
Challenges associated with geotechnical design of pump stations in Tauranga

A.K. Murashev, P. Tan & P. Alves

WSP, Wellington, New Zealand.

ABSTRACT

The Opal Drive and Wairākei wastewater pump stations were constructed as part of Tauranga City Council's network resilience upgrades. Both sites are underlain by deep Holocene sands with high groundwater, creating significant liquefaction, settlement, and lateral spreading hazards. To meet IL2 and IL3 seismic performance requirements, a dual foundation system was adopted incorporating non-vibratory stone column ground improvement paired with deep bored piles.

This paper outlines the design approach, verification processes, and construction controls required to deliver compatible ground improvement and foundation systems in highly liquefiable soils. Emphasis is placed on transferable design principles, including minimising construction-induced disturbance, forming a non-liquefiable crust for surface structures, managing soil-structure interaction effects, and implementing monitoring frameworks to validate performance. These lessons provide a practical foundation for geotechnical design of critical infrastructure in similar environments across New Zealand.

1 PROJECT BACKGROUND AND CHALLENGES

The Opal Drive and Wairākei pump stations are located on the Papamoa coastline, where the subsurface profile is dominated by loose to medium-dense Holocene Ocean Beach Deposits comprising sands with intermittent fine soils. These deposits are highly susceptible to liquefaction during strong seismic shaking, extending to depths greater than 30 m, and are underlain by very dense sand and gravel of the Papamoa Formation Ignimbrite. Groundwater levels are consistently high across both sites, often occurring within 1–2 m of the ground surface, contributing to the liquefaction hazard. The shallow groundwater table also introduced construction-stage challenges, particularly the requirement for dewatering during excavation, which could cause dewatering-induced settlement. Consequently, the liquefaction hazard poses a significant risk to nearby structures and underground services, with potential for ground deformation and loss of support during a major seismic event.

In addition to liquefaction, the geological conditions (loose-medium dense sand) create elevated risks of construction-induced ground movement, particularly vibration-induced settlement. The loose saturated sands are highly sensitive to vibratory energy from construction activities resulting in soil settlement that may affect buried services, and nearby buildings. This hazard was a significant constraint during the design and construction planning.

At the Wairākei site, the risk profile is further intensified by its location adjacent to the Wairākei Stream, where a free-face condition increases susceptibility to lateral spreading under seismic loading.

These combined hazards created a challenging environment that required a design capable of mitigating ground deformation associated with both seismic loading and construction activities. This context formed the basis for adopting a dual foundation system integrating non-vibratory ground improvement and deep bored piles designed to minimise seismic structural damage and ensure operational requirements are met

2 SITE DESCRIPTION

The pump stations are comprised of ancillary structures including the terminal chamber, storage tank, biofilter building, generator, and control rooms. Figure 1 shows the Wairākei pump station site layout, illustrating the arrangement of these structures and the designated ground improvement areas (shaded purple for 10 m deep stone columns and blue for 13.5 m deep stone columns). The site layout for Opal Drive pump station is similar in configuration to Wairākei pump station.

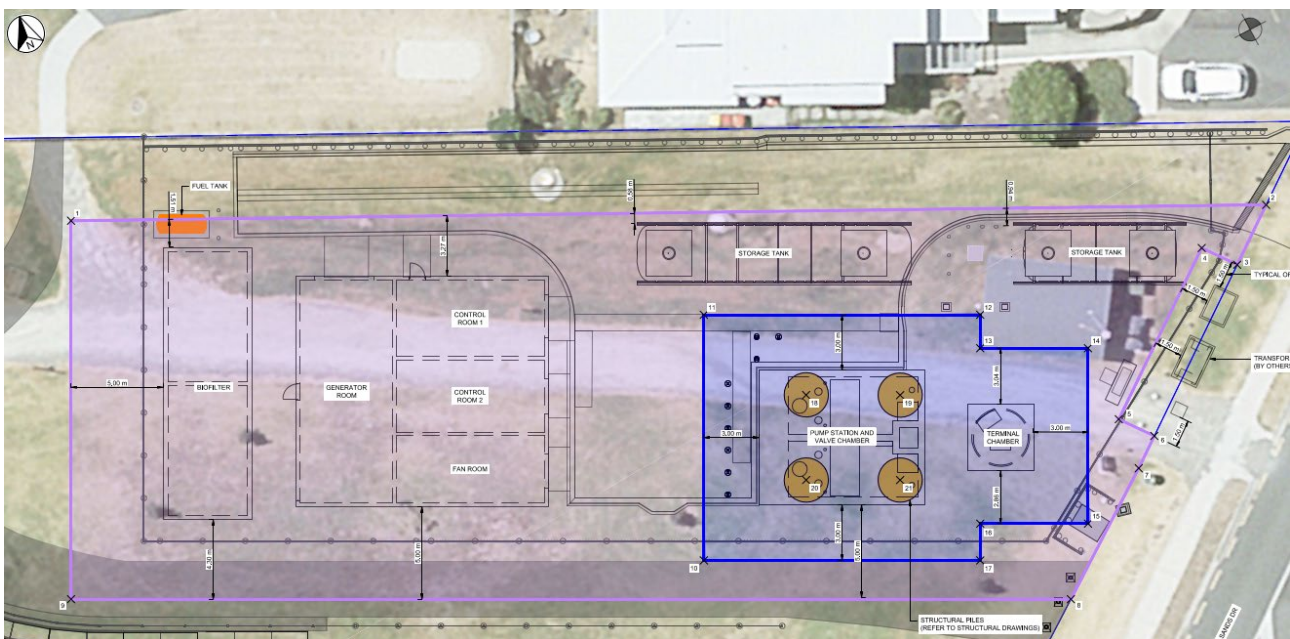


Figure 1: Wairākei pump station site layout and ground improvement plan

A ground model was developed using data from the site investigations and supporting geological references. Figure 2 presents the representative long section for the Wairākei pump station, illustrating the vertical arrangement of the key subsurface units, the depth of the target ground-improvement zone, and the position of the dense underlying formation that controls pile embedment. Although not shown in the figure, the ground surface gently slopes toward the Wairākei Stream, which is relevant to understanding the site's lateral-spreading susceptibility. The long section therefore provides a concise visual summary of the stratigraphic framework that informed both the ground-improvement and foundation-design considerations.

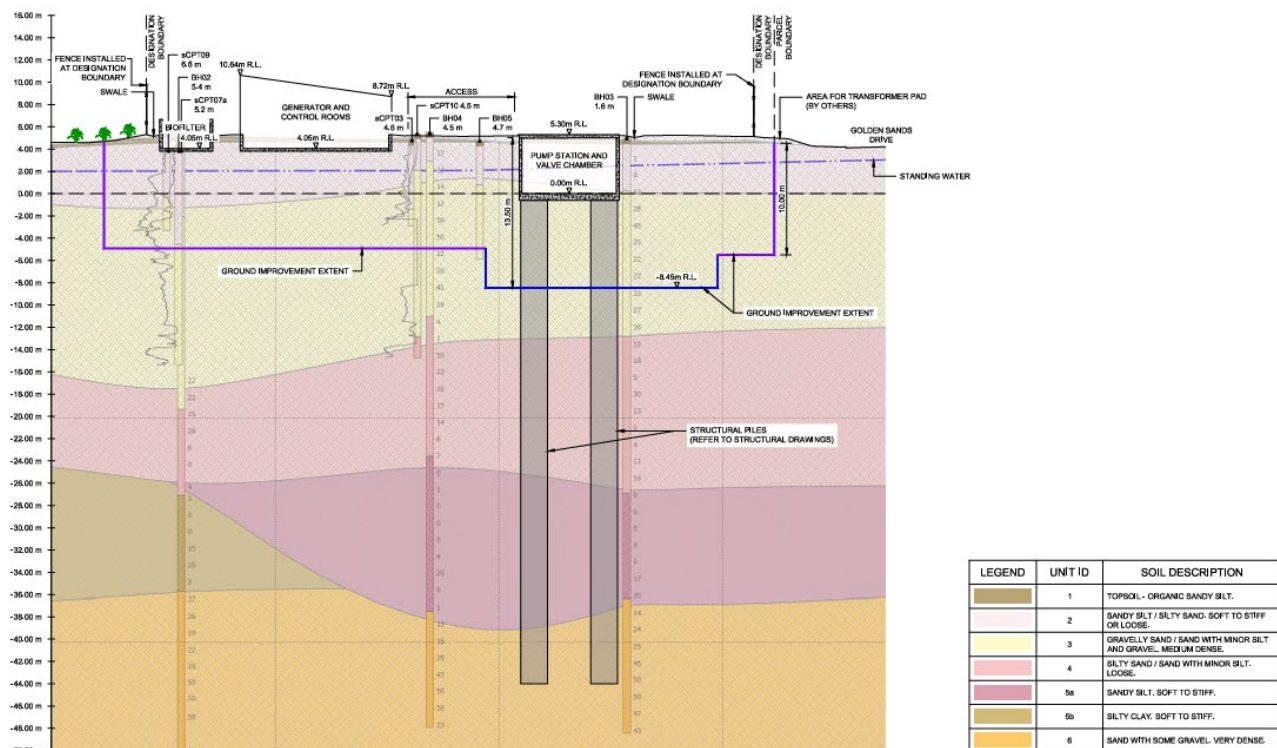


Figure 2: Wairākei pump station representative long section

3 DESIGN REQUIREMENTS

The pump station's ancillary structures are designed to IL2 while the pump station structures (wetwell) are IL3 structures, with a design life of 50 years for seismic design events.

The geotechnical design earthquake parameters, such as the Peak Ground Accelerations (PGAs) and earthquake magnitudes are based on the updated MBIE Earthquake Geotechnical Engineering Practice, Module 1 (NZGS and MBIE, 2021). Although the 2022 New Zealand National Seismic Hazard Model (NSHM) recommends slightly lower PGAs with slightly higher magnitudes for the site, the differences are not significant for this project. Accordingly, the design continued to follow the code-specified MBIE Module 1 parameters.

The design earthquake parameters for the pump station structures are as follows:

- For IL2 structures, a design earthquake with annual exceedance probability (AEP) of 1/25-year and 1/500-year were considered for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) seismic events, respectively.
- For IL3 structures, a design earthquake with AEP of 1/25-year and 1/1000-year were considered for SLS and ULS seismic events, respectively.

3.1 Seismic performance requirement

The geotechnical and structural design for the project targeted the following seismic performance criteria:

1. Under an SLS earthquake, there will be no damage to the wetwell (i.e., IL3 structures), it will remain fully operational.
2. Under an ULS earthquake, the wetwell will experience damage but will be repairable.
3. Associated structures other than the wetwell (i.e., IL2 structures) will have no damage under a SLS earthquake and remain fully operational.

4. Associated structures other than the wetwell (i.e., IL2 structures) may experience unreparable damage under an ULS earthquake.
5. Pipelines will remain fully operational under a SLS earthquake and up to 1/100 AEP earthquake.
6. Pipelines may be damaged beyond repair under an ULS earthquake.

4 CONCEPTUAL DESIGN APPROACH

Considering the geotechnical challenges and seismic design requirements, several foundation design options were considered for the pump stations, each offering different levels of cost efficiency and structural resilience. These options include the following:

- Deep pile foundation.
- Reinforced concrete (RC) reaction slab (i.e. a separate RC slab beneath the wetwell RC foundation slab) with post-earthquake relevelling using screw jacks, over stone column ground improvement.
- RC reaction slab with post-earthquake relevelling using JOG grouting, over stone column ground improvement.
- Cutter soil mixing (CSM) ground improvement with RC reaction slab.

Although deep bored piles offer dependable support and bypass liquefiable soils, applying piles to every structure is impractical and not justified for IL2 components. Conversely, ground improvement alone cannot satisfy the higher performance demands of IL3 structures. The selected approach therefore employs non-vibratory stone columns, complemented by deep bored piles specifically for IL3 structures such as the wetwell. This dual foundation strategy provides a coordinated system that enhances seismic resilience by matching the mitigation method to the performance demands of each structure.

4.1 Ground improvement

Ground improvement was implemented using non-vibratory stone columns to depths ranging from 7 m to 13.5 m below ground level (mbgl). A non-vibratory stone column is a ground-improvement technique that uses a hybrid drilling assembly to penetrate the ground and form the column without generating vibration, instead of using a traditional vibrating probe (McMillan Civil, 2020). This solution creates a non-liquefiable crust beneath the ground surface, ensuring adequate support for the pump station's ancillary structures and their reinforced concrete shallow foundations, while also reducing liquefaction-induced settlements and lateral deformation across the site

Several installation techniques are available for stone columns. However, the non-vibratory installation method was adopted to avoid vibration-induced settlement and associated damage to the existing structures and underground services at the project sites.

4.1.1 Verification trials

Non-vibratory stone column field trials were undertaken on site by McMillan Civil to confirm the effectiveness of the proposed ground-improvement methodology (Figure 3). Based on the trial results, an Area Replacement Ratio (ARR) of 25% was adopted for both the Wairakei and Opal Drive pump stations, including all ancillary structures, with testing demonstrating increased soil stiffness and reduced liquefaction susceptibility within the treated zone. CPT confirmed that a non-liquefiable crust of approximately 6-7m thickness has been successfully formed. The target performance criteria for the stone columns were to reduce the risk of differential settlement and lateral spread within the pump stations. These verification trials validated the achieved improvement depths and confirmed that the target performance criteria were met without adverse effects on nearby structures or services.



Figure 3: McMillan Civil's non-vibratory stone column rig

4.1.2 Slope stability and lateral spreading of improved ground

Slope stability and lateral-spread analyses were undertaken to assess the effects of the improved ground conditions at the Wairākei site. Material parameters for the non-liquefiable crust were derived from pre- and post-improvement CPT data collected during the verification trials. In the Slope/W modelling, treated zone was represented by assigning improved soil parameters. The improved zone was modelled as a single unit with material strength parameters derived using an area-weighted average that's consistent with the adopted 25% ARR. The analyses indicated that the resulting Factor of Safety (FOS) were higher than those calculated without ground improvement. At the wetwell location, the expected lateral displacement under the design seismic event was reduced from approximately 300mm to 100mm, and both total and differential settlement demands were significantly lowered.

4.1.3 Effect of stone columns on piles

The wetwell piles were required to be installed before the stone columns, meaning the piles would be exposed to the lateral stresses generated during stone column installation prior to construction of the wetwell base slab. Non-vibratory stone columns are installed using a displacement-type method in which the drilling assembly penetrates the ground, displaces in-situ soil, and compacts the backfilled stone during extraction without vibration. This installation process will induce elevated horizontal stresses that can increase bending moments and shear forces in adjacent piles and influence the performance of temporary excavation support. This issue is specific to the Wairākei pump station, where the ground-improvement footprint extends beneath the wetwell; at Opal Drive, ground improvement was not required in this area.

To quantify these effects, a soil–structure interaction assessment was undertaken using PLAXIS 3D. The installation of the stone columns was modelled using a cavity-expansion approach to simulate soil displacement and densification, allowing the resulting lateral stress distributions on the piles to be estimated. These outputs were then used to calculate corresponding pile bending moments, shear forces, and deflections. Additional simplified structural modelling of a single pile supported by lateral soil springs and

loaded by the assessed high lateral stresses was performed to validate the results and assess structural demand under the induced lateral loads.

Monitoring of pile head movement will be carried out during stone column installation to verify the analytical predictions and ensure that the piles are not overstressed/damaged during construction.

4.2 Deep pile foundation

Reinforced concrete bored piles with diameters ranging from 1.2 m to 2.4 m were constructed under bentonite slurry to support the wetwells. This installation method was selected to minimise vibration and therefore reduce the risk of construction-induced settlement in the loose saturated sands. The poor ground conditions, characterised by deep liquefiable layers, also posed the potential for significant subsidence induced by liquefaction and associated downdrag (negative skin friction) on the piles. To mitigate these effects, the piles were embedded into the underlying very dense sand strata below the liquefiable horizon, approximately RL -31 m (36 m bgl) at Opal Drive and RL -42 m (47 m bgl) at Wairākei. This ensured adequate end-bearing resistance and minimised differential settlement risks. Figure 4 presents a 3D view of the buried piled wetwell configuration adopted at the Opal Drive pump station.

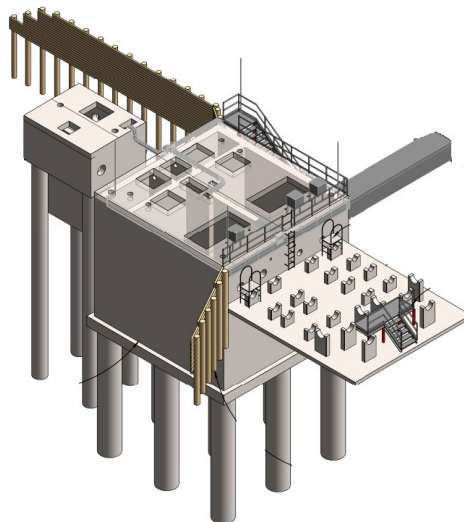


Figure 4: 3D visualisation of the buried piled wet well

4.2.1 Pile Construction

Reinforced concrete piles with sacrificial steel casings were initially considered for the wetwell foundations. However, installing the steel casing would have generated significant vibration during driving, increasing the risk of ground-surface settlement due to densification of the loose saturated sands. This posed an unacceptable risk to nearby existing structures and critical wastewater assets.

To avoid these vibration-induced settlement effects, bored piles constructed under bentonite slurry were adopted instead. The bentonite slurry stabilised the pile excavation, eliminating the need for a sacrificial casing and allowing the piles to be formed with minimal ground disturbance. This approach significantly reduced the likelihood of vibration-induced settlement in the surrounding soils. Figure 5 shows on-site construction of the bored piles at the Opal Drive pump station.



Figure 5: Bored piles being constructed by March Construction

5 CONSTRUCTION VIBRATION

Given the site's susceptibility to vibration-induced settlement, maintaining comprehensive Quality Assurance (QA) records was essential throughout construction. These records provided formal evidence that vibration and dewatering monitoring, as well as the associated mitigation measures, were implemented correctly and consistently.

The QA documentation included a construction vibration assessment completed by the contractor, which recorded vibration levels during stone column installation, piling activities, use of vibratory rollers, and heavy vehicle movements. This monitoring ensured compliance with project-specific vibration limits and helped manage potential risks to surrounding infrastructure.

In addition, settlement monitoring on a weekly basis was undertaken to track ground movements during excavation and ground-improvement works. The results confirmed that settlements remained within acceptable limits and provided early warning of any deviations that could compromise structural performance or require adjustments to construction methods.

6 DEWATERING

Construction of the wetwells required deep excavations extending below the groundwater table, necessitating controlled dewatering to provide a stable working environment. Lowering groundwater levels in the loose saturated sands presented a risk of inducing settlement in adjacent structures and underground services due to groundwater drawdown. To manage this risk, a detailed dewatering-induced settlement assessment was undertaken, including an evaluation of the vulnerability of nearby assets and the development of project-specific acceptable settlement criteria to ensure their protection.

Extracted groundwater was reinjected at locations offset from the excavation to limit the extent of the drawdown cone and reduce associated subsidence. Groundwater levels and ground-surface settlements were monitored daily during dewatering and were located around the excavation perimeter and near sensitive structures, supported by QA procedures to verify that monitoring and mitigation measure were correctly implemented. The dewatering operations were successfully completed, with observed settlements remaining within the established allowable thresholds.

CONCLUSION

The dual foundation system adopted for the Opal Drive and Wairākei pump station demonstrates how complementary mitigation measures can address multiple geotechnical hazards simultaneously. Non-vibratory stone columns reduced liquefaction-induced settlement and lateral spread demands, while deep bored piles ensured structural stability for critical structures and bypassed liquefiable deposits. Construction monitoring confirmed the key design assumptions and illustrated the importance of integrating geotechnical, structural, and construction considerations into a unified design philosophy.

This paper highlights the importance of selecting construction-compatible ground improvement methods, engaging deep foundations when required to meet design performance criteria, and sequencing works to manage soil-structure interaction. The combination of non-vibratory stone columns and deep bored piles proved to be a cost effective, resilient and practical approach to managing liquefaction hazards, with continuous monitoring ensuring that constructed performance aligned with design intent. For the wetwell structure, pile foundations were adopted as Tauranga City Council aimed to achieve a more resilient asset, whereas stone columns remained the more economical option for the surrounding ancillary structures. The principles demonstrated are broadly applicable to infrastructure built on similar environment in New Zealand, where multiple seismic and ground deformation hazards overlap.

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