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Foundations for a clarifier at the Te Maunga Wastewater treatment plant; design and construction

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ABSTRACT

The Te Maunga Wastewater Treatment Plant required the design and construction of a new clarifier as part of the Tauranga City Council plan to provide a resilient treatment process path. This required that some of the site geohazards, such as liquefaction and associated lateral spread, be well understood for the foundation design. Therefore, the ground investigations and assessments aimed to reduce the uncertainty around the depth and extent of liquefiable layers. Once the geotechnical assessments were completed, an optioneering process followed in conjunction with early contractor involvement, to assess several options for foundations and ground improvement options. The outcome resulted in the selection of driven piles for the clarifier foundation. The pile type chosen consisted of an open-ended steel casing with a bottom driven concrete plug, and subsequent installation of reinforcement and concreting inside of the casing. As part of the design process, a site trial was undertaken to understand the construction risks and to gather information about pile capacity. During construction, the design assumptions were confirmed, and acceptance criteria were established by means of dynamic testing and a static load test.

1 INTRODUCTION

The city of Tauranga has been growing and this is expected to continue in coming years. As a result, Tauranga City Council (TCC) is planning to upgrade the Te Maunga Wastewater Treatment Plant (WWTP) to address the increasing demand. Part of the plan is to provide a new process train at the treatment plant with a high level of seismic resilience to allow some operational continuity after a large seismic event.

The clarifier design required knowledge of the geotechnical conditions and associated risks. A shallow groundwater level, which is typical of the site, has the potential to induce buoyancy and became an important challenge during construction for the temporary excavations required. From the seismic perspective, the ground is prone to liquefy which can induce settlements, lateral spread effects and increased buoyancy forces on the structure.

This paper describes the proposed development, the site ground conditions, geotechnical risks, the clarifier resilience criteria, key steps followed during the foundation design and construction stage.

2 PROPOSED DEVELOPMENT

The new clarifier was constructed adjacent to the existing two clarifiers at the plant, refer to Figure 1.

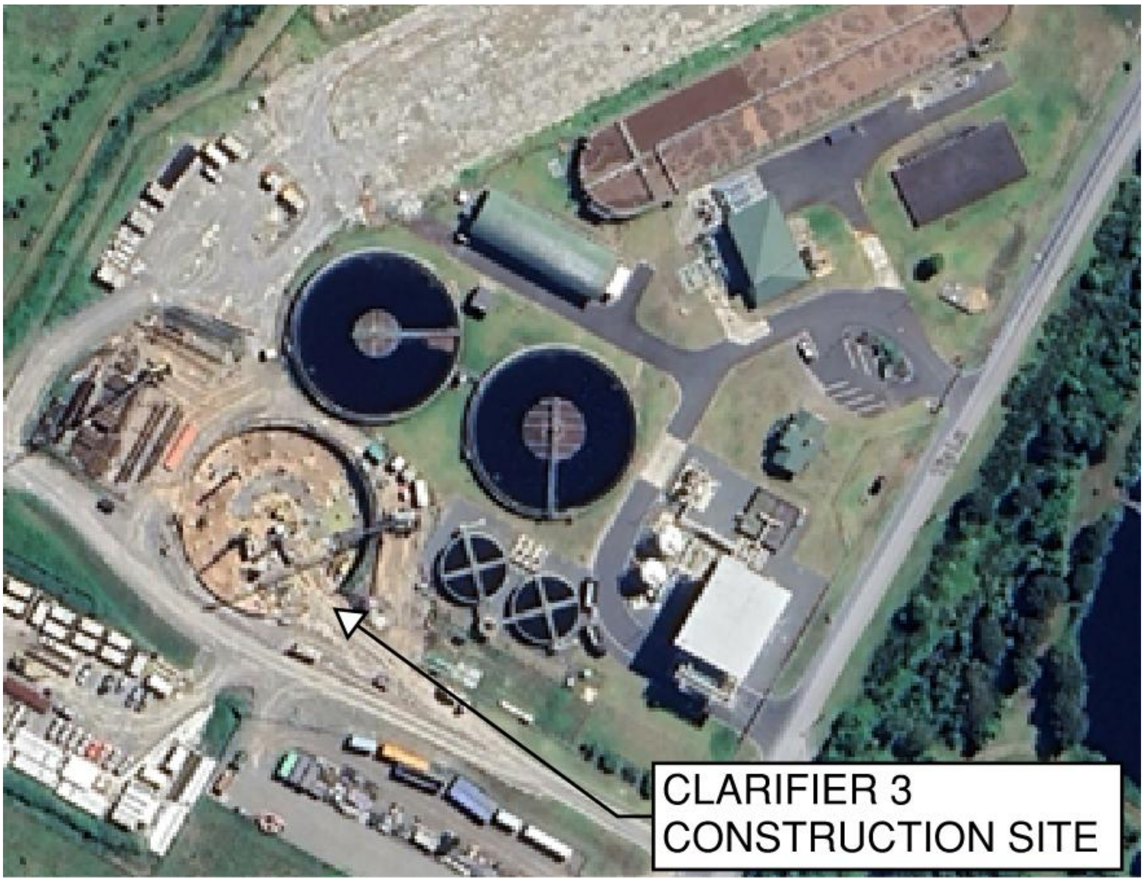

Figure 1. Satellite aerial photo of the Te Maunga Clarifier 3 during construction (Google, 2024).

The new clarifier is a concrete, circular, post tensioned tank of approximately 40m in diameter. It has an internal sloped concrete floor to a central sludge well. A vendor designed, clarifier mechanism is supported at the centre. The structure features a precast concrete internal launder channel and external walkway among other elements. The clarifier is approximately 5m deep with a variable embedment into the ground that varies between 2m and 6m approximately. Figure 2 is a clarifier cross section that illustrates some of these aspects.

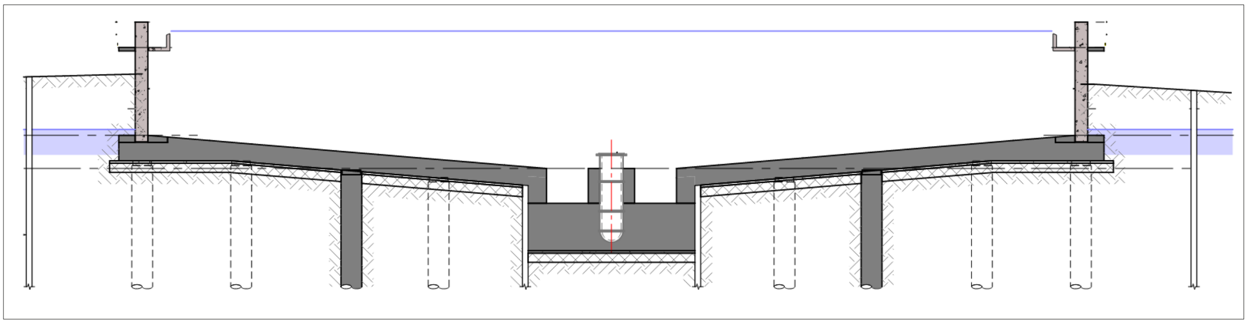

Figure 2. Clarifier cross section showing the internal sloped concrete floor and central sludge well.

3 GROUND CONDITIONS

The site is flat with an approximate elevation of 2mRL (Moturiki 1953 vertical datum). The ground conditions consist of a top layer of approximately 2m of fill, which is underlain by Holocene beach deposits. The latter consists mainly of loose sands with some lenses of gravel. The beach deposits extend as deep as -12mRL and are prone to liquefy. Underlying the Holocene deposits is the Matua Subgroup which consists of fluvial sands and gravels that are often pumicious. Beds or lenses of fine-grained soil are frequently distributed within this unit and are often interpreted as Tephra. The Matua Subgroup is a competent Pleistocene age unit typically comprising medium dense to dense pumiceous sands and gravels, and is considered to be non-liquefiable. However, the tephra units generally exhibit low bearing capacity (CH2M Beca, 2021a). Figure 3 illustrates the ground conditions.

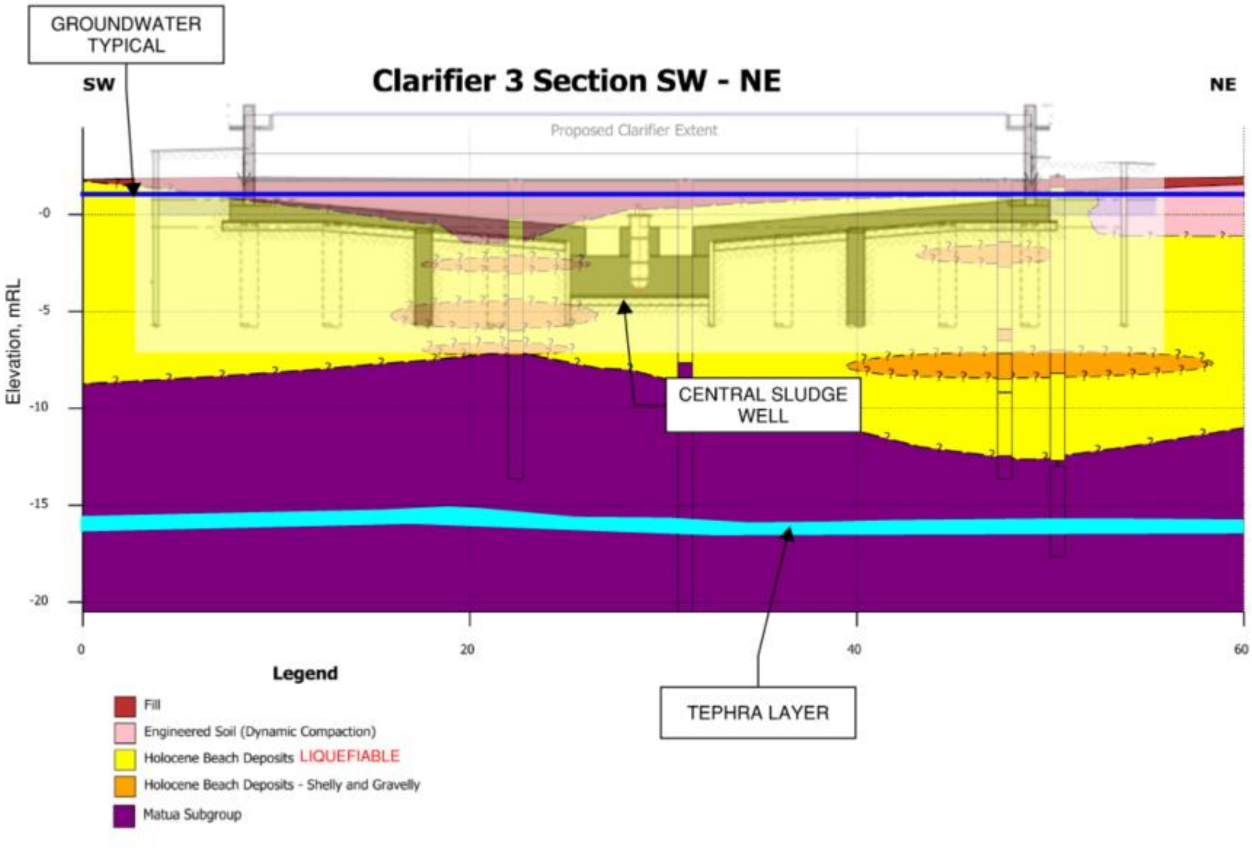

Figure 3. Cross section illustrating the ground conditions at the site (CH2M Beca, 2021b). An elevation image is superimposed to indicate the structure depth relative to the ground.

4 GEOTECHNICAL RISKS

The following key geotechnical risks were considered for foundation design.

- Static buoyancy: This relates to periods of maintenance when the clarifier would be emptied. The depth below ground of the clarifier structure is variable, with the central well at approximately 6m below the existing ground. There is a buoyancy risk when the clarifier is empty due to the considerable volume of the structure below groundwater.
- Liquefaction effects: The Holocene deposits can liquefy during a seismic event. The associated risks are settlements in the order of 200mm, ground lateral spread towards the closest free face (an existing oxidation pond located at approximately 180m from the site) and seismic buoyancy.

5 DESIGN CRITERIA

The clarifier is designated as an Importance Level 3 structure (consistent with AS/NZS1170.0 which specifies wastewater treatment facilities as being of high value to the community, as agreed with TCC). The design life of the clarifier is 100 years.

5.1 Seismic resilience criteria

The seismic resilience was agreed with TCC based on the expected performance of the clarifier for different levels of earthquake shaking intensity (Beca Limited, 2022). Structural performance expectations were defined for each level of seismic intensity and its associated limit state. These are summarised in Table 1.

Table 1: Seismic resilience criteria

Earthquake Shaking Intensity	**Limit State**	**Annual Probability of Exceedance**	**Structural Performance Expectation**
Minor	Serviceability Limit State (SLS)	1/25 year	The Clarifier should remain in operation with no damage during and following an SLS event. Settlements of the Clarifier's wall and floor should be minimal. Settlement limits, for continued unaffected operation, are governed by tolerance requirements of the mechanical components.
Moderate	Serviceability Limit State2 (SLS2)	1/500 year	The Clarifier should retain its contents but may suffer minor damage necessitating repair. These repairs should be feasible to complete within the order of days to weeks. The Clarifier should not require draining to undertake the repair work.
Severe	Ultimate Limit State (ULS)	1/2500 year	The Clarifier may suffer damage which renders it inoperable. It should not collapse. The clarifier should be repairable. Although it may be to a lower level of service than the original construction. Repairs may take in the order of several months.

6 SELECTION OF THE FOUNDATION SYSTEM

Several foundation options were initially considered for the clarifier. The selection was the result of a two-stage Multi-Criteria Assessment (MCA) carried out during an Early Contractor Involvement (ECI) process which involved TCC, Beca Limited (Beca) and HEB Construction Ltd (as the ECI contractor). Based on the results of the MCA, a piled foundation was chosen as the preferred option. The selected piled system are bottom driven reinforced concrete piles with permanent casings. The piles have steel casings which were installed (vibrated) open ended to the required depth. Once installed, the inside of the casing was excavated to a specified depth and a reinforced concrete plug installed at the bottom of the casing and driven to obtain the required end bearing resistance. A mandrel was placed beneath the hammer to transmit the driving forces to the plug. Figure 4 shows the concept of the bottom plug inside an open casing with the reinforcing cage and concrete infill installed inside the casing after driving.

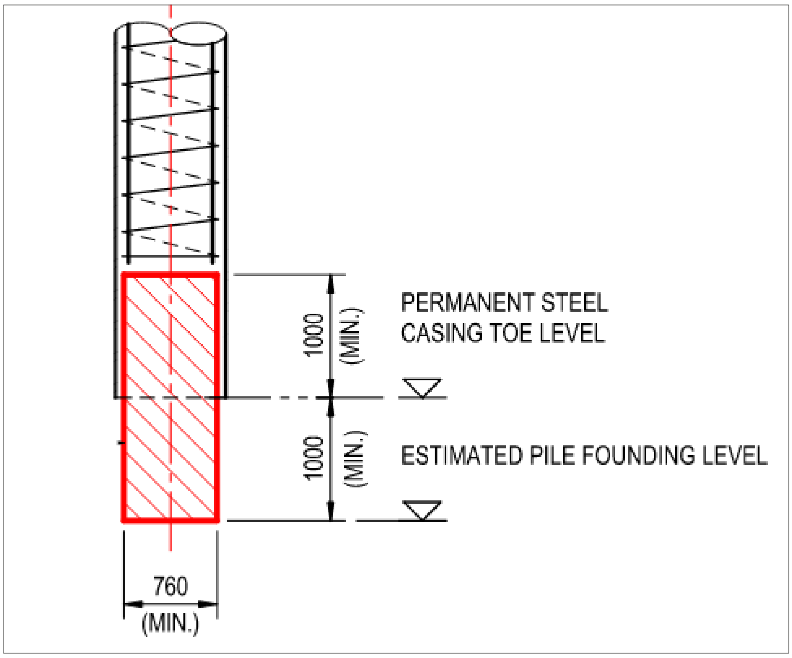

Figure 4. Illustration of a pile with a bottom plug inside an open casing.

7 SITE TRIAL

A site trial of the selected pile system was undertaken, which consisted of the installation of a pile casing and driving plug, inside the clarifier footprint. The objective was to understand the construction risks and undertake testing to inform the design.

The key aspects learned from the site trial were the following:

- The capability of the installation equipment (vibrators and hammers) was tested, and the required adjustments were made.
- The removal of spoil inside the casing requires leaving a soil plug at the bottom end of the casing to avoid base heave.
- The PDA test results provided a magnitude of skin friction of approximately 500kN below elevation -12mRL. This information was used for design as this would be the available skin friction when widespread liquefaction occurs.

- The pile casing capacity estimations from the Hiley formula were generally higher (around 3 times) than from the PDA measurements.
- It was concluded that a static load test would be required, at the start of production piling, to prove the end bearing capacity of the driving plug. A correlation between the static load test results and set card measurements would be the basis to develop acceptance criteria for the plug.
- Vibration monitoring was undertaken during pile driving, and it was found that, generally the vibration levels were within the compliance limits recommended for adjacent reinforced concrete structures.

8 DETAILED DESIGN

Following the successful site trial, it was decided to implement a foundation system consisting of 76 piles arranged in a radial layout. The pile casings would be 910mm in diameter, the driving plug had to be a minimum of 760mm in diameter and 2000mm in length, and the pile tips extended to elevation -20mRL. The pile design considered the cases of static and seismic downward and upward buoyancy loads, cyclic ground loads, inertial loading and lateral ground spread.

8.1 Construction testing

Part of the design process involved nominating verification testing during the pile installation. Dynamic testing (PDA tests) was proposed for 15% of the production pile casings and a static load test was proposed for one of the production piles plugs. The PDA tests were aimed at confirming the skin friction resistance on the casings and the static load test to confirm the bearing capacity of the driving plugs. Pile driving monitor (PDM) data was obtained for every pile to subsequently estimate pile capacities using the Hiley formula. The static load test and some of the PDA testing was proposed for the first piles so that acceptance criteria could be developed early for the production piles.

8.2 Strength reduction factor

The selection of the strength reduction factor was made following a procedure recommended in the standard AS2159 Piling – Design and Installation (Standards Australia Limited, 2009), which assess risks associated with the geotechnical knowledge of the site, design aspects and construction. The geotechnical knowledge risk was estimated to be moderate based on the possible variability of the ground conditions at the site. The design risks were estimated to be low based on the designer’s experience, the method of evaluating geotechnical parameters, the design methodology employed and the use of in-situ data. The construction risks were also estimated to be low based on the expected monitoring and proposed construction verification testing. The overall reduction assessment resulted in a low risk. Based on this outcome a strength reduction factor of 0.65 was selected. This was applied to the design geotechnical ultimate strength.

8.3 Required geotechnical ultimate strength

The pile vertical capacity is obtained from the combined contribution of the casing skin friction and the end bearing capacity of the driving plug. From the site trial, it was estimated that an ultimate skin friction strength of 500kN could be obtained below elevation -12mRL (inside the non-liquefiable unit). From initial estimations of static and seismic (liquefied) buoyancy, it was considered that this value of ultimate skin friction strength was appropriate.

The other component of the pile vertical capacity is the ultimate end bearing strength of the driving plug. Considering the skin friction value of 500kN, the chosen strength reduction factors and the vertical demands, the required ultimate end bearing strength of the driving plug had to be a minimum of 3200kN. Therefore, the required geotechnical ultimate strength was a minimum of 3700kN.

8.4 Structural Aspects

From the structural perspective there were some design challenges. The piled raft required strict crack control measures to limit leakage. TCC required the structure to have a high level of resilience even after a high level of seismic shaking. To limit damage the foundation was designed as nominally ductile, but this increased the loading. To counteract this without significantly increasing lateral movement (such as a pinned connection) a semi-rigid connection was implemented between the top of the piles and the pile raft which forms the tank base. This semi rigid pile to raft connection limits the forces the pile can induce into the slab during seismic loading as a hinge develops and therefore limit cracking and leakage of the pile raft. To achieve this the pile reinforcing across the hinge zone was detailed to provide confinement and shear capacity under yielding of the longitudinal bars from the pile into the raft. The pile casing was isolated from the underside of the raft to prevent the steel tube casing from contributing to the strength of the connection.

9 CONSTRUCTION

The development of an acceptance criteria was carried out during the early stages of pile installation. For the casings, it consisted of correlating Hiley formula capacity estimations with the results of PDA testing. Similarly, for the driving plug, correlations were made between the results of the static load test and the measurements (set and rebound) taken on the installation mandrel.

9.1 Static Load Test

A static load test was undertaken on one driving plug of a designated pile. As per the design, the plug was driven and advanced to a minimum of 1000mm beneath the casing tip. PDM data was obtained for a capacity (Hiley formula) estimation and the static load test was undertaken. The test arrangement consisted of a mandrel which was positioned inside the pile casing resting on the driving plug. Hydraulic cylinders loaded the top of the mandrel and deformation gauges recorded the settlement. The hydraulic cylinders reacted against a central frame, which was also supported by other frames at each end of it, for a total of six reaction piles. Figure 5 shows a photo of the test.

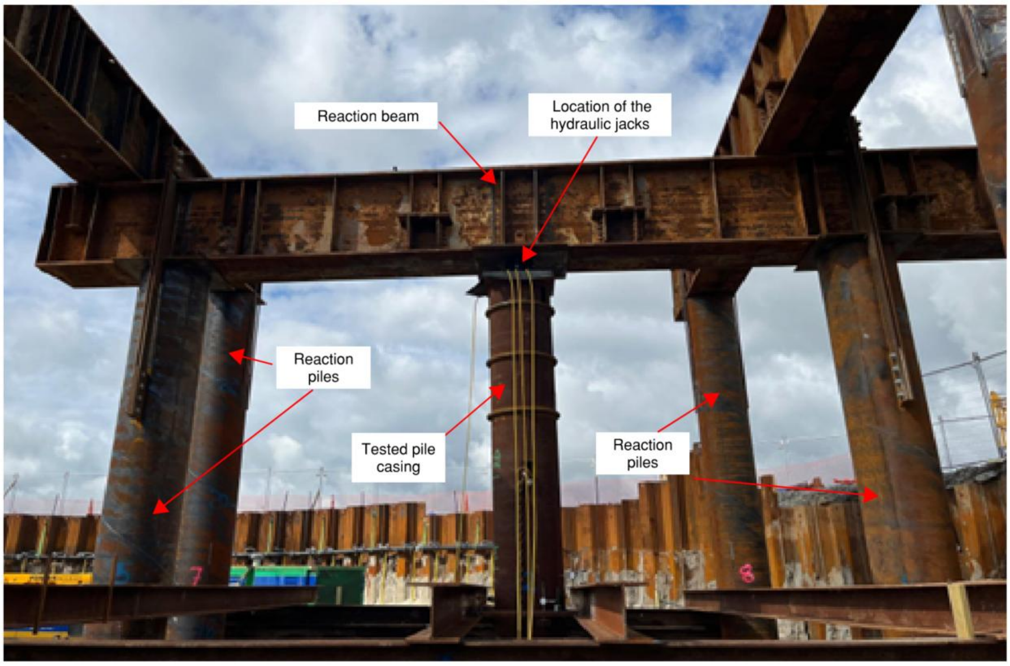

Figure 5. Image of the static load test arrangement

The loading sequence of the test was undertaken according to AS2159 Piling – Design and Installation (Standards Australia Limited, 2009), which requires two load cycles. For the first cycle, the load was increased up to the pile service load, which for this case had a magnitude of 1750kN. For the second load cycle, the load was increased up to a 110% of the design ultimate end bearing strength (3200kN), equating to a maximum applied load of 3520kN. Dial gauges on top of the mandrel were used to measure the approximate settlement of the plug.

The acceptance criteria for the service load cycle was that the settlement had to be less than 13mm under the maximum service load and less than 8mm after unloading. The maximum settlement, recorded under the service load, was 15mm. Although the value was larger than the maximum, it was considered acceptable because it is likely that part of the measured value corresponds to an elastic compression of the mandrel that loads the bottom plug. After unloading the service load cycle, the remaining settlement was in the order of 5mm and thus satisfactory.

For the ultimate load cycle, the acceptance criteria was that settlement had to be less of 59mm during the ultimate load application and permanent settlement of less than 51mm after unloading. For the test, the maximum settlement was in the order of 25mm and in the order of 15mm after unloading. Therefore, the performance was considered acceptable.

9.2 Test data

The development of the acceptance criteria involved reviewing data for both the pile casing and the bottom plug.

9.2.1 Pile casing

PDA test data was obtained at one casing after removing the soil from inside the casing. A skin friction of 253kN was measured below elevation -12mRL. This value was approximately 6.4% of the measured Hiley capacity (with soil inside) estimated for this casing. The measured skin friction was less than the proposed design requirement of 500kN below -12mRL.

All other PDA tests undertaken on casings were with soil still inside the casings. The information gathered was used to confirm some design assumptions but not used for the development of the acceptance criteria.

9.2.2 Bottom plug

Data for one bottom plug, as follows:

- PDM data obtained during the installation of the bottom plug. According to the PDM data, the measured set was 6.5mm per blow with a rebound of approximately 25mm. The rebound is measured at the mandrel used for the installation and, as it detached from the plug, it is unlikely that the measured rebound corresponds to the true rebound at the plug (which is likely to be less than the measured value). From the measured set and rebound, the corresponding Hiley capacity was estimated to be in the order of 3600kN.
- Static load test results, which were previously described.

9.3 Revised distribution of the ultimate geotechnical strength

For each pile, the design required a design ultimate geotechnical strength of 3700kN, derived from a minimum design ultimate skin friction strength of 500kN (below -12mRL) for the casing and a minimum design ultimate end bearing strength of 3200kN for the plug. However, from the reviewed test data, a skin friction deficit of approximately 300kN was found. As this deficit could impact the capacity of the foundations to resist static and seismic buoyancy, a review of the uplift demands was made.

The estimation of the buoyancy uplift force was based on the application of the Archimedes principle. For static conditions, the uplift force was calculated by multiplying the clarifier volume below groundwater by the unit weight of water. For the seismic (liquefied) case, the same procedure was used but the volume was multiplied by the total unit weight of the soil (i.e. the fluid). The uplift force was then divided by the number of piles to determine the uplift demand per pile.

After reviewing the uplift demands it was concluded that the measured skin friction strength was sufficient. Consequently, the following was proposed for the design ultimate geotechnical distribution:

- For the casing, a minimum value of design ultimate skin friction strength (beneath -12m RL) of 200kN.
- For the driving plug, a minimum value of design ultimate end bearing strength of 3500kN.

9.4 Acceptance criteria

As a results of the revised strength distribution, the following acceptance criteria were proposed:

- Pile casings: For the pile casing, it was proposed to consider the derived ratio of a resulting skin friction for an empty casing (beneath -12m RL) of approximately 6.4% of the estimated Hiley capacity for that casing (refer 3.1). Therefore, to obtain a skin friction capacity (beneath -12m RL) of 200kN, a minimum Hiley capacity of 3200kN was required for a full casing (i.e., prior to excavation).
- For the bottom plug, it was proposed to use the achieved set (for the tested plug), of 7mm per blow as reference for a plug that achieves the required 3500kN. Although the measured rebound was not representative of the rebound of the plug, it was still required to be recorded. This is because a higher measured value could be an indication of a higher rebound of the plug and thus a reduced plug capacity.

9.5 Structural aspects

The following were notable structural aspects of the foundation construction:

- A polystyrene void former was placed inside the top of the pile casing to create necking and isolation between the concrete and pile casing with a smaller reinforcement cage inside the main reinforcement.
- The raft slab reinforcement was placed in a radial and circumferential layout to achieve the inverted conical shape.
- A trial concrete mix was poured for the 800mm thick raft to confirm heat of hydration.
- The raft was constructed in two pours, the external perimeter ring was poured first to allow precast wall placement and then the centre poured with concrete volume of approximately 900m^3.

9.6 Foundation completion

The pile installation started around mid-October 2023, with the pre-production testing undertaken during November 2023. The acceptance criteria were agreed in late November and the foundation works were completed in late April 2024. No issues were reported as all of the 76 piles complied with the acceptance criteria. Figure 6 is an image of the clarifier with piling completed.

Figure 6. The new clarifier (at the bottom of the image) with piling completed

10 CONCLUSIONS

Several learnings were gathered during the development of the clarifier 3 foundation. These can be summarised in the following conclusions:

- The early contractor involvement was beneficial when selecting the foundation system.
- The site trial provided useful information for the design and helped for an early identification of risks (construction and design)
- One of the risks identified during the site trial was around the uncertainty on end bearing strength of the driving plug. Undertaking a static load test early during construction mitigated this risk.
- The Hiley method did not provide reliable capacity estimations. However, the correlation of the Hiley results with the PDA and static load results allowed for the development of a suitable acceptance criteria.
- A semi-rigid pile connection can be utilised to optimise a raft foundation design.

11 ACKNOWLEDGEMENTS

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