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Design of pump station foundations by numerical modelling – Part 1

M.J. Burrows & I. Antonopoulos
Stantec New Zealand, Christchurch.

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. Preliminary design of the suction tank foundation utilises machine borehole and CPT investigation data to inform a simplified 1D liquefaction triggering assessment, and selection of suitable foundation options. This concludes that ground improvement by installation of cement grout columns is a suitable foundation solution of the suction tank. Detailed design proceeds based on material properties and liquefaction triggering from the 1D assessment and utilizes PLAXIS 2D to arrange an interlocking lattice-wall arrangement beneath the pump station. PLAXIS 2D is used to assess the performance of the proposed suction tank and soil at the site. Analysis concludes that the proposed ground improvement is suitable for the structures at the site, and discusses the limitations of simplified liquefaction triggering methods, and the potential for soil kinematic loads affecting structural elements.

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 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 pseudo-static ground conditions utilising the finite-element software PLAXIS 2D (Bentley Systems Inc, 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 1 of this assessment is presented herein.

1.1 Project Details

The Jeffreys water supply 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 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. A generalised soil model and representative $(N_1)_{60}$ values are presented in Table 1. $(N_1)_{60}$ values are included to give perspective on soil density and have been interpreted from the available CPT trace files using commercially available software CPeT-IT v2.0.1.61 (Geologismiki, 2007). Groundwater has been assessed at approximately 1.0m below ground surface.

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

1.3 Ground Performance During Historic Earthquakes

The New Zealand Geotechnical Database (NZGD) comprises ground deformation mapping observed during the Canterbury Earthquake Sequence (CES). Figure 2 shows a graphic representation of total vertical ground settlement associated with all events of the CES.

Figure 2: Vertical ground movements at the Jeffreys Reserve and surrounding area after all events associated with the CES (Canterbury Geotechnical Database, 2012)

The average approximate settlement after all events at the site is approximately 0.15m and is considered indicative of the free field settlement after all events. No evidence of lateral spread or ground cracking adjacent to the Wairarapa Stream were noted during inspection after significant earthquake events (Canterbury Geotechnical Database, 2012 and 2013). Inspection of the damaged suction tank after the earthquake event in February 2011 showed the tank settled by approximately 0.09m.

2 METHODOLOGY

2.1 Liquefaction Triggering Assessment

A simplified, one-dimensional (1D) assessment of liquefaction triggering and calculation of associated free-field liquefaction induced settlements are undertaken utilising the commercially available software CLiq (Geologismiki, 2006) and the methodologies set out by Idriss & Boulanger (2014) and Zhang et al. (2002). The ground water table is set at 1.0m below ground surface for the assessment.

The assessment identifies liquefaction triggering in layers 4, 6, and 8, corresponding to the loose sand layers situated below the ground water table, and calculates total settlements of 110 – 140mm during a ULS earthquake event (PGA = 0.61, Mw = 7.5).

2.2 Soil Parameters

Soil parameters are assessed using CPeT-IT, various published correlations, and the authors professional experience with similar soils and ground conditions to use with the Mohr-Coulomb constitutive model.

Table 2: Soil parameters and response to liquefaction triggering

Layer No.	Sat. Unit Weight (kN/m^3)	Effective Friction Angle (°)	Effective Cohesion (kPa)	Average Young's Modulus (kPa)	Liquefaction Triggering
1	16.6	28	10	5,000	No
2	19.0	38	-	38,500	No
3	16.2	28	10	12,400	No
4	17.9	31	-	30,400	Yes
5	16.7	28	10	17,200	No
6	17.6	31	-	31,700	Yes
7	16.8	28	10	19,100	No
8	17.5	31	-	31,400	Yes
9	19.0	38	-	75,000	No

2.3 Numerical Model

The intent of the numerical model is to simulate the site response (including structures) to earthquake shaking by employing a staged analysis to replicate (1) static ground conditions prior to earthquake shaking, and (2) those present at the onset of liquefaction. The primary damage mechanism to the existing tank at the Jeffreys Pump Station site was a differential settlement of approximately 90 mm across the foundation. In order to capture the damaging effects of liquefaction, stage 2 of the numerical model assumes that liquefaction has already triggered, and the deposit is allowed to undergo post-liquefaction deformations. The calibration of constitutive soil models to realistic residual soil strengths is outlined below.

A drained Mohr-Coulomb type analysis is utilised in PLAXIS 2D and soil layers are applied as per Tables 1 & 2. In order to capture topographical effects, in particular the presence of the Wairarapa Stream, two sections are analysed one running approximately north-south and one east-west. A staged construction model allows for assessment of free-field conditions and after installation of the ground improvements.

In order to define appropriate Mohr-Coulomb material parameters for layers that are identified as liquefiable, the model is calibrated using the observed tank and ground performance during the CES, and according to guidance set out in “Seismic effective stress analysis: modelling and application” (Cubrinovski, 2011). Residual soil parameters are selected that concur with a settlement of approximately 0.1m at the ground surface, based on calculated and observed ground displacement during historic earthquake events. Soil permeabilities and groundwater flow conditions are assessed by the PLAXIS 2D standard function which allows for selection of the most common soil types and is based on the Hypres classification series (Wosten et. al., 1999). The fine-grained soil model was selected for silt and clay layers, and coarse for sand and gravel.

Structural elements are introduced to the calibrated ground model including 0.6m thick cement grout walls, extending to the layer of dense gravel and spaced at approximately 5.0m centres (based on the work of Nguyen et al, 2013). A 1.25m thick geogrid reinforced gravel raft is introduced to replace the surficial weak silt and clay material, on which suction tank foundations directly bear. Structural inertial loads are applied corresponding to a ULS design earthquake, magnitude of the load is assessed to be maximum 80 kPa and is distributed across the foundation plate element accordingly. As the load from the tank during earthquake loading is cyclical, a pseudo static worst-case is shown where the load is concentrated towards the perimeter footing. Model geometry, soil layers, and structural elements are shown in Figure 3.

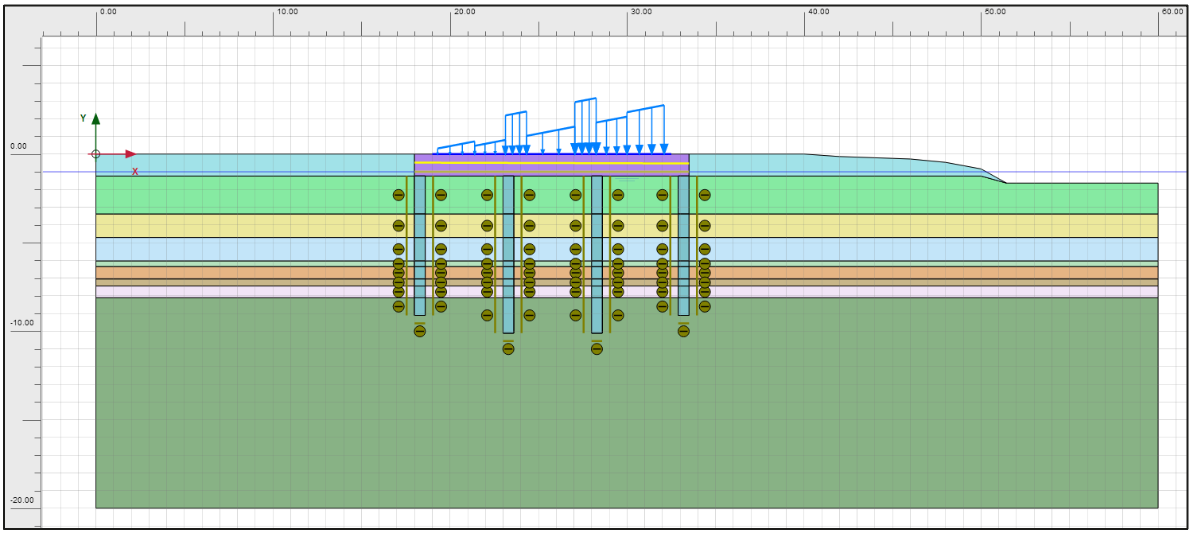

Figure 3: Finite-element model representing the north-south section (PLAXIS 2D, 2019)

Structural elements shown in Figure 3 above are itemised as follows:

Purple horizontal foundation	Engineered gravel fill
Yellow strip within purple foundation	High strength geogrid
Blue vertical foundation	Grout lattice walls
Beige perimeter on lattice walls	Soil-structure interface elements

3 RESULTS AND DISCUSSION

The model is run to completion for both the north-south and east-west sections and interrogated to show performance of soil and structural elements. Figures 4 & 5 present vertical settlements, showing anticipated free-field settlement of 0.1m while mitigating settlements at the location of the pump station to less than 0.025m.

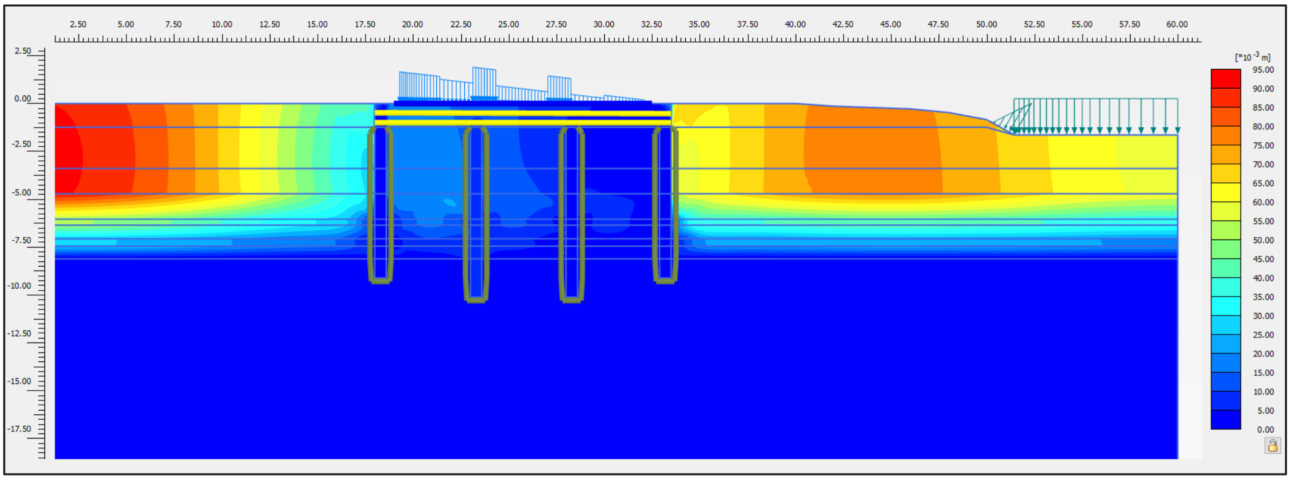

Figure 4: Vertical ground displacements through north-south section (PLAXIS 2D, 2019). Note load on right of figure represents the adjacent stream and water load.

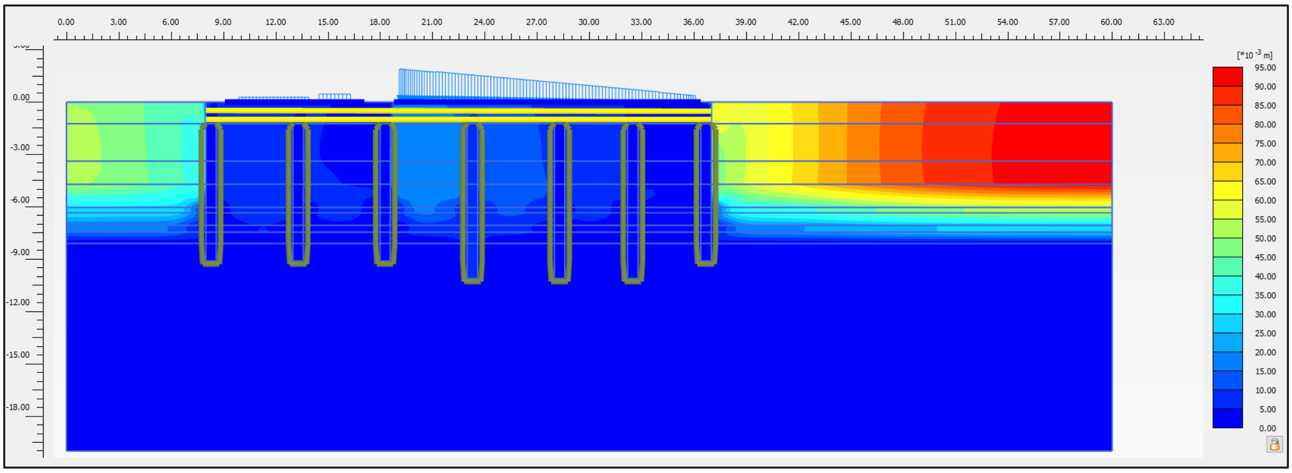

Figure 5: Vertical ground displacements through east-west section (PLAXIS 2D, 2019).

The model is further interrogated to confirm integrity of the cement grout lattice walls and shows that these remain intact after earthquake structural inertial loads and pseudo-static soil conditions are applied. Detailed deflection of the plate foundation shows differential settlements are less than 0.015m. Horizontal displacements at all points on the models are less than 0.020m, this includes any lateral movement of the stream bank towards the free face.

Based on pseudo-static assessment utilising PLAXIS 2D, the design forms a reasonable foundation solution where vertical and lateral deflections are within structural tolerance, and all structural element remain sound.

However, it is important to note the limitations of this assessment, firstly, simplified liquefaction methods only allow for post-liquefaction reconsolidation settlements in level ground free-field deposits. There is pronounced non-uniformity of liquefaction effects and consequent ground deformation, so it is rational to consider the calculated settlements based on the simplified methods only as a proxy for the damaging effects of liquefaction rather than a reliable estimate of ground settlement (NZGS, 2019). As such, calibrating the model to settlements assessed by empirical estimates may be considered a proxy only.

Secondly, a pseudo-static approach does not allow for the cyclic earthquake actions to impart kinematic soil loads on structural elements. Kinematic actions from liquefied soils, and importantly non-liquefied soils that retain significant shear strength during and after earthquake shaking may attribute considerable lateral forces on vertical elements such as the cement grout walls. The unreinforced cement grout walls have limited capacity to cope with shear forces generated by kinematic soil loads. This is considered a key risk for the suitability of the ground improvement to function through multiple earthquake events at this site.

4 CONCLUSIONS

Considering the PLAXIS 2D results we conclude that this type of modelling is not addressing the kinematic behaviour of the ground during earthquake shaking for this particular site. As such, the results obtained reflect a case which exists only when the worst-case load combinations and residual soil characteristics align. In addition, this may be considered intrinsically unrealistic because it does not show the interaction between the improved and green-field (unimproved) areas, or it doesn't account for kinematic variability during an earthquake event and soil reconsolidation after the significant duration of that event.

As such, the authors have undertaken a further 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 and is presented in Part 2 of this paper.

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