NUMERICAL MODELLING OF MICROPILES

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ABSTRACT

Micropiles lend themselves to the formation, using vertical and raked piles and a connecting beam, of A-frames which create a very strong and rigid buried structure. These can be used to support significant lateral forces, such as in bridge abutments and in slope stabilisation works.

Design for slope stabilisation is readily carried out using the tried and tested FHWA method, based on limit equilibrium and satisfying the requirements of limit state or LRFD design. However, it has also been found necessary to examine serviceability limit state design criteria, and this is readily done through numerical methods, in this case 2D plane strain using PLAXIS 2D2018. The results can give clients reassurance with regard to the sort of ground and structure movements they should expect.

Examples are given of applications for both bridge analysis and design, where conventional structural analysis was used, and for slope stabilisation work.

INTRODUCTION – BRIDGE FOUNDATIONS

Byron Shire Council (BSC) area lies about 60 km south of the Gold Coast, in northern coastal New South Wales, as seen in Figure 1. In late 2017 the Council called for tenders to construct five bridges to replace small timber bridges in the south west of the shire as seen in Figure 2, to improve safety for road users, achieve higher load limits, and reduce maintenance costs. At least one of the bridges had been closed to

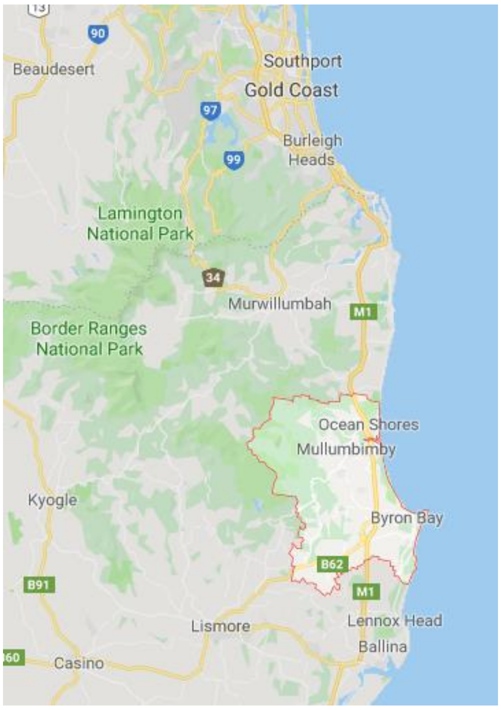

Figure 1. Location of Byron Shire

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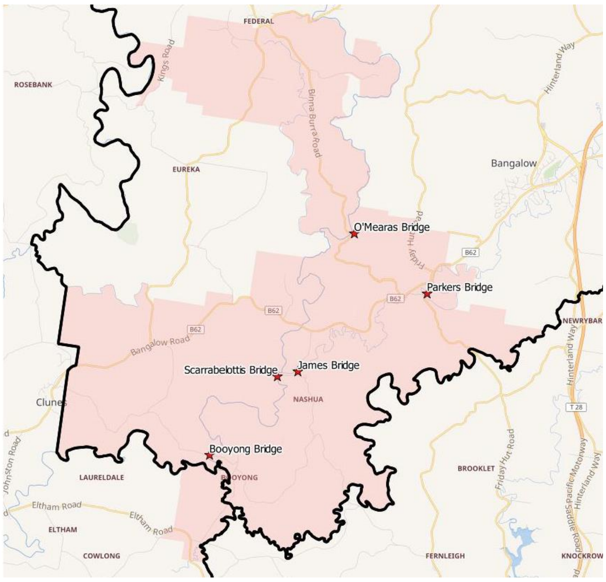

Figure 2. Bridge locations

traffic completely following damage caused by flooding in June 2016 which overtopped the bridge deck by about 1.3 m.

Site investigations had been carried out at each abutment of each bridge site, and recommendations had been made for conventional foundations consisting of large diameter bored piles or driven precast concrete piles. Designs for both of these foundation types had been included in the tender package. The replacement bridges were to be modular steel bridges which were being sold by the Australian Defence Force Line of Communication, as illustrated in Figure 3.

Figure 3. A typical LOC modular steel trussed bridge

ALTERNATIVE DESIGN

During the tender evaluation process BSC explored an alternative foundation system based on micropiles proposed by a specialist piling contractor in association with one of the main contractors. This had the advantage that it made us of much lighter and more manoeuvrable plant, which minimised the impact of the load restrictions on local bridges and problems of narrow road widths. The contract was awarded to this team on the basis of the alternative design.

BSC were being advised by a specialist bridge designer, but neither had any previous experience of the use of micropiles for bridge foundations. It was therefore agreed that the specialist piling subcontractor would carry out the design, but that an independent consultant would be hired to carry out an independent design/peer review. National Geotechnical Consultants were engaged to provide this service, having previously worked with the same specialist piling contractor to carry out numerical modelling of micropiles for slope stabilisation work.

DESIGN APPROACHES

The basic design by the specialist piling contractor was carried out using the software package GROUP2016, as described in Wilson (2019). The design uses multiple piles (16 for each abutment of all bridges except one which uses 20) in an A-frame arrangement. Each pile consists of a hollow deformed bar with a sacrificial cutting head, through which a weak grout (0.8:1 water/cement ratio) was pumped during forming of the hole. This grout was expected to stabilize the hole, partially by impregnating the surrounding soil, as well as flush the drillings to the surface. Once the required depth had been reached the flushing grout was replaced with a thicker bonding grout (0.45:1 water/cement ratio) from the toe up to the head of the pile. In order to provide additional bending restraint to the length of the pile near the surface, where moments were expected to be greatest, circular hollow sections (CHS) were placed around the bar.

The specialist bridge designers provided tabulated load case and load combination information, as dead and live vertical loads, together with ultimate load factors, lateral loads due to braking, and scour depths for each of the five bridges, as well as unfactored lateral loads due to stream forces and three specific load combinations. This resulted in six load cases as shown in Table 1.

Table 1. Six load cases used for design

Load case	**Load combination**
1	1.1 x Permanent effects + 2 x Live Load + 1.8 x Braking load + 1 x Stream force at deck level
2	Permanent effects + 1 x Live Load + 1 x Braking Load + 1.3 x Stream force at deck level
3	Permanent effects + Ultimate stream forces (1:2000 AEP flood)
4	0.9 x Permanent effects + 0 x Live Load + 1.8 x Braking load + 1 x Stream force at deck level with scour
5	0.9 x Permanent effects + 0 x Live Load + 1 x Braking Load + 1.3 x Stream force at deck level with scour
6	0.9 x Permanent effects + Ultimate stream forces (1:2000 AEP flood) with scour

Consideration was given to a review of the contractor’s design but, since much of this was contained in printout from proprietary programs, it was decided that this would not meet the needs of BSC. Noting also that NGC did not have the GROUP16 program, it was decided that an independent design would be carried out using alternative software. In order to analyse the pile groups to determine the maximum compressive and tensile axial loads, and the maximum bending moments and shears, the industry standard program PIGLET Version 5.1, developed by Professor Mark Randolph of the University of Western Australia, was used. To quote the manual, the program “analyses the load deformation response of pile groups under general loading conditions. The program is based on a number of approximate, but compact, solutions for the response of single piles to axial, torsional and lateral loading, with due allowance made for the effects of interaction between piles in the group. In these solutions, the soil is modelled as a linear elastic material, with a stiffness which varies linearly with depth." The important aspect of this program is that it models the soil as a continuum, such that pile-soil-pile interaction is captured. This is significantly different from traditional structural programs such as Spacegass and Strand7, which model piles as linear elastic springs on a rigid base, in which no pile-soil-pile interaction is modelled. The GROUP16 program used by the designers is understood to use p-y curves to model the effect of lateral loads on the piles, and it is not clear to what extent this models pile-soil-pile interaction, nor is it clear to what extent this is significant for the pile geometry at these bridges.

As noted above, additional CHS pipes were added near the ground surface to increase the moment capacity of the micropiles. In order to model the axial loads the various sections were combined into an equivalent solid bar, but this underestimated the stiffness (second moment of area) by a factor of about 3. The functionality of PIGLET which allows a different modulus value to be used for lateral loading was therefore employed to give an EI value which was 4070 kNm^2, similar to the bar plus two CHS pipes at 4179 kNm^2. With this modification the axial loads, bending moments and shears were acceptably close to those determined by GROUP16.

INTRODUCTION – SLOPE STABILITY

A completely different project involved the use of the same A-frame structures to stabilize a rural road built on sidelong ground. Because of the topography and geomorphology traditional methods using retaining structures such as gabions, and coarse rock fill, were unsuitable. The ground surface, and about 3 or 4 layers of superficial soils, were all dipping at about 34°.

Again the specialist piling contractor designed the micropiled A-frames, as illustrated in Figure 4. The designs were carried out in accordance with the FHWA

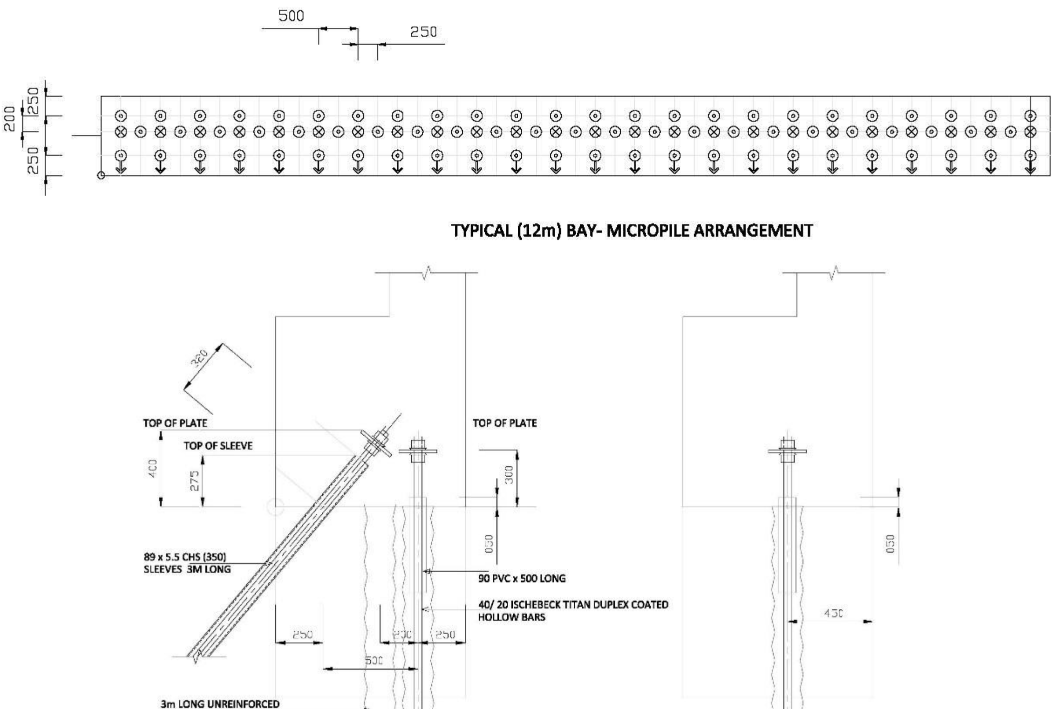

Figure 4. Plan and section of micropiles for slope stabilization

manual for micropiles (FHWA, 2006). This follows an LRFD (limit state design) approach which ensures structural safety but does not give a useful guide to actual performance. For that reason NGC were requested to carry out some numerical modelling using PLAXIS 2D in order to estimate the likely movements of the A-frame, capping beam and retaining wall.

NUMERICAL MODELLING – BRIDGE FOUNDATIONS

Using the soil data from each of the geotechnical reports, a geotechnical model was created for each abutment. There were differences across the width of the river in each case, and the lateral load for the flood case had to be applied in a downstream direction. With the limitations of 2D modelling sections were analysed in both the longitudinal (along the axis of the bridge) and transverse directions. This was a reasonable approximation for the longitudinal sections if considered as a slice through the centre of the headstock, which is relatively long and narrow. It is not quite so easy for the transverse section which, basically, considers a continuous platform into the paper supported by arrays of piles at selected centres.

One of the limitations in 2D finite element (FE) methods is the means of modelling the piles. Up until 2015 they could only be modelled as plates, which meant that they were continuous sheets into the paper. The EA and EI values per metre could be adjusted to give the correct equivalent stiffness for piles at known centres, but clearly it was not possible for soil to flow through the sheets in the same way as it could through the spaces between piles. This was improved in 2015 by the inclusion of embedded beam elements, specifically to model piles, soil nails, and the fixed length of ground anchors. These elements are attached to the FE mesh at either end, but exist outside the mesh along their length. Input data includes section (solid or hollow circular pile, or solid square pile), dimensions, weight density and elastic modulus, shaft friction and end bearing, and interface behaviour. This makes the modelling of pile behavior in a 2D model much easier, though not entirely without difficulty. In the subject case we needed to have two piles with different section properties connected end to end, to model the change in stiffness resulting from the additional CHS pipe, but the configuration of the embedded beam model does not allow them to be coupled end to end.

In order to overcome this difficulty it was necessary to bracket the problem. First the upper length of each micropile was modelled, with the appropriate section properties, to determine the bending moments and shear forces in that section of the pile. Secondly the lower length of each pile was modelled, connected to the capping beam with an "anchor", having appropriate section properties but no connection with the soil, such that the axial forces in the lower section could be predicted. The highest numerical results were tabulated as shown in Table 2.

Table 2 (a). PLAXIS output for longitudinal sections

	Bridge				
Model output	1	2	3	4	5
Longitudinal					
Ux (mm)	1.7	2.3	0	2	0
Uy (mm)	-9.3	-12	-5	-7	-12
Axial force in anchor (kN)	75.4	151.5	56	94	46.5
Axial force in micropile (kN/m)	56.5	113.5	41.6	70	36.5
Bending moment (kNm/m)	0.5	6.5	0.2	0.75	1.2
Shear force (kN/m)	0.7	6.6	0.4	1	2.3

Table 2 (b). PLAXIS output for transverse sections

	Bridge				
Model output	1	2	3	4	5
Transverse					
Ux (mm)	20	25.5	7	12	17
Uy (mm)	-39	-7.7	-21	-16	-32
Axial force in anchor (kN)	154	152	155	155	154
Axial force in micropile (kN/m)	256	256	258	258	257
Bending moment (kNm/m)	25.5	3.9	10	16.5	10
Shear force (kN/m)	26.5	7.5	14	19	15

These results, which were all determined using unfactored loads and strengths appropriate to FE analysis and therefore at the Serviceability Limit State, showed a number of things:

1. While each bridge was subject to separate analysis, in terms of soil profile and detailed load cases, the results appeared to be compatible with each other
2. The headstock response appeared to be stiffer in the longitudinal direction, which was reasonable when considering that this represented a 1 m slice of the 10 m long headstock, whereas in the transverse direction the 2D analysis was as if there were multiple headstocks 1 m wide and parallel with each other supported by micropiles at 600 mm centres, and each subject to the applied loads. The actual response is likely to be between the two
3. These suggested movements under serviceability loads were of the order of 20 mm, and were compatible with those suggested by the PIGLET analyses
4. Axial forces in the micropiles were less than those predicted by PIGLET, probably because the FE analysis allowed some support of the headstock by the soil
5. Bending moments and shear forces were small, even if they were multiplied by 3 to take account of the lower lateral stiffness of the equivalent bar compared to the bar and two steel tubes
6. Overall the results were compatible with those produced by the designer and the design was confirmed.

As an additional exercise a factor of safety on mobilized shear strength was carried out using the strength reduction technique in PLAXIS. In this process the shear strength parameters, c' and φ', were progressively reduced, allowing yield and perfectly plastic behavior to occur, until failure was reached and the applied load could not be supported. The results are shown in Table 3.

Table 3. Factors of safety

Bridge	Geotechnical Factors of Safety	
	Longitudinal section	Transverse section
1	2.8	2.1
2	1.6	2.3
3	2.04	2.5
4	2.36	2.06
5	1.48	2.6

Some typical graphical outputs from the modelling are shown in Figures 5 to 16.

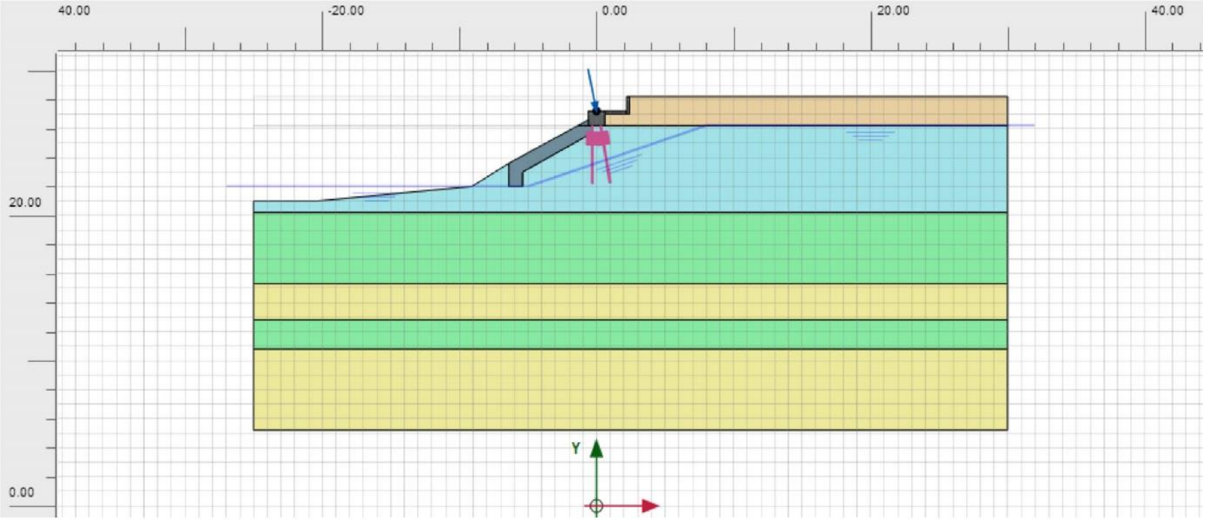

Figure 5. Bridge 2 input with top section longitudinal

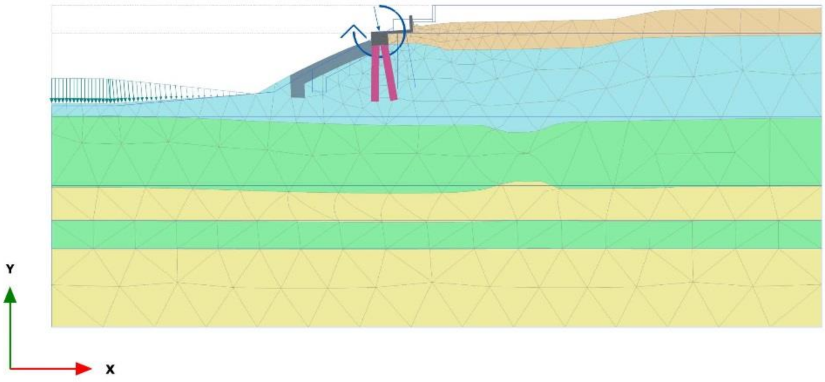

Figure 6. Bridge 2 deformed mesh for top section at SLS longitudinal

Figure 7. Bridge 2 bending moments for top section at SLS longitudinal

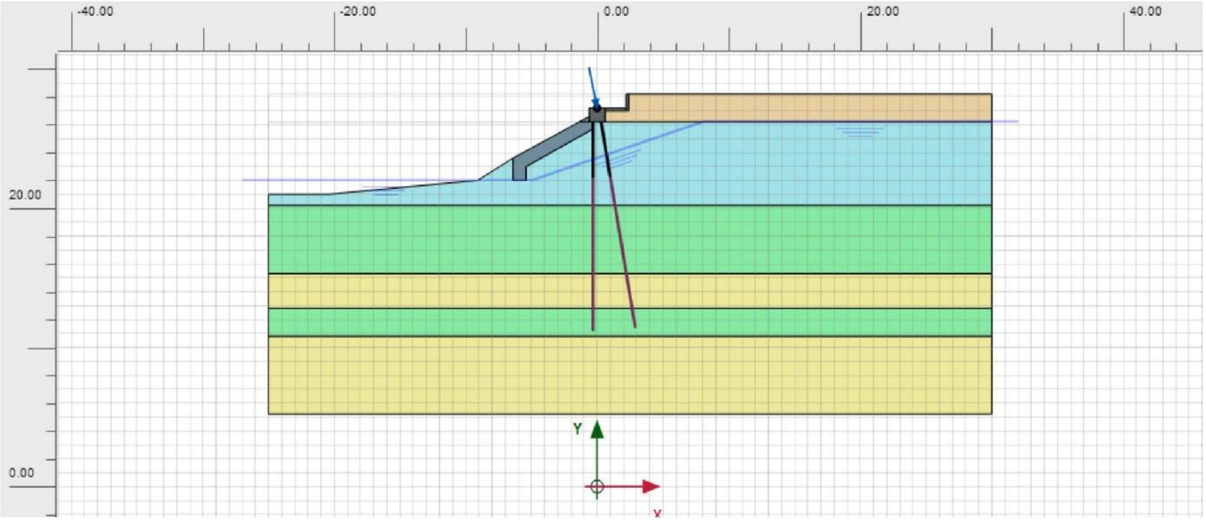

Figure 8. Bridge 2 input for lower section longitudinal

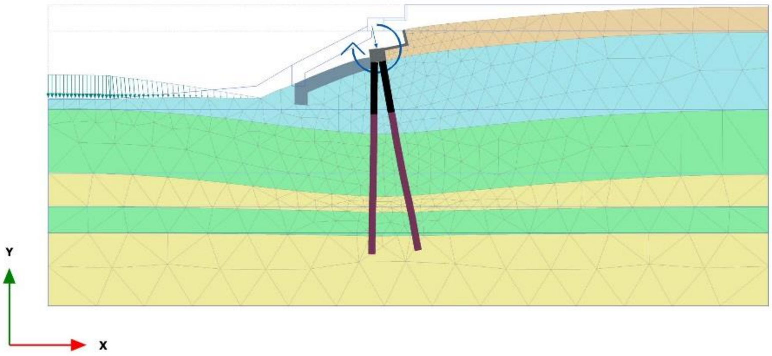

Figure 9. Bridge 2 deformed mesh for lower section at SLS longitudinal

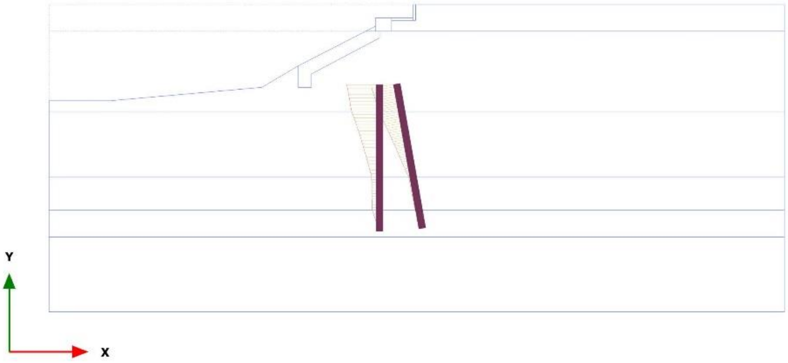

Figure 10. Bridge 2 axial forces in lower section at SLS longitudinal

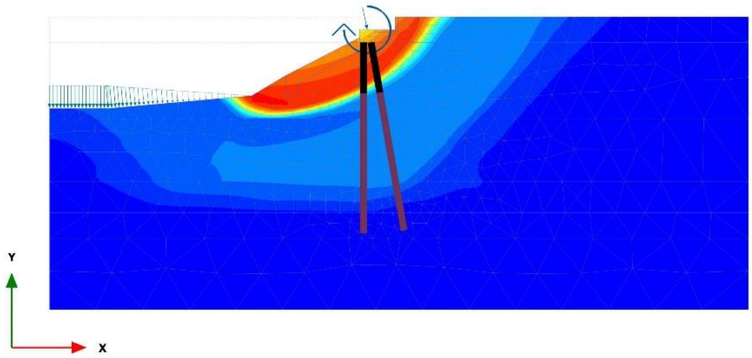

Figure 11. Bridge 2 failure surface longitudinal

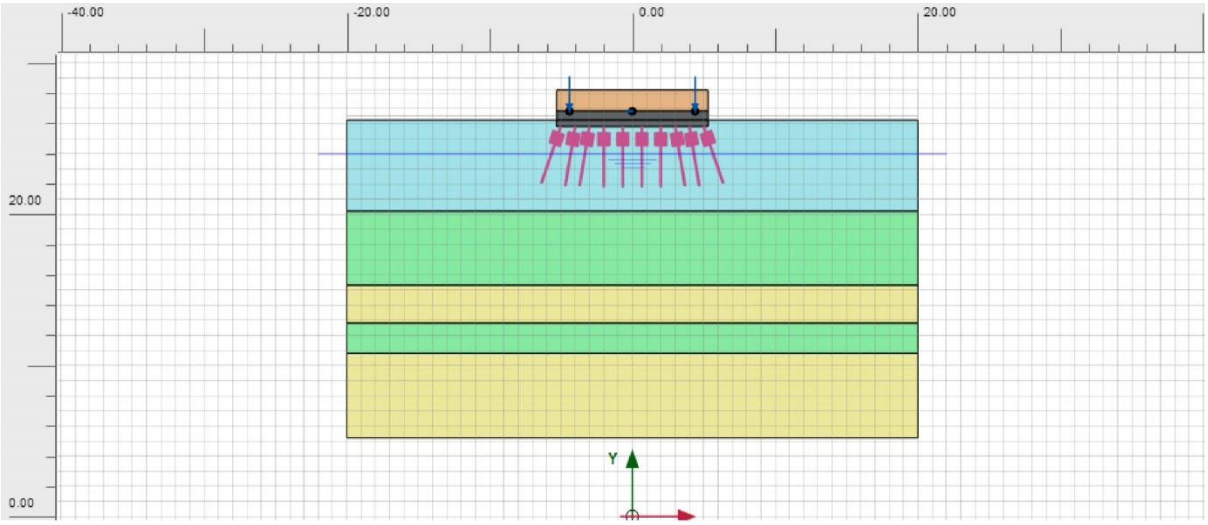

Figure 12. Bridge 2 input with top section transverse

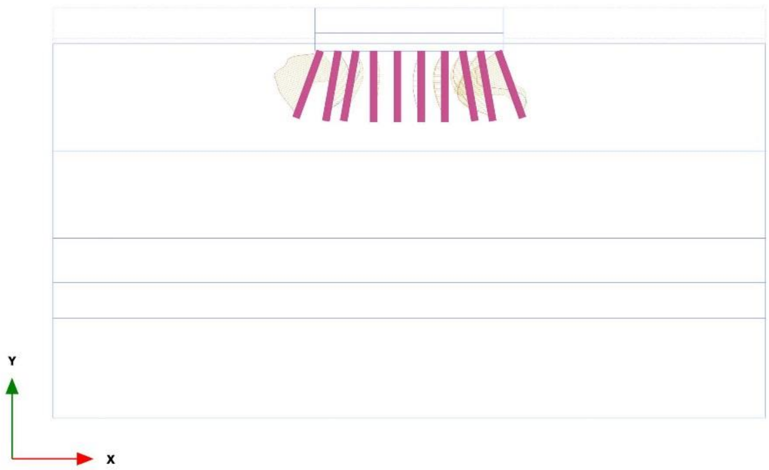

Figure 13. Bridge 2 bending moments for top section at SLS transverse

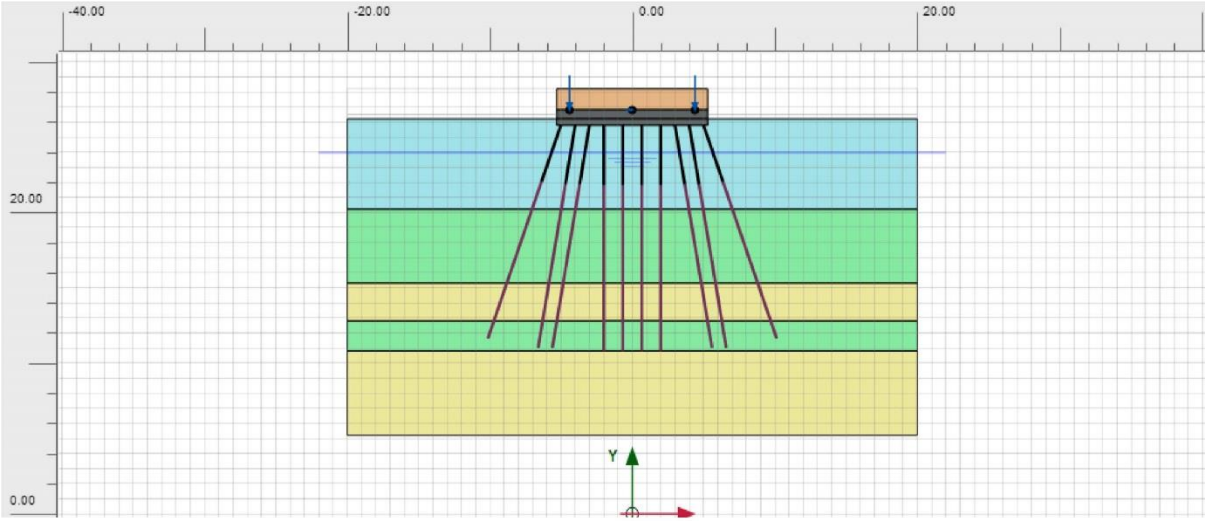

Figure 14. Bridge 2 input with lower section transverse

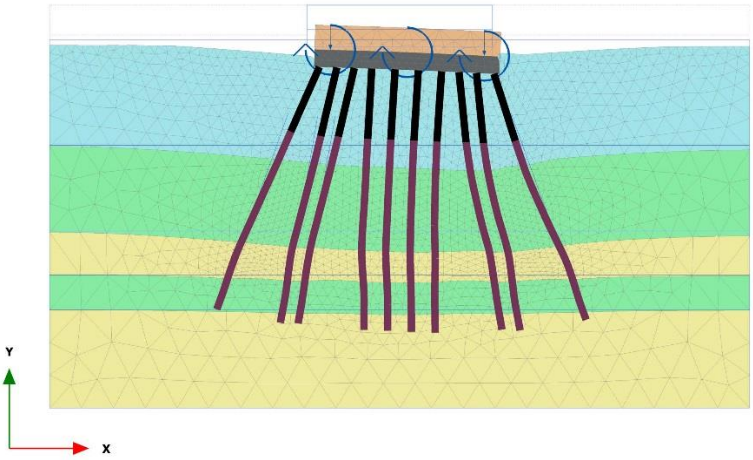

Figure 15. Bridge 2 deformed mesh for top section at SLS transverse

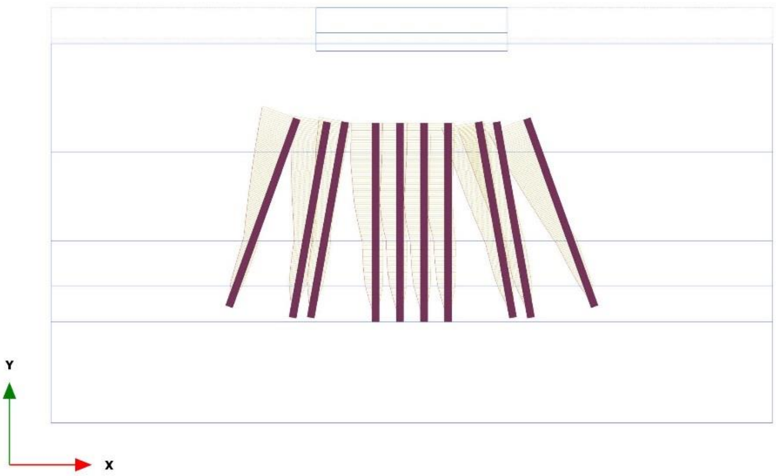

Figure 16. Bridge 2 axial forces for lower section at SLS transverse

NUMERICAL MODELLING – SLOPE STABILITY

The modelling for the slope stability case was different, since the design was in accordance with the FHWA Design Manual and had been accepted with regard to the ultimate limit state. In this case the Queensland Department of Transport and Main Roads (TMR) were concerned as to what the deflections might be, probably because micropiles are relatively slender and therefore might be seen as flexible.

The input geometry with retaining wall, micropiles and traffic loading is shown in Figure 17, while Figure 18 shows the deformed mesh for the serviceability limit state.

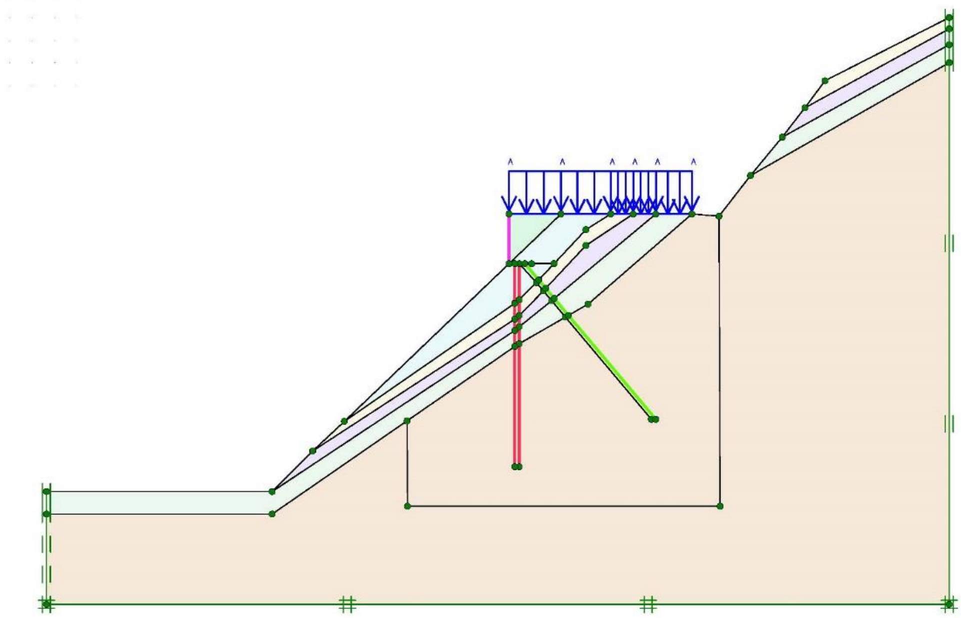

Figure 17. Micropiled A-frame structure input

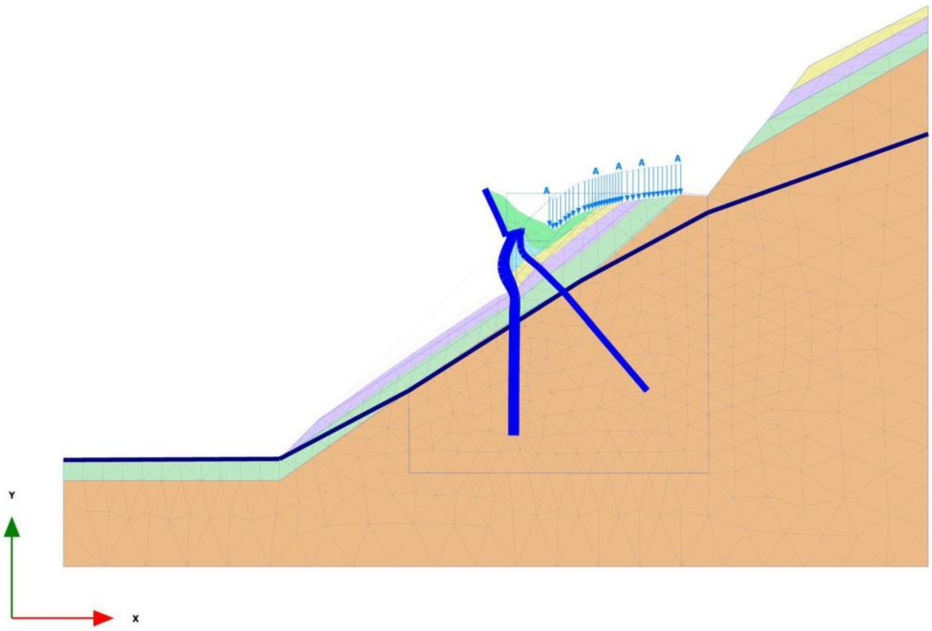

Figure 18. Deformed mesh at SLS

The A-frame structure means that bending moments and shear forces tend to be minimal, but this could depend on whether the structure is modelled with fixed or pinned connections into the headstock. Both arrangements were tested and found to show very little difference except at the top, as seen in Figure 19 to 20.

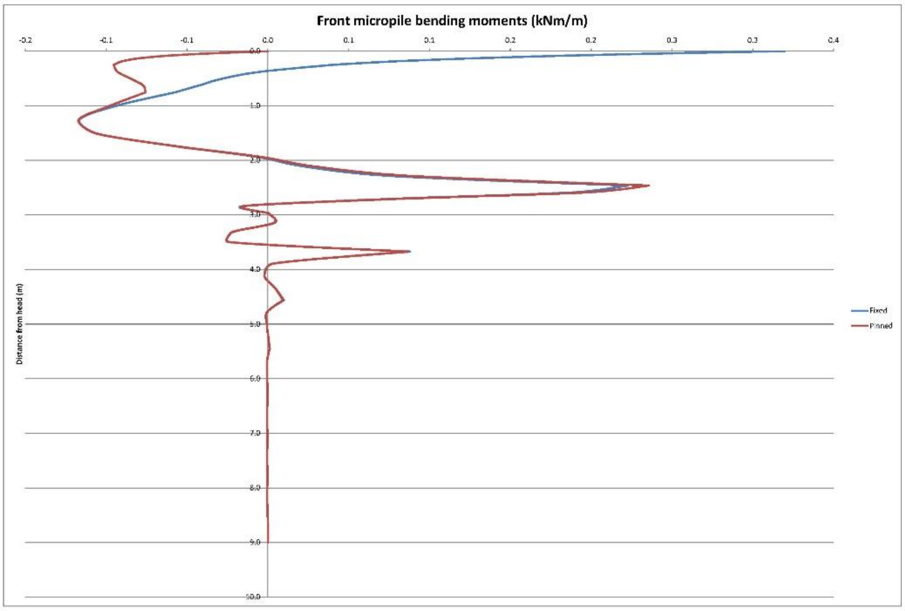

Figure 19. Front micropile bending moments at SLS for fixed and pinned conditions

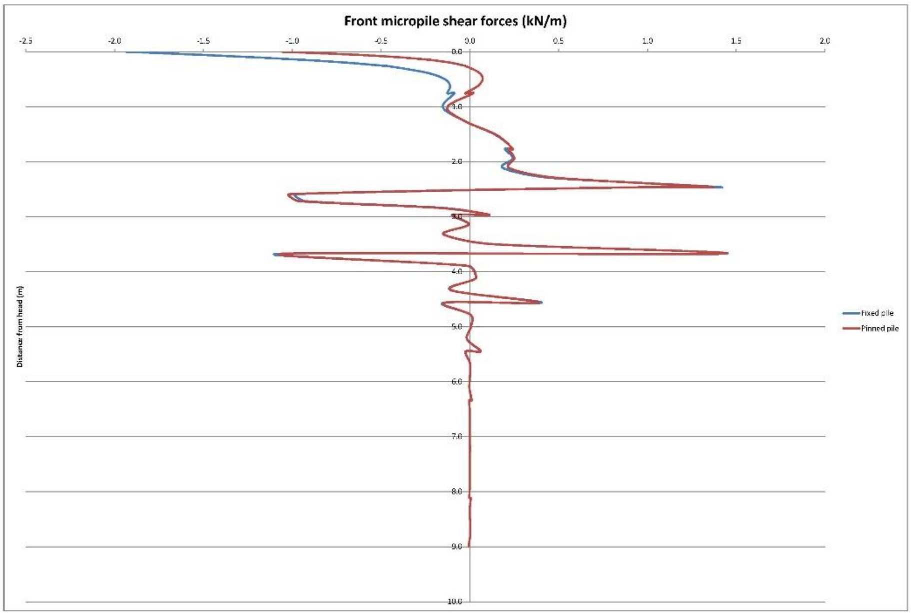

Figure 20. Front micropile shear forces at SLS for fixed and pinned conditions

Summaries are shown in Figures 21 to 23 for fixed end conditions.

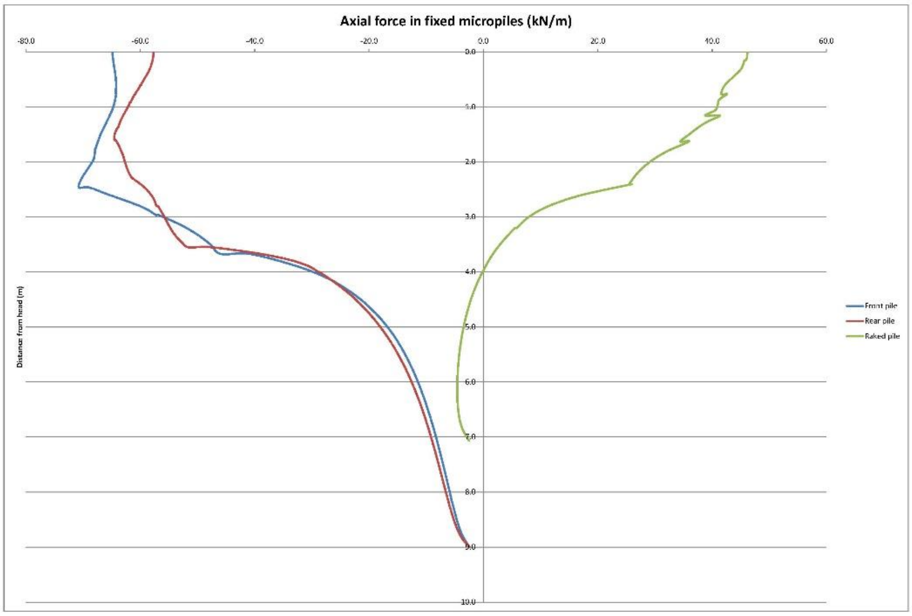

Figure 21. Axial forces in front and raking piles

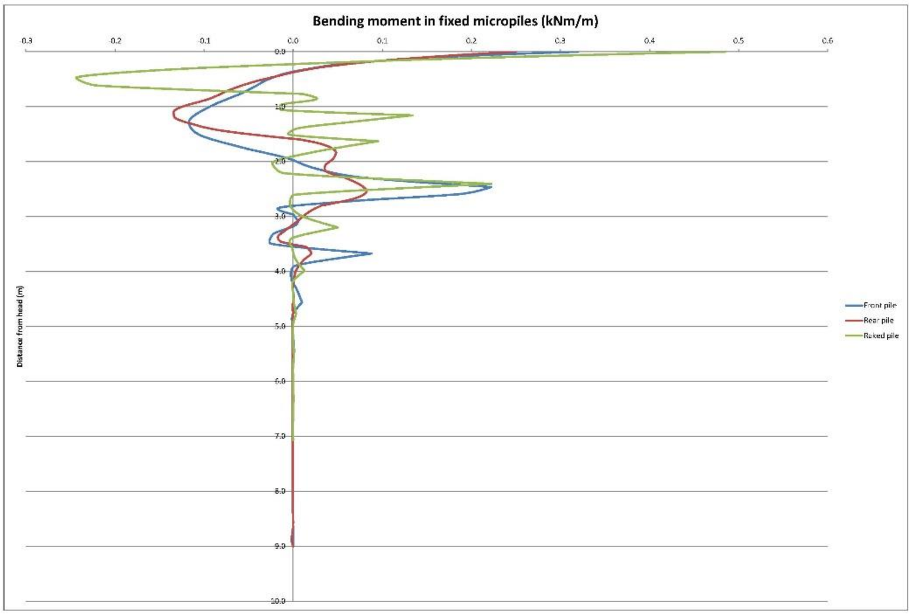

Figure 22. Bending moments in front and raking piles

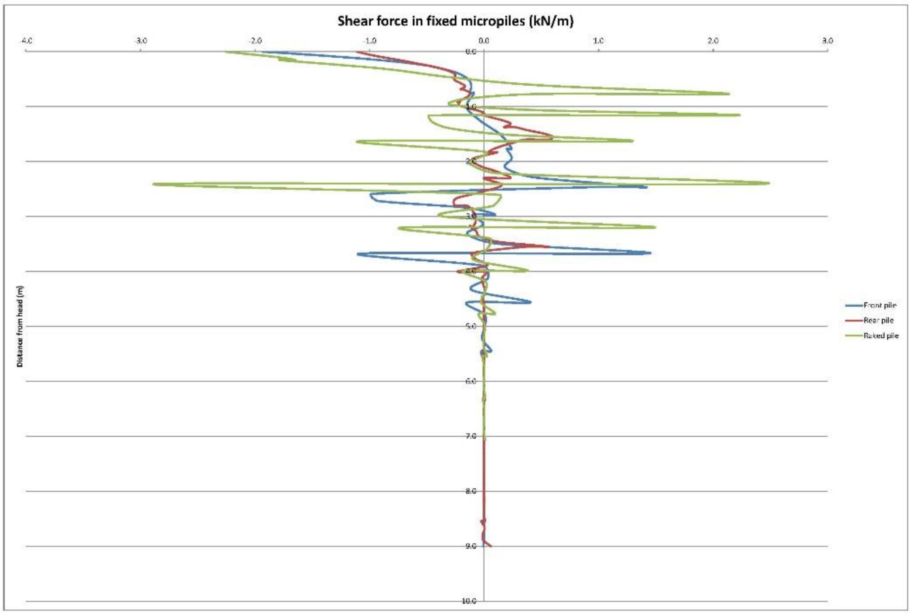

Figure 23. Shear forces in front and raking piles

All of these results were compatible with the design values from the FHWA method, which also gave TMR assurance that everything was in order. The predicted movements using unfactored soil properties and loads also showed very low values, with a maximum of less than 4 mm deflection of the pavement and negligible deflection of the structure, as seen in Figures 24 and 25 for the fixed and pinned conditions respectively.

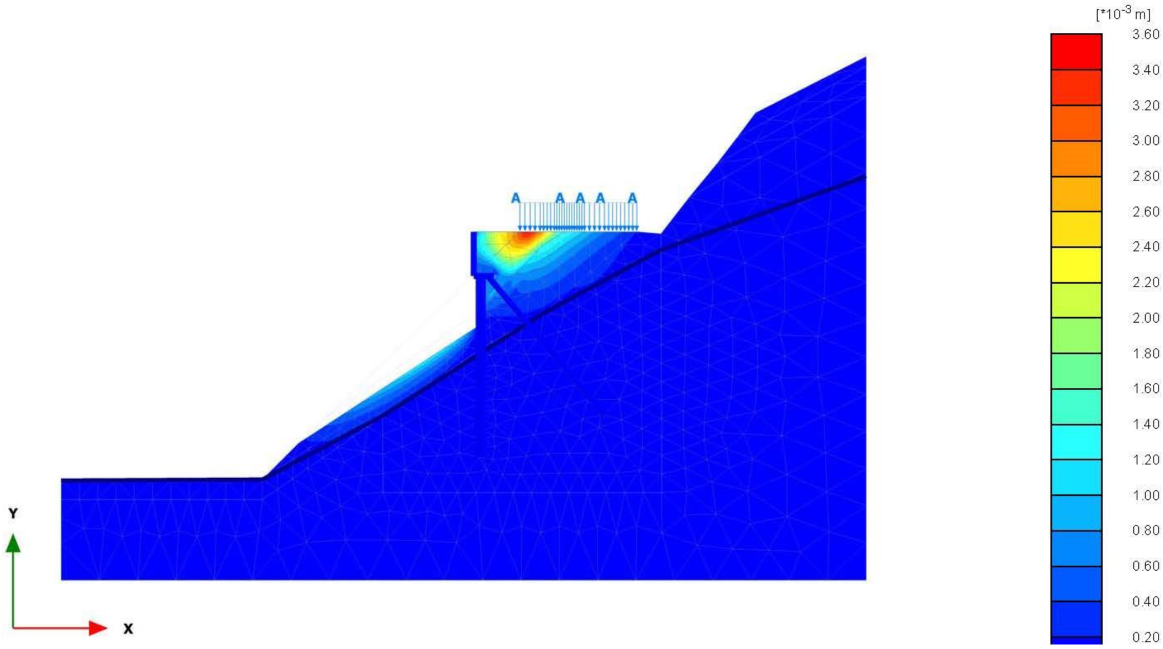

Figure 24. Final displacements with fixed micropile heads

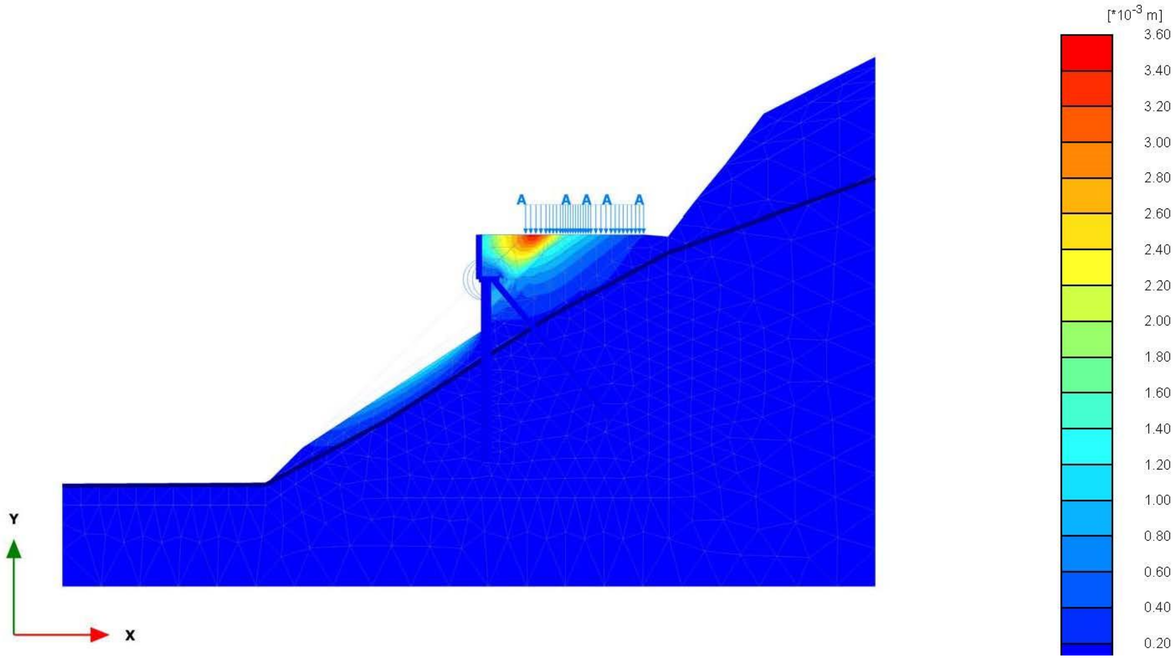

Figure 25. Final displacements with pinned micropile heads

It is noted that these look identical in the figures, but were actually 3.49 mm for the fixed heads and 3.52 mm for the pinned heads.

It does also need to be noted that PLAXIS is geotechnical software, carrying out geotechnical analysis. It can be used to look at ultimate geotechnical limit states, as in the factor of safety calculations shown in Figure 11, but the structural limit states need to be separately examined, and the forces such as bending moments and shear forces determined at SLS need to be factored up to give ULS values of load effect. AS 5100.3 suggests that a factor on output of 1.5 should be used, whereas CIRIA (2017) suggests a factor of 1.35.

CONCLUSIONS

Numerical modelling should not replace the FHWA method as the primary design tool, since there is a significant amount of invaluable experience which is built into the method. However it can be a useful additional tool, particularly with regard to predicting likely movement, and also with regard to providing an independent review of a design for checking and verification purposes.

The embedded beam functionality available in PLAXIS 2D is useful for modelling piles and soil nails, including micropiles, but extra care is needed when section properties change along the length of the micropile.

ACKNOWLEDGEMENTS

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