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Geotechnical challenges and design solutions for Yarrow Stadium redevelopment

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ABSTRACT

New Plymouth Yarrow Stadium's two grandstands were declared earthquake-prone and closed to the public in 2017. The $50M repair and refurbishment project was approved in 2019 and subsequently reviewed and modified to allow for the impacts of the COVID-19 pandemic. The Stadium's West Stand supported on a 10 m high fill embankment experienced seismic structural damage due to the lateral movement of the embankment. The foundation soils and the embankment are formed by sensitive Taranaki Ash. While the building structure could be strengthened to withstand design earthquakes, future seismic settlement and lateral movement of the embankment had to be prevented by improving the ground at the toe of the embankment slope. Geotechnical design challenges included assessment of cyclic softening of the Taranaki Ash and consideration of several ground improvement and foundation strengthening options. Geotechnical design process and design optimisation procedures are described.

1 INTRODUCTION (PROJECT DESCRIPTION)

Yarrow Stadium was developed as a modern stadium for Taranaki in New Plymouth, New Zealand in 2002. The West Stand is a piled steel frame structure with a reinforced concrete (RC) canopy, located on a fill embankment (Figure 1). The West Stand fill embankment is approximately 10 m high above the playing field level, with an average slope angle of 30 degrees.

The November 14, 2016 Kaikōura Earthquake has caused movement and settlement of the West Stand embankment slope, resulting in cracking of the fill around the existing pile caps and bleacher pad foundations resting on benches cut into the fill embankment. Detailed seismic assessment of the West Stand carried out by TSE and WSP identified a number of structural and geotechnical deficiencies. Substantial cracking of the fill embankment beneath the RC bleachers was observed, with the cracks extending around the pad foundations supporting the bleacher seating, indicating instability of the soil benches. The structure was declared to be earthquake-prone in 2017 and closed to the public.

Figure 1. View of the site (left), West Stand (right)

The benefits and the value to the community from a stadium like Yarrow Stadium greatly surpass the revenue directly generated from sporting events. The community multi-use and the economic activity generated in the Taranaki economy makes this stadium an important piece of a vibrant urban centre which attracts people and businesses. Hence, the repair and redevelopment of the stadium were deemed as a strategic investment by the New Plymouth District Council (NPDC) and Taranaki Regional Council (TRC) who commissioned a $50M project to strengthen the West Stand and rebuild the East Stand structures. WSP was commissioned to undertake geotechnical seismic assessment and geotechnical design for the seismic strengthening of the West Stand.

2 SITE CONDITIONS

The historical geotechnical information for the West Stand dates back to the construction of the stadium, with boreholes and Cone Penetration Tests (CPTs) carried out in 1997 and 2001.

In 2018 WSP carried out additional geotechnical investigations (boreholes, CPTs, hand augers with Scala Penetrometer tests) to refine the West Stand ground model and assist with the design of the strengthening. The typical ground profile comprises:

- 10 m high fill embankment formed by sensitive volcanic firm to stiff silt (reworked Taranaki Ash), over
- 7 m of in-situ soft to firm saturated sensitive volcanic silt (Taranaki Ash), over
- 8 m of in-situ firm organic silt (Lacustrine deposits), over
- More than 10 m or medium dense to very dense sandy silty gravel (Welded Tuff)

Geotechnical investigations indicated that the fill embankment is formed by various silt materials. These materials have been classified as sensitive Taranaki Ash. A layer of in-situ soft to firm silt of the same nature is present at the base of the fill embankment.

In-situ Geonor Shear Vane tests of the Taranaki Ash materials indicated that their residual shear strength was 7 to 16 times lower compared to the peak strength value, indicating that these materials are sensitive to very sensitive. Also, there was some uncertainty with respect to the material’s liquefaction potential.

The ultimate limit state (ULS) peak ground acceleration for the stadium site is 0.33g.

3 CYCLIC BEHAVIOUR OF TARANAKI ASH

Understanding of cyclic performance of the Taranaki Ash silt forming the fill embankment and its foundation was one of the main challenges for the project. Substantial reduction in the material strength under cyclic loads could potentially lead to very large vertical and horizontal displacements of the fill embankment which would result in overstressing the piles supporting the building and instability of the bleachers.

WSP, in collaboration with the University of Auckland developed a detailed field and laboratory testing programme to study the sensitivity of Taranaki Ash silts and their potential for cyclic softening and liquefaction. There are no commonly accepted methods to assess liquefaction potential or predict strength loss for sensitive volcanic silts under cyclic loading. It was therefore critical to carry out a study of the dynamic response of the sensitive soils. Cyclic triaxial tests of undisturbed soil samples from boreholes were carried out for a range of cyclic stress ratios (CSRs) to confirm dynamic response of the sensitive soils.

The Taranaki Ash soil samples were tested in the Auckland University testing laboratory. Typical cyclic triaxial test results are shown on Figures 2 and 3. CSR value for the tests shown on Figures 2 and 3 was 0.52. The test results showed stiffness degradation as the number of loading cycles increased (Figures 2 and 3).

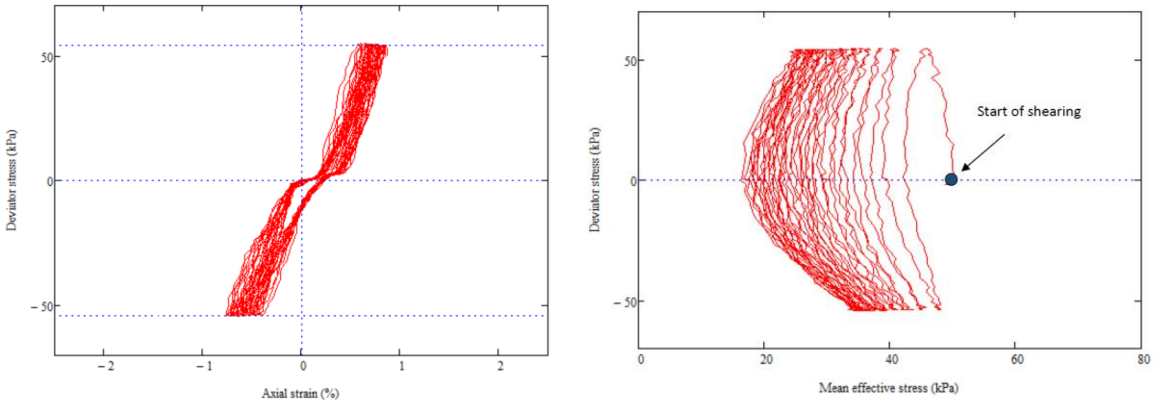

Figure 2. First 20 load cycles: deviator stress – axial strain loops (left); effective stress paths (right)

The axial strain amplitude reached about 5% at about 50 cycles (Figure 3). However, the soil showed no sign of strength loss due to sensitivity and was able to maintain the same level of shear strength albeit requiring further deformation to mobilise the strength as the duration of loading increased throughout the test.

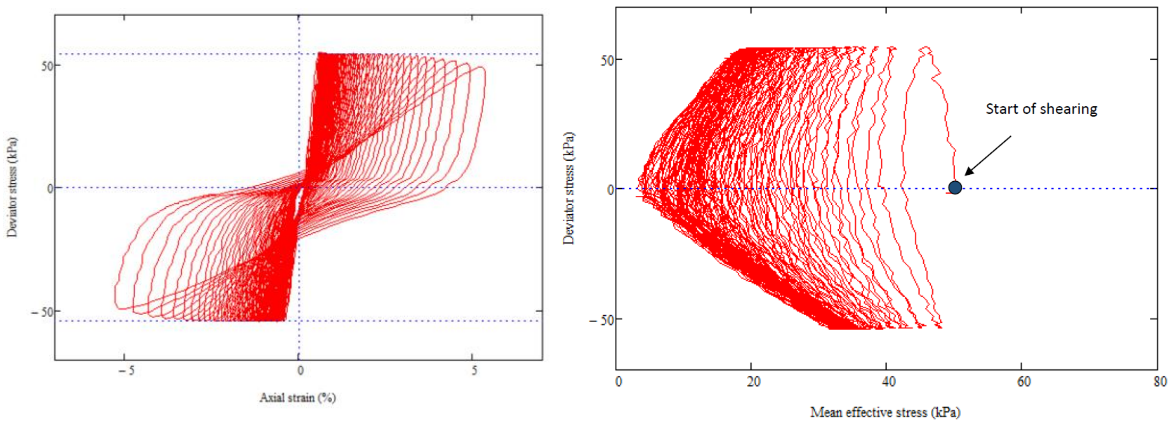

Figure 3: 65 load cycles; deviator stress – axial strain loops (left); effective stress paths (right)

While some excess pore pressure build-up was recorded, no full liquefaction condition ($r_u = 1$) was achieved. We therefore concluded that the silt is not prone to liquefaction, cyclic softening or substantial strength loss due to sensitivity. A more detailed consideration of Taranaki Ash sensitivity and liquefaction issues is given in Murashev & Tai (2021).

The cyclic triaxial tests provided valuable information on the dynamic behaviour of the soil and enabled us to assess the stability of the fill embankment, the building platform, the slope benches and develop reliable new structural foundation options with a higher level of reliability.

4 ORIGINAL FOUNDATION SYSTEM

The existing West Stand is supported on three types of foundations (Figure 4): driven RC piles supporting the West Stand structure and the canopy, shallow RC pads supporting bleacher beams, and a continuous RC foundation beam along the rear wall of the structure.

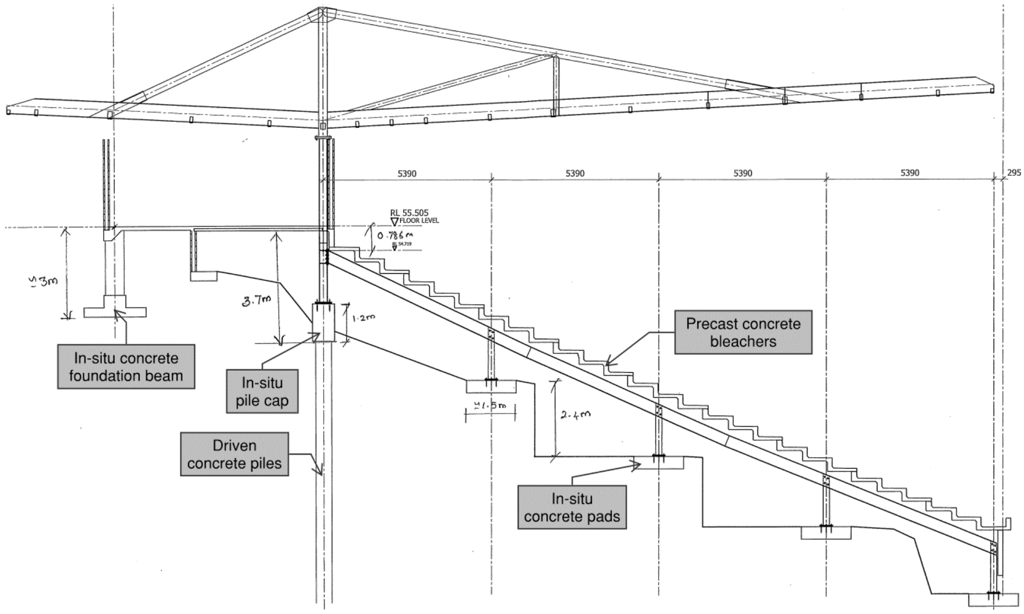

Figure 4: West Stand original foundation system

5 SLOPE STABILITY

As part of the preliminary slope stability assessment, the global stability of the West Stand embankment and local stability of the slope benches providing support to the pad footings were analysed using *Slope/W* software for static and seismic conditions. The results have shown that the fill embankment was stable under static conditions and serviceability limit state (SLS) earthquake. However, for the ultimate limit state (ULS) earthquake, some lateral displacement (up to 0.4 m) would be expected, and the magnitude of the displacement is sensitive to the strength parameters of the fill and the underlying soil. Such a large lateral displacement would result in failure of the existing RC piles and columns, and deferential slope movement in the longitudinal and transverse direction. Our analysis also indicated that the slope benches supporting the bleacher pads could potentially fail in the SLS earthquake under full live load.

It should be noted that the slope displacements and potential for bench failures would be even higher if substantial soil strength reduction would occur due to cyclic softening, liquefaction or sensitivity. Therefore, our study of the cyclic behaviour of the site materials (refer to Section 3) enabled us to avoid unnecessary conservatism.

Based on our analysis, we concluded that large global movement of the fill embankment and local failures of bleacher foundations should be prevented. Considered options included ground improvement to reduce lateral movement of the whole embankment, piles to support the bleachers and mitigate the risk of bench failures and steel soil anchors at the rear of the structure to resist overturning moment generated by the weight of the canopy.

Based on the consideration of the existing and new RC piles performance, we decided to limit the slope displacements to less than 200 mm at pile locations, as larger displacements would result in failure of the piles.

6 DESIGN CONCEPT

The adopted design concept is shown on Figure 5, and includes: steel beams between the canopy columns and the ground beam (elements 4), piles to support belchers foundations (elements 11) , additional buttressing geogrid-reinforced structural fill (shown by grey shading) to increase stability of the slope, ground improvement (shown by red shading) at the toe of the slope to reduce the overall horizontal movement of the embankment, ground anchors (elements 6) to reduce bending moments and displacements of the piles supporting the canopy.

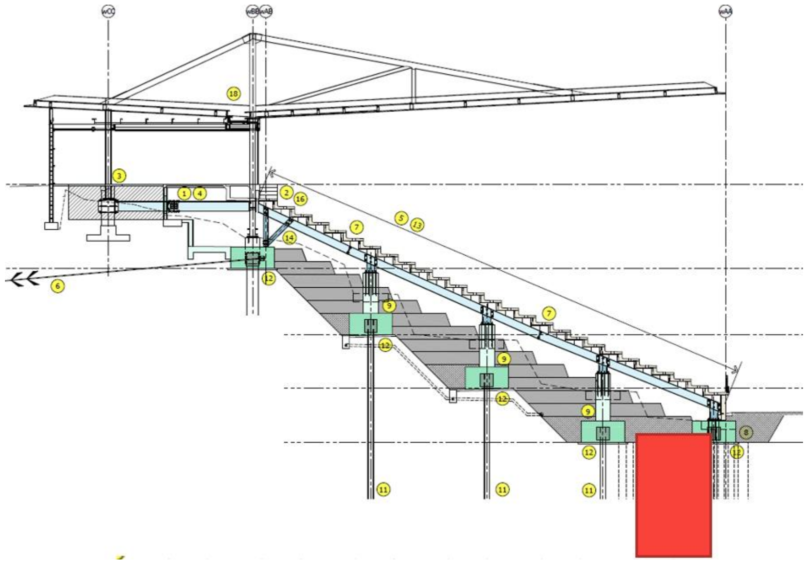

Figure 5: West Stand strengthening concept

Several ground improvement options were considered, including sheet piles in cellular arrangement, deep soil mixing and steel tube piles. The ground improvement options were compared in terms of the slope displacement reduction level, cost, construction and performance risks as well as resilience and sustainability. Sheet piles in cellular arrangement at the toe of the slope were initially considered. However, given the spatial variability of the soils, the presence of underground service and the risk of not being able to drive the sheet piles into the very dense gravels, this option was discounted. The deep soil mixing (DSM) option was considered in detail. This included consideration of different layouts of stabilised soil elements in plan to optimise the design and reduce the cost. The steel tube pile option was also considered in detail.

7 DEEP SOIL MIXING

A detailed 3D ground model was developed (Figure 6) and finite element (FE) software Plaxis 3D was used to analyse the problem. In pseudo-static Plaxis analysis of seismic performance of the model, the ground was subjected to horizontal volume forces inducing the assessed slope displacements from simplified analysis

(Figure 7). Subsequently, different types of ground improvement were introduced into the Plaxis 3D model, and slope displacement reduction resulting from each ground improvement option was calculated in Plaxis 3D.

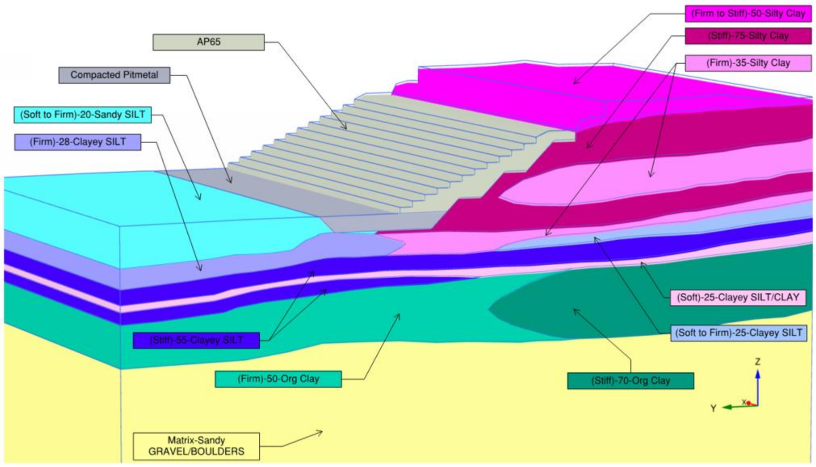

Figure 6. Ground model in Plaxis 3D

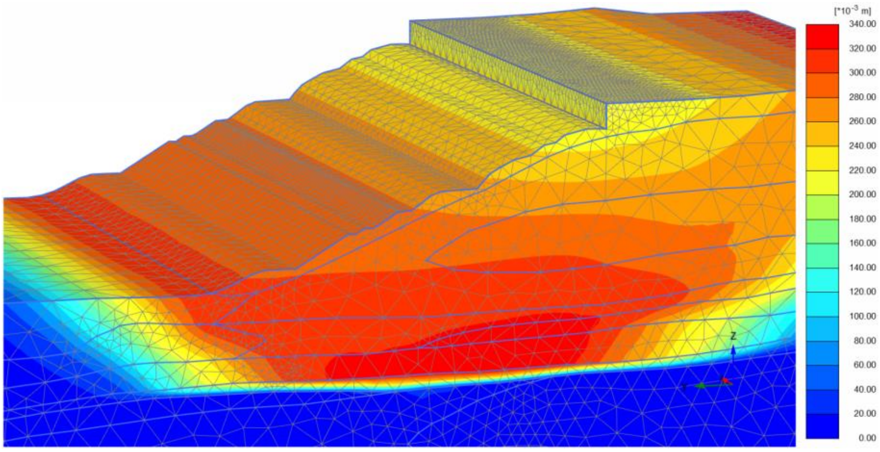

Figure 7 Free-field ULS slope displacements (Plaxis 3D output)

In-situ DSM is a solidification technique that involves mechanically mixing in-situ soils with cement grout or binder slurry rotating augers attached to rods to form a grid of individual DSM columns or panels comprising of overlapping DSM columns. The final product is stabilised soil mass consisting of a weakly grouted soil with unconfined compressive strength, in our case, of approximately 2 MPa. Laboratory reactivity test were caried out to prove the efficiency of DSM technique in our site materials and confirm their post-improvement strength. Three DSM options were analysed: Mass DSM of a large soil block at the toe of the slope; Cellular DSM and T-shaped DSM. The mass stabilisation option would be more costly than cellular or T-shaped DSM. Therefore, only cellular and T-shaped DSM were considered in detail.

7.1 Cellular DSM

This option comprised of 6m x 6m DSM cellular walls with a wall thickness of 0.6 m and a depth of 6.5m located at the toe of the slope. Our Plaxis analysis showed substantial concentration of bending moments at the joints between the longitudinal and transverse walls. The bending moments exceeded the capacity of the DSM walls (Figure 8). Different geometries of the cellular walls were analysed, however bending moments in the walls were excessive. To reduce bending moments and increase the bending capacity of the walls, a more rigid T-Shaped DSM option was analysed.

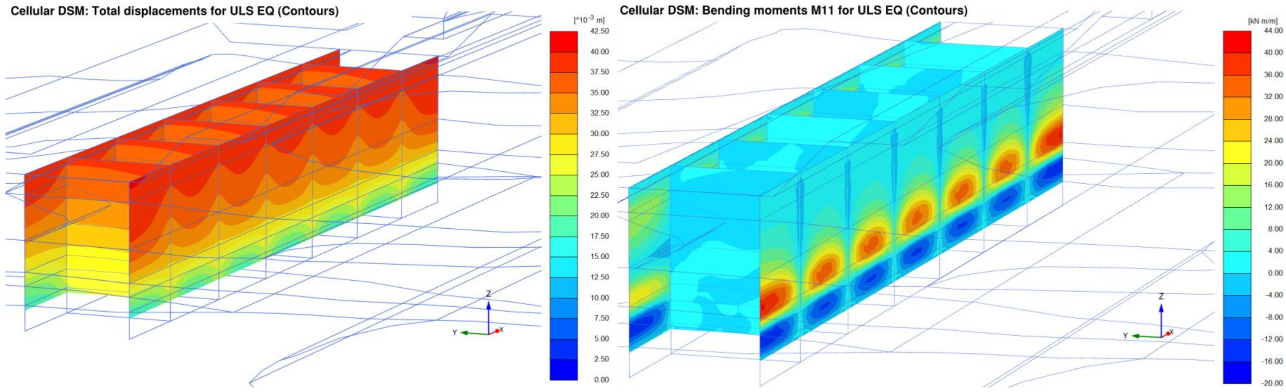

Figure 8: Cellular DSM - Plaxis 3D outputs: total displacements (left), bending moments M_{zz} (right)

7.2 T-shaped DSM

This option comprised a row of T-shaped DSM elements with a 3m x 1.3m flange and 6m x 1m web (in plan) with a depth of 6.5m located at the toe of the slope at 7.3 m spacings. Results of our Plaxis 3D analysis (Figure 9) showed concentration of bending moments at the web to flange joints but the moments were within the DSM T- shaped section capacity and the slope displacements were reduced to less than 150 mm.

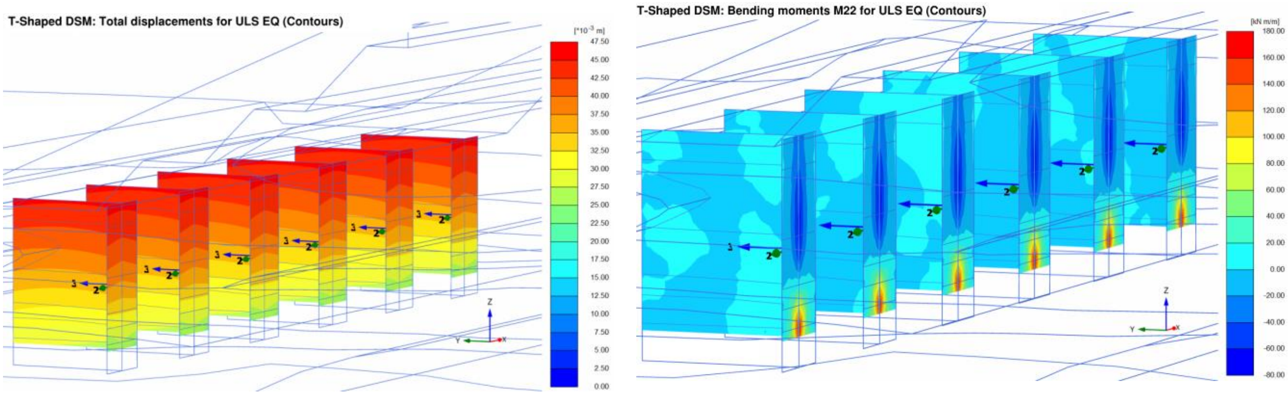

Figure 9: T-shaped DSM - Plaxis 3D outputs: total displacements (left), bending moments M_{xx} (right) in T-shaped DSM elements

8 GROUND IMPROVEMENT PILES

Ground improvement in the form of a row of steel tube piles was also considered. The design of the ground improvement piles involved analysing different pile sizes, lengths and spacings and comparing the results in terms of ground displacement and structural performance of the piles. Similar to the DSM option, the ground improvement piles were placed at the toe of the embankment slope. Our Plaxis 3D analysis (Figure 10) showed that a single row of 1 m diameter 12 m long steel tube piles at 7.3 m spacings would reduce slope displacements to a tolerable level, with structural capacity of the piles not being exceeded. Slope displacements were reduced to less than 140 mm.

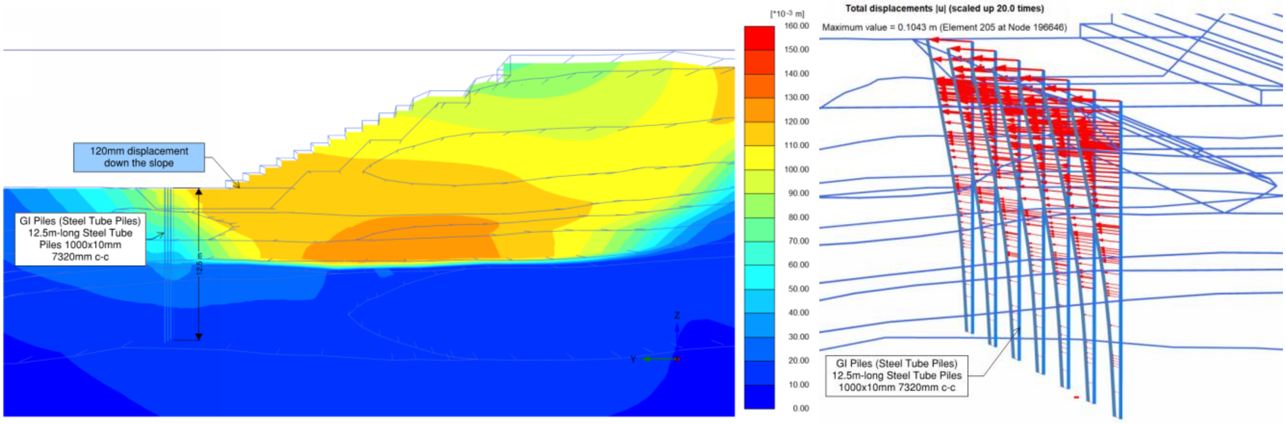

Figure 10. Ground Improvement Piles - Plaxis 3D outputs: total slope displacements (left), total pile displacements (right)

9 DISCUSSION AND CONCLUSIONS

DSM and steel piles were considered to improve the West Stand embankment stability. Different layouts and shapes of DSM were considered, including mass stabilisation of a large block of soil at the toe of the embankment, cellular DSM walls, and T-shaped DSM elements at the toe of the slope. A 3D finite element analysis was carried out to assess shear and bending in the cement-stabilised soil walls and steel piles as well as embankment displacements. Our analysis indicated that all DSM layout options reduce the maximum seismic displacement of the fill embankment to a tolerable level. However, the required volume of stabilised soil was 50% and 65% less for the cellular and T-shaped DSM respectively, compared to mass stabilisation of a large soil block. Steel piles would have more ductile behaviour compared to the DSM walls that could get overstressed and experience brittle failure in the ULS earthquake. The ground improvement steel tube piles option was adopted due to more reliable seismic performance, quick and simple construction and lowest construction cost. The ground improvement piles, geogrid-reinforced buttressing fill and the piles supporting the bleachers were successfully constructed in 2021.

10 ACKNOWLEDGEMENTS

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11 REFERENCES

Murashev, A. & Tai A. 2021. Performance of sensitive Taranaki volcanic silts under cyclic loading. Proceedings of the 2021 New Zealand Society for Earthquake Engineering Annual Technical Conference.