 7. Their stress-strain traces for the shaft pile case are plotted in Figures 5 and 6 respectively.




*(a)* *(b)* *(c)*

*Figure 5: Liquefied layers - plots of shear strain vs. shear stress for Layer 4 (a), Layer 6 (b), and Layer 8 (c)*




*(a)* *(b)*

*Figure 6: Strain-softened layers - plots of shear strain vs. shear stress for Layer 5 (a), and Layer 7 (b)*

## 4 DISCUSSION – CONCLUSIONS

It is important to note that that lateral ground movements at the site are not the result of the phenomenon widely known as lateral spreading or lateral stretch. Lateral deformations are the result of the site-specific layering of liquefiable and non-liquefiable soil strata, and their performance during cyclic earthquake shaking. Lateral movement between these layers create a situation where any vertical structural elements embedded in the soil become subject to large shear forces from the kinematic soil loads.

By distinction, lateral spreading as defined by Rauch, 1997 comprises the finite, lateral displacement of gently sloping ground, or ground adjacent to a free face because of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. This phenomenon typically results in cracking and significant localised lateral movement at the ground surface. No observations of lateral spreading were recorded at the Jeffreys Pump Station site during historic earthquakes and this is confirmed by the FLAC time history plastic modelling.

The magnitude and location of lateral ground movement are such that an un-reinforced concrete-based lattice ground improvement is unlikely to provide the requisite resilience for the structures at the site. Primary concerns are that at least the external walls will not survive a design earthquake event, this in turn would render the ground improvement ineffective in any future event and any remediation would be inhibited by the presence of the damaged lattice walls below the structures.

It is also important to note that a the pseudo-static 2D finite element analysis discussed in Part 1has not identified lateral displacement and associated kinematic loading of soil at depth during earthquake shaking. Furthermore, the piles were assessed to have adequate shear and bending capacity to withstand lateral ground movements and associated kinematic loads. Piles are expected to tilt within tolerance during a ULS earthquake event and to be functional in the future.

A review of both sets of assessments i.e. Parts 1 & 2 of this review, a reinforced concrete bored (shaft) pile foundation solution, socketed a minimum of 2.00m into the dense sandy gravel at approximately 9.00m below ground surface, was recommended to be adopted. We believe this provides the greatest level of resilience during future earthquake events while meeting other project requirements such as speed of construction, non-impact or non-vibratory installation techniques, and cost. It also has the benefit of being well understood foundation construction method by local contractors and robust QA procedures can be put in place to ensure piles meet all design criteria.

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