Challenges associated with the geotechnical design of Te Kaha – Canterbury's new Multi-Use Arena

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ABSTRACT

Te Kaha is a new permanently covered multi-use arena in Christchurch and is the final anchor project in the Christchurch Central Recovery Plan, following the Canterbury earthquake sequence of 2010-2011. The $683-million project will have a seating capacity of 30,000 for sports events and will hold up to 36,000 spectators for large music events. The Arena is located in the central city on a site with recent earthquake history and evidence of liquefaction. Extensive seismic hazard studies, geotechnical and hydrogeological investigations and geotechnical design have been completed to allow facilitate construction of the foundation system. The liquefiable site soils have been improved by installation of rammed aggregate piers and a combination of stiff rafts and intersecting grillage beams over the improved ground form the foundation system of the Arena. The adopted foundation system, soil-structure interaction analysis for static and seismic conditions, as well as some design and construction challenges and complexities are described.

1 PROJECT CONTEXT

The Canterbury Earthquake Sequence of 2010-2011 (CES) caused significant damage to much of Christchurch City, including the stadium located at Lancaster Park. Te Kaha, a new permanently covered multi-use arena, is currently under construction. Artist impressions of the completed arena are presented in Figure 1. Te Kaha is located in Central Christchurch on three city blocks bounded by Hereford Street, Tuam Street, Barbadoes Street, and Madras Street (Figure 2).

Construction of the arena began in 2022 and is programmed to be completed by mid-2026. All ground improvements have been constructed and most of the arena foundation is complete.

Figure 1: Artist impressions of Te Kaha (images sourced from www.Te-Kaha.co.nz).

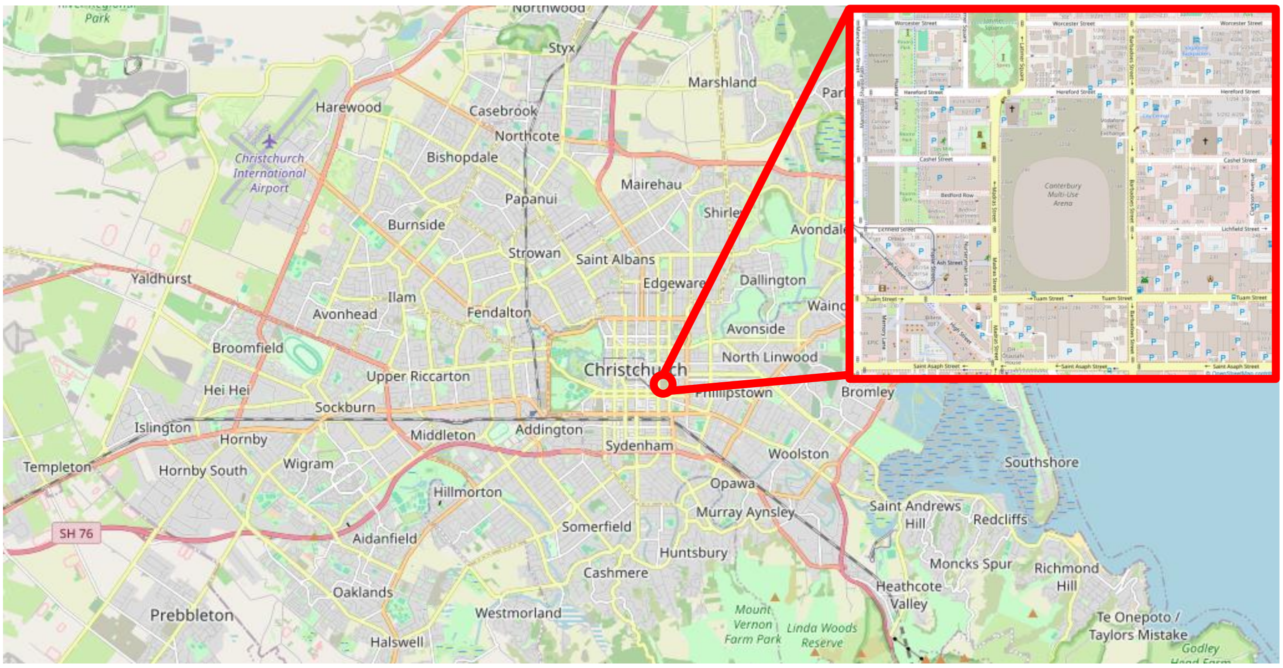

Figure 2: Location of Te Kaha in Christchurch City (annotated image sourced from openstreetmap.org).

2 SITE CONDITIONS

2.1 Site history and geology

The site encompasses three central city blocks which were formerly occupied by several single and multi-storey buildings, two road carriageways, and flat car parking areas. Most of the structures were damaged in the February 2011 earthquake and subsequently demolished to leave a generally flat site with a horizontally and vertically variable layer of demolition debris and uncontrolled fill. The fill material is underlain by Holocene river deposits; described as a ‘modern river floodplain/low-level degradation terrace’ comprising unweathered, variably sorted gravels, sands, silts, and clays (Forsyth et al., 2008).

Geotechnical investigations at the site encountered interbedded sands, silts, and gravels, as shown in Figure 3. Soil units and layering were generally consistent across the site, with the exception of an intermediate dense gravel layer from approximately 5.5 to 8.5 m below ground level (bgl), which was present in the northwest part of the site but absent in the southeast (refer Figure 2). Geotechnical boreholes were extended to over 60 m bgl and encountered both silt and gravel layers approximately 15 m beneath the top of the Riccarton Gravel Unit, which is often considered a proxy for bedrock in central Christchurch. These deeper units are not shown in Figure 3.

2.2 Seismic Hazard

Te Kaha was designed as an importance Level 3 (IL3) structure with Ultimate Limit State (ULS) design return period of 1000 years, Serviceability Limit State 1 (SLS1) deign return period of 25 years, and SLS2 design return period of 250 years.

A site-specific seismic hazard assessment, comprising a site-specific probabilistic seismic hazard assessment (PSHA) and seismic ground response analysis (SGRA), was completed to estimate design ground motions for Te Kaha. Design ground motions adopted for liquefaction analysis are presented in Table 1. Further discussion of the project benefits of the PSHA follows in Section 3.1.2.

Table 1: Design earthquake ground motions for liquefaction analyses.

Parameter	**SLS1 (25 yr)**	**SLS2 (250 yr))**	**ULS (1000 yr)**
Moment Magnitude (M_w)	6.5	6.8	7.0
Peak Ground Acceleration (PGA)	0.14g	0.34g	0.49g

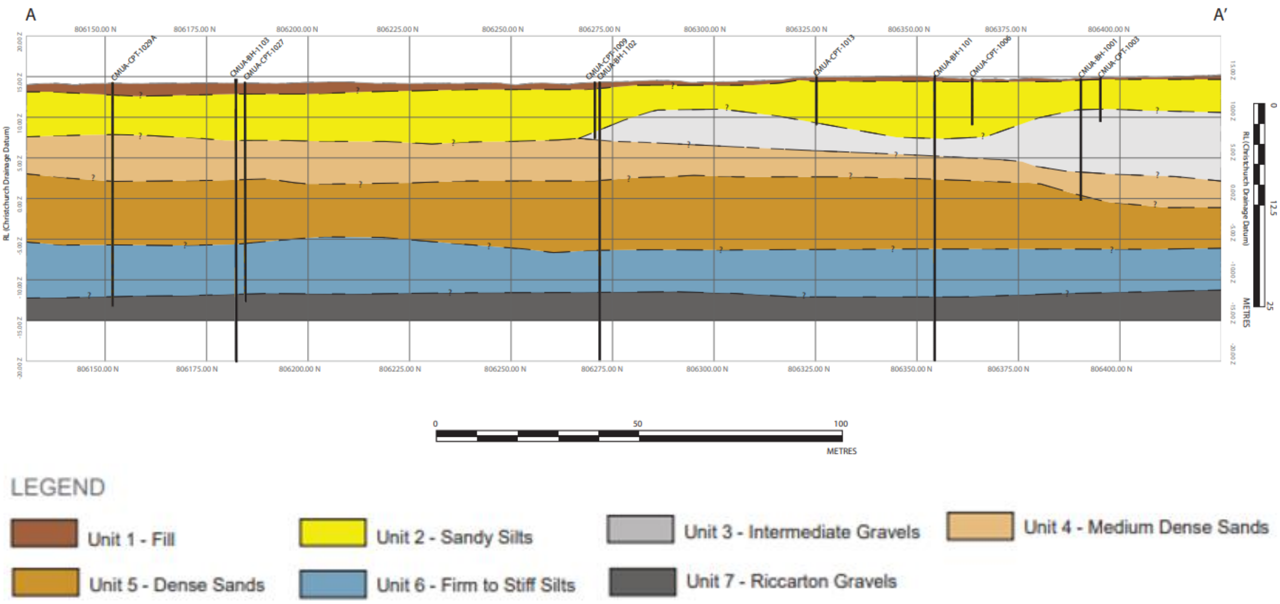

Figure 3: Geological long section through the Te Kaha site (north on right hand side of section).

2.3 Liquefaction Risk

2.3.1 Past earthquake site performance

Observations of ground damage and liquefaction following each significant CES event are documented in the New Zealand Geotechnical Database (NZGD). Key insights into the past earthquake performance are:

- The most severe shaking intensity at the site is estimated to be PGA 0.45 g during the Mw 6.2 earthquake on 22 February 2011. This level of shaking is nearly equivalent to the 1000-year PGA and Mw for the site.
- No evidence of liquefaction was observed over most of the site after the 22 February 2011 earthquake; however, evidence of liquefaction was observed on the eastern edge of the site and in the blocks to the east of the site across Barbadoes Street.
- The total vertical ground movement of the site was measured to be generally neutral (i.e., no significant residual settlement or heave) across all the main CES events. This is consistent with observations of none to minor liquefaction.
- No ground cracking was mapped within 300 m of the site following any of the CES events.

Back analysis of liquefaction potential considering 22 February 2011 ground motions indicates total reconsolidation settlement in the order of 100 to 200 mm for both the Te Kaha site and surrounding blocks. This exceeds the vertical ground movement measured by LIDAR survey and indicates more severe liquefaction than what was observed on the site, i.e., the site appears to have performed better than expected based on simplified liquefaction analyses, however the reasons for this are not immediately apparent.

2.3.2 Liquefaction analysis

Liquefaction analysis conducted in accordance with standard industry practice (Idriss and Boulanger, 2014; Zhang et al., 2002) indicated liquefaction in multiple layers of the soil column, particularly during a design ULS event. Liquefaction analysis of a typical pre-ground improvement CPT is presented in Figure 4.

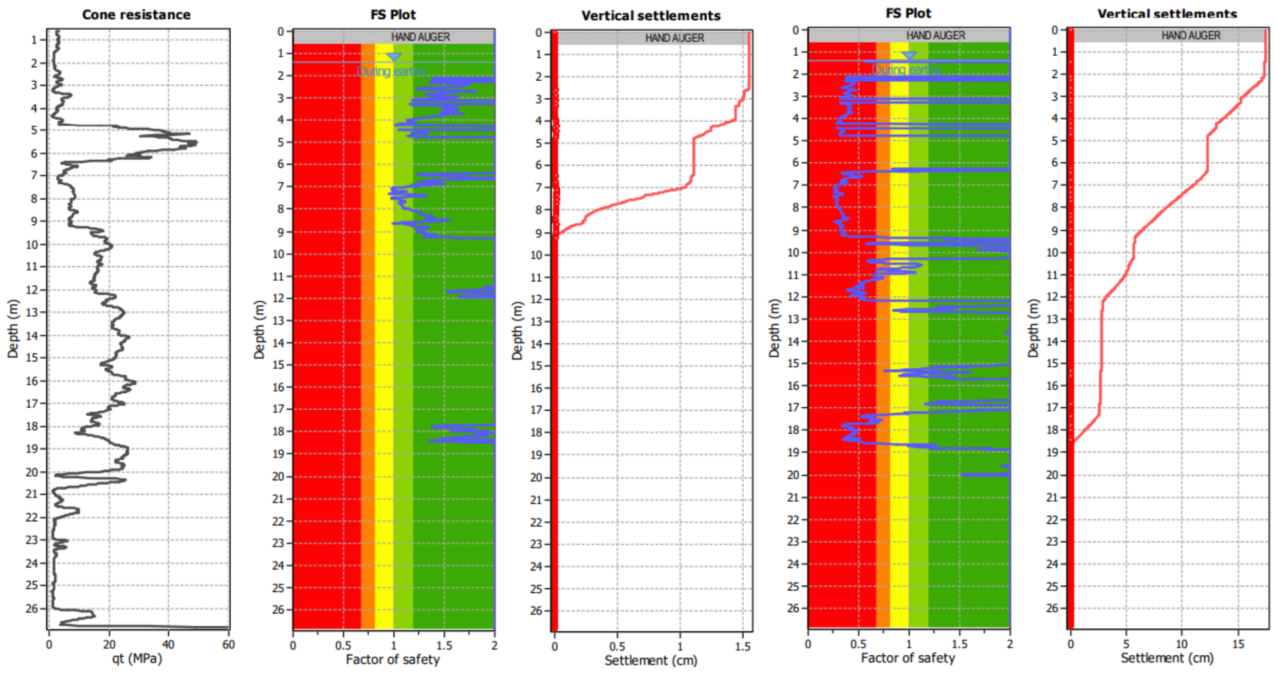

Figure 4: Results of liquefaction analysis of typical CPT (from left to right: CPT cone resistance; Factor of Safety against liquefaction in SLS event; estimated free-field settlement in SLS event; Factor of Safety against liquefaction in ULS event; estimated free-field settlement in ULS event).

3 DESIGN CONSIDERATIONS AND OPTIONS

3.1 Design Challenges

3.1.1 Soil Conditions

A brief description of soil units and challenges encountered in their geotechnical performance is as follows:

- Unit 1: Uncontrolled gravel fill, typically 1 m thick; challenges associated with variable thickness and material properties, as well as significant demolition debris and other obstructions.
- Unit 2: Silty sand and sandy silt, typically 4.5 m thick; poor bearing capacity and highly susceptible to liquefaction in its virgin state.
- Unit 3: Dense sandy gravel, typically 3 m thick; difficult to penetrate with some foundation or ground improvement types, layer not consistently present across the site.
- Unit 4: Medium dense sand, typically 4 m thick; highly susceptible to liquefaction.
- Unit 5: Dense sand, typically 10 m thick; susceptible to liquefaction.
- Unit 6: Firm silt, typically 5 m thick; potential for consolidation settlement.

3.1.2 Updated Understanding of Seismic Hazard

Planning and design phases of Te Kaha coincided with planned updates to the New Zealand National Seismic Hazard Model (NSHM) that incorporated improved scientific understanding of the sources of earthquakes in New Zealand (e.g., seismic source models) and how earthquake ground motions are transmitted through the Earth's crust (e.g., ground motion models). The risk of changed understanding of seismic hazard during the project, due to the then pending updated NSHM was identified as a project risk.

New Zealand Standard 1170.5 allows for "special studies" to depart from the prescribed method of developing earthquake design actions. A site-specific PSHA and SGRA were completed for Te Kaha as a special study in lieu of using the codified method. This allowed consideration of pertinent changes in the scientific understanding of seismic hazard that have occurred since the last substantial update of New Zealand Standard 1170.5. Figure 5 presents a comparison of the 1000-year return period uniform hazard spectra from NZS 1170.5, the 2022 revised NSHM, and the Te Kaha site-specific PSHA. This demonstrates relative agreement between the PSHA and revised NSHM, providing an element of validation that the site-specific seismic hazard assessment, and consequently the Te Kaha project, has captured the best-available understanding of seismic hazard at the site at this time.

3.2 Options Considered

Three main geotechnical design options were considered during concept design – shallow foundations with ground improvement, deep piled foundations, or a combination of both. Early in the concept design phase it was determined that shallow foundations alone would not provide adequate bearing capacity or manage the liquefaction hazard.

Performance and sustainability aspects of foundation systems considered are summarised in Table 3.

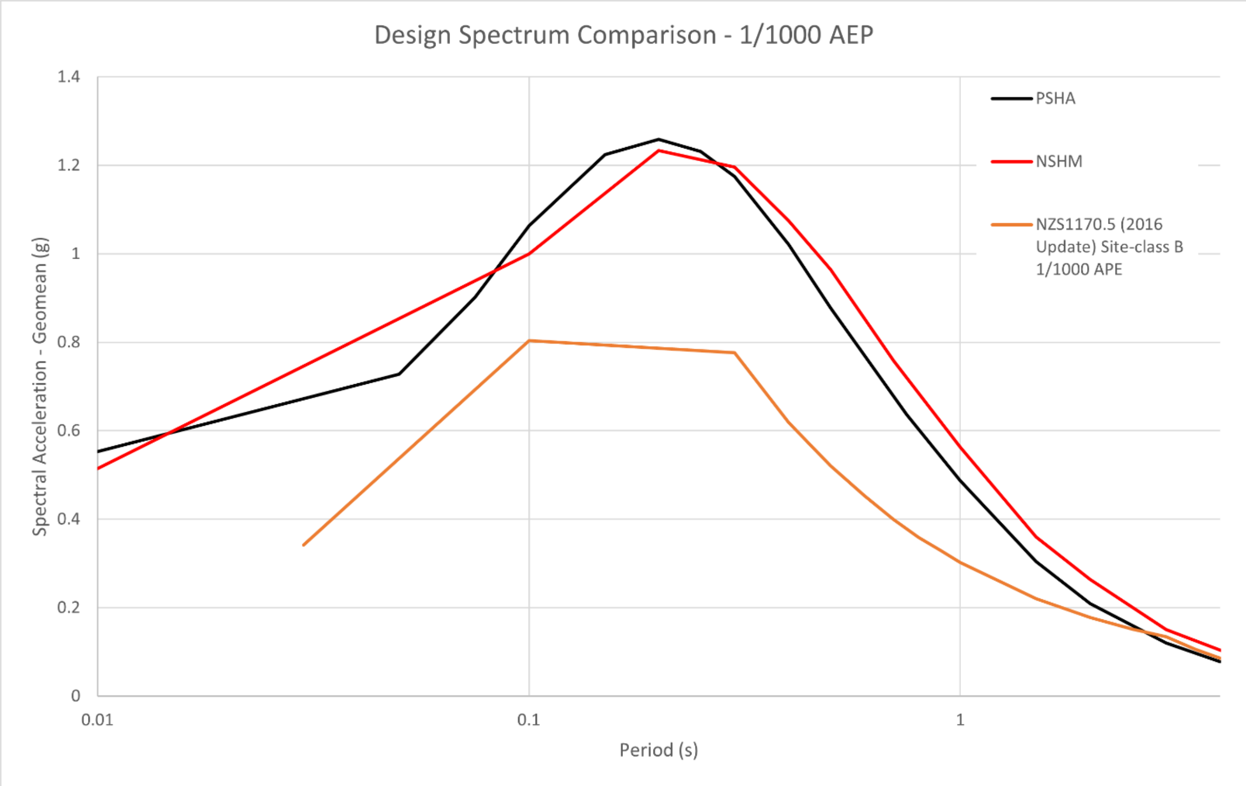

Figure 5: Comparison of 1/1,000 Annual Exceedance Probability (AEP) uniform hazard spectra from NZS 1170.5, CMUA site-specific PSHA, and the 2022 revised NSHM (sourced from nshm.gns.cri.nz).

Table 3: Summary of advantages and disadvantages of selected ground improvement techniques.

Ground Improvement Technique	**Advantages**	**Disadvantages**
Shallow foundations with ground improvement	- Liquefaction hazard can be significantly reduced to provide a non-liquefiable crust. - Many techniques and specialist contractors allow flexibility in solution. - Some techniques incorporate tension elements without the need for a separate solution to accommodate uplift loads. - Lowest carbon footprint and construction cost due to reduction in steel and concrete volumes.	- Performance meets design criteria but lower level of resilience than other options, particularly for events exceeding ULS.
Deep pile foundations	- Large pile capacities (compression and tension) can be achieved by extending piles to Riccarton Gravel layer. - Well understood performance and ability to accurately load test piles	- Riccarton Gravel layer may be insufficiently thick for suitable founding, depending on pile type. - Liquefaction hazard not mitigated so may have experience negative skin friction loads. - High carbon footprint and construction cost due to material type and quantity.

Deep foundations with ground improvement	- Best performance because of liquefaction mitigation and extension to non-liquefiable layer. - Opportunity to use shallower founding layer if ground can be sufficiently improved. - Well understood performance and ability to accurately load test piles.	- Highest carbon footprint and construction cost due requirement for two solutions.

3.3 Adopted Foundation Option

The adopted foundation option is a system of stiff raft foundations and intersecting grillage beams on an 8 m thick improved ground crust. A performance specification for ground improvements was prepared so that all techniques could be fully considered. A summary of key specification requirements is as follows:

- Liquefaction hazard mitigation to be proven by post-ground improvement CPT testing and qualitative discussion of secondary improvement benefits (increased drainage, etc.).
- Foundation support performance (settlement and bearing capacity) to be improved.
- Uplift capacity to be provided in areas specified by the structural engineer.

The selected ground improvement solution comprised Geopier Rammed Aggregate Piers® (RAPs). Key drivers for the selection of RAPs included:

- RAPs are a suitable ground improvement method for the soils present on site.
- RAPs are a displacement technique which provides ground densification as well as reinforcement, particularly in sandy materials.
- RAP aggregate columns provide a drainage path to dissipate porewater pressure buildup during cyclic loading.
- RAPs can be installed with a steel harness that can be connected to the foundation to provide uplift resistance.
- Good track record of local projects adopting similar ground improvement techniques.
- Relatively low carbon footprint due to the natural and locally sourced aggregate and low quantities of spoil.

4 GROUND IMPROVEMENT

A system of RAPs was designed and constructed in accordance with the performance specification. RAPs generally extend to 8 m bgl in the south and to the top of the intermediate gravel layer in the north of the site, providing a non-liquefiable crust of approximately 8.5 m below finished floor level. Uplift resistance was provided by “tension” RAPs comprising a steel plate at the base of the RAP pier and two steel bars extending into the foundation beams. Key findings of the site performance post-ground improvement include:

- Liquefaction analysis of post-ground improvement CPT testing generally indicates densification and a reduction in liquefaction potential in sandy soils (I_C<1.8).
- Transitional soils (I_C of 1.8 to2.05) and some sandy layers (I_C<1.8) did not show adequate densification to mitigate the liquefaction hazard estimated by CPT analysis. 72 fines content tests were completed to investigate the disparity between indicated I_C values and lack of densification following construction of the ground improvement. These tests demonstrated that the soils encountered have a high silt content and are less susceptible to liquefaction than indicated by CPT testing (Lees et al., 2015).

- Several tension RAPs were tested by applying cyclic tension loads to determine the as-built load capacity and stiffness. The number and location of tension RAPs was determined with the structural engineer. In some locations, the length of tension RAPs needed to be increased to meet the tensile demands of the structure.
- Plate load testing of RAPs indicated that the design target modulus had been achieved.

5 FOUNDATION STRUCTURE

A system of stiff raft foundations beneath the western and northern stands, and intersecting grillage beams beneath the southern and eastern stands has been adopted for Te Kaha (Figure 6).

An iterative approach was adopted for soil structure interaction (SSI) analysis completed for the structural and geotechnical design of the foundations. The geotechnical analyses completed included:

- Liquefaction assessments of soils pre and post ground improvement.
- 3D settlement assessment of the arena structure to determine equivalent Winkler springs for use in structural analysis.
- Bearing capacity assessment of individual ground beams and raft foundations considering static and post-liquefaction soil conditions.
- Refinement of soil-structure interaction effects adopting 3D finite element modelling.
- Geotechnical review of the arena structural finite element model to ensure that the geotechnical parameters are used correctly.

6 CONSTRUCTION CHALLENGES

Substructure construction for Ta Kaha is mostly complete at the time of writing. Figure 7 shows various stages of the construction. Subsequent sections detail challenges encountered on the site during construction of the foundation and substructure.

6.1 Obstructions and historical demolition debris

Historical demolition debris was encountered over much of the site and significant obstructions were uncovered in the uncontrolled fill layer. These presented challenges during construction of the RAPs and foundation system. Figure 8 shows two examples of significant obstructions encountered on the site.

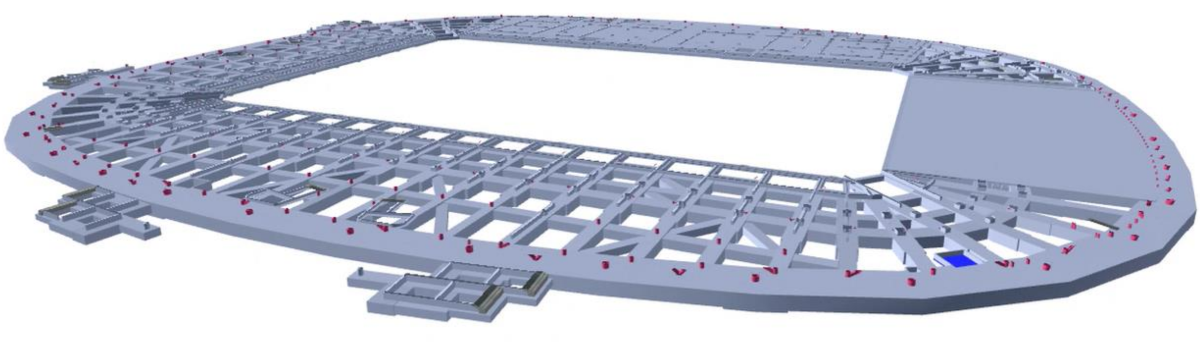

Figure 6. 3D image of the arena foundation system (North is to the right of frame, image sourced from Mott MacDonald).

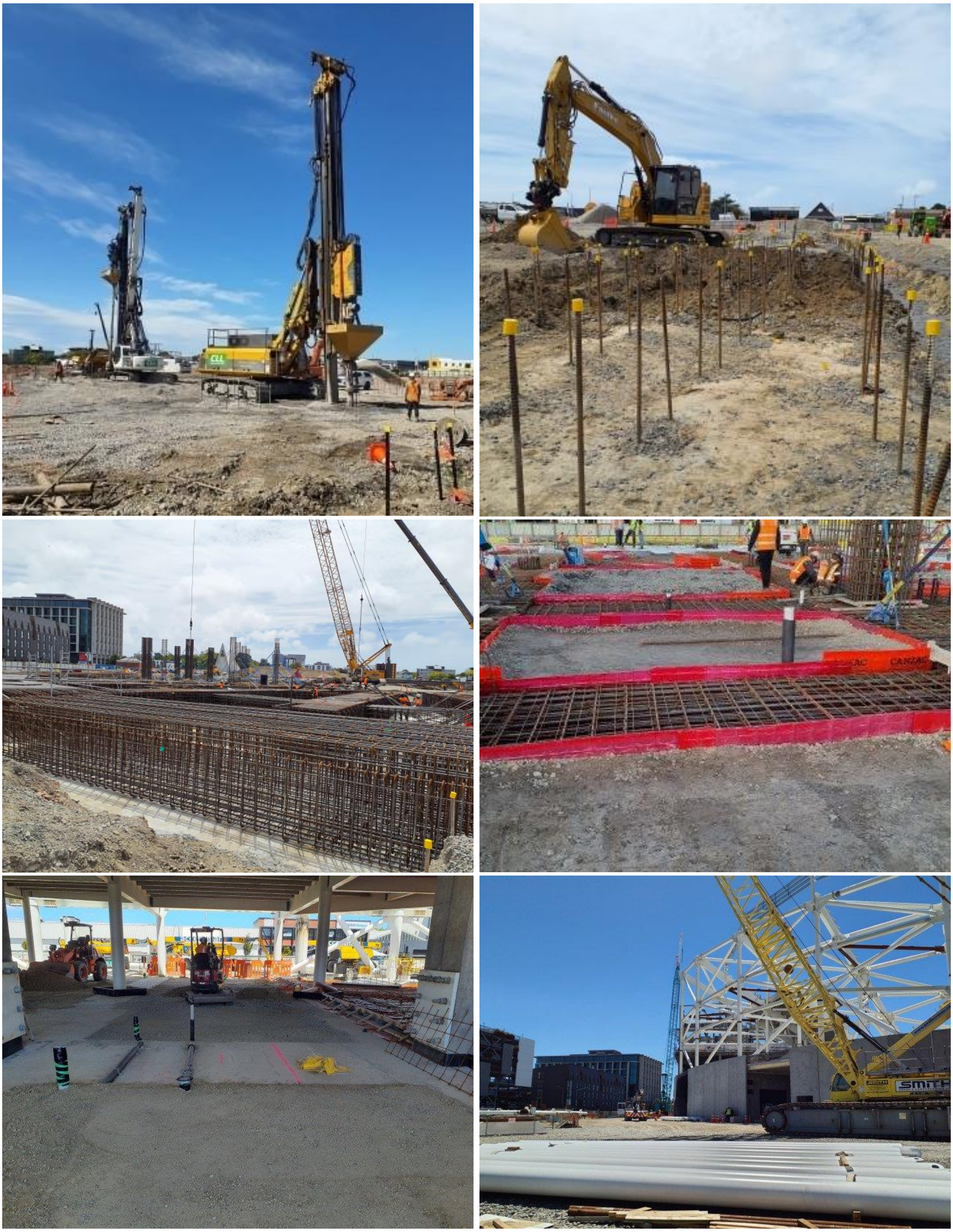

Figure 7. From top left to bottom right: RAP construction; excavation of ground beams and exposed subgrade; placement of steel reinforcing; infilled grillage beams; finished grillage beam pods; commencement of superstructure construction on completed foundation system.

Figure 8. Left - historical foundation beam encountered during enabling works; right - historical oil sump within the foundation footprint encountered during foundation excavation.

6.2 Variation in Subgrade Material

All foundation subgrades were inspected by geotechnical engineers to ensure suitable bearing material. Natural variation in material, material softened due to long term saturation, and uncontrolled materials in locations of thick fill were encountered across the site. Unsuitable materials were excavated and replaced with compacted AP65 hardfill or mass concrete. In some locations where deeper replacement was required, a layer of clean gravel wrapped in geotextile was installed as a starter layer to provide a solid base on which to compact.

6.3 Groundwater and Artesian Pressures

Shallow groundwater at the site and artesian pressures in deeper soil units were a factor in adopting a shallow foundation solution. The founding level was based on the shallow groundwater level and was intended to minimise its impact on construction and reduce the need for dewatering.

Dewatering wells were installed deeper than the base of the ground improvement. where required. Pumping water from a lower level ensured that ground water would be drawn downward vertically through soil layers and RAP columns, minimising the potential for lateral groundwater flow that could draw in material from adjacent silty soils into the RAP columns and compromise their performance.

All construction elements were significantly shallower than the artesian aquifer in the Riccarton Gravel layer, however, several undocumented historical wells connecting the aquifer to the ground surface were encountered during excavation for foundations. In each case these wells were appropriately decommissioned, and the subgrade was excavated and replaced where saturation from the wells had softened the material.

6.4 Reuse of site-won material

Key drivers for adopting a system of intersecting grillage beams were cost saving and sustainability. Using site-won materials as infill between grillage beams contributed to these savings significantly.

Site-won material was difficult to classify as an earthworks material with appropriate compaction targets due to the resultant matrix being a mix of predominantly four materials: uncontrolled fill on the site (Unit 1); natural sandy silts and silty sands (Unit 2); clean stone from the top of RAP columns within the excavation zone; imported AP65 used as the temporary RAP construction platform. Furthermore, the proportions of each material type within the stockpile varied depending on the location from which it was excavated.

The following methods were adopted to ensure that adequate compaction was achieved where site-won materials were reused on site as engineered fill:

- Increased frequency of laboratory and in-situ earthworks testing for site-won materials to identify changes in suitable compaction targets.
- Laboratory maximum dry density and optimum moisture content targets adjusted for over-sized particles in accordance D4718 (ASTM International, 2015).
- Field compaction trials including plateau testing completed onsite to determine minimum compaction method and confirm that assumed target was suitable.
- Combination of field testing and method compaction adopted to ensure minimum compaction effort was applied.
- Extensive plate load testing of placed material to confirm that minimum performance requirements had been achieved.
- Frequent visual inspections of all infill pods to identify settlement of placed fill.

6.5 Settlement monitoring

Settlement monitoring pins have been installed on the foundation beams and raft to monitor the static performance of the substructure as load from the superstructure is applied during construction. This monitoring is in its early stages, but to date has indicated settlements within the range of settlements predicted during detailed design.

7 CONCLUSIONS

Geotechnical works for Stage 1 of Te Kaha construction are complete at the time of writing this paper. Significant and complex geotechnical risks and challenges have been identified and overcome during the planning, design, and construction phases. All ground improvements have been constructed and most of the arena foundation is complete. The arena structure settlements are being monitored. The adopted settlement monitoring system will help to assess the arena structure performance under static loads and in future earthquakes.

8 ACKNOWLEDGEMENTS

The following parties contributed to the geotechnical design and construction at Te Kaha, and their work has been referenced throughout this paper: WSP – lead geotechnical engineers for Te Kaha; CLL Solutions Limited and Tonkin + Taylor Limited – design and construction of ground improvement; Bradley Seismic Limited – peer review of probabilistic seismic hazard assessment (PSHA) and seismic ground response analysis (SGRA); Mott MacDonald NZ Limited – project civil and structural engineer; Lewis Bradford Limited – structural peer reviewer; Faulks Investments Limited – earthworks subcontractor; McMillan Drilling, A1 Diggers Limited, WSP Laboratories – geotechnical investigations subcontractors and field and laboratory testing; Geocivil Limited – earthworks verification testing; and Besix Watpac – head contractor and project manager for Te Kaha.

REFERENCES

ASTM International (2015). "*Standard Practice for Correction of Unit Weight and Water Content for Soils Containing Oversize Particles*". D4718/D4718M, ASTM International, West Conshohocken.

Forsyth, P. J., Barrell, D. J., & Jongens, R. (2008). *Geology of the Christchurch Area.* GNS Science, Wellington, New Zealand.

GNS Science – Te Pū Ao. *New Zealand National Seismic Hazard Model*. https://nshm.gns.cri.nz/ (accessed 15 January 2024)

Idriss, I., & Boulanger, R. (2014). *"CPT and SPT based Liquefaction Triggering Procedures".* UCD/CGM-14/01, University of California at Davis, Davis.

Lees, J, van Ballegooy S and Wents, FJ (2015). "Liquefaction susceptibility and fines content correlations of Christchurch soils". *6th International Conference on Earthquake Geotechnical Engineering,* 1-4 November 2015, Christchurch.

Standards New Zealand (2004). "*NZS1170.5: Structural Design Actions. Part 5: Earthquake Actions - New Zealand*". Standards New Zealand, Wellington, 76pp. https://www.standards.govt.nz/sponsored-standards/building-standards/NZS1170-5

Zhang G, Robertson p and Brachman R (2002). "Estimating liquefaction-induced ground settlements from CPT for level ground." *Canadian Geotechnical Journal,* **39:** 1168-1180.