# MICROPILES FOR COMBINED DEEP SLIP AND STRUCTURAL LOADING

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## ABSTRACT

The use of grouted injected micropiles in an A-Frame configuration as a solution to slope stability problems has been well documented in case studies around the globe. Typically, they consist of a series of downslope and upslope micropiles installed through the failure plane with the pile heads terminating in a concrete capping beam. Despite its wide usage across the world, its use in Queensland, Australia is limited. The Queensland Department of Transport and Main Roads (TMR), with over 33,353 kilometres of road network under its jurisdiction, has traditionally continued to use established methods such as soil nails and soldier piles in managing slip repairs on their network. However, following severe weather events in 2011 to 2013which resulted in multiple slip failures, micropiles were first introduced as temporal stabilising measure.

Since their introduction in 2011, TMR has subsequently allowed the use of micropiles as a permanent stabilising measure on a trial basis for the Gatton Clifton Road and Kin Kin Road slip repairs. The approval for their use followed a rigorous design / design review processes. It was intended that the performance of the remediated sites, which were instrumented and are currently being monitored by TMR, will form the basis for the acceptance or otherwise of the method as permanent stabilising measure on TMR road network.

This paper examines the use of hollow bar micropiles which have been used in an A-Frame arrangement to permanently remediate two deep slip failures at Gatton Clifton Road and Kin Kin Road in Queensland, Australia. At the Kin Kin Road slip, the micopile structure in addition to providing the stabilising action for a deep slip failure, also catered for the applied forces from a 1.5m retaining wall which facilitated a widening of the road.

## 1.0 BACKGROUND

Several seasons of severe wet weather in South East Queensland, Australia between 2011 and 2013 resulted in a large number of slope stability issues to various degrees within the Queensland road network (Hinds, 2015; Ezeajugh et al 2016). Much of the instability was found in the rural network on dual lane roads vital for the local communities for both their economic and social importance.

A range of measures within TMR's tool box for stabilising such slips include re-grading the failed slope to a stable batter geometry, the use of rockfill stabilising berms, gabion wall retention, soil nailing, geosynthetic reinforcement and the use of large diameter bored/ CFA piles. The bored piles are often installed on the road shoulder with a series of 750mm to 900mm diameter reinforced concrete bored shafts at centres of up to three pile diameters with the pile heads

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terminating in a continuous concrete capping beam. Installation of these types of structures usually requires piling rigs weighing over 40 tonnes along with other associated machinery and equipment that requires a full road closure during construction.

The full closure of these roads in rural areas is often controversial and extremely inconvenient for the local community. Quite often, there is also the significant expense of carrying out temporary works to support the large piling rigs and supporting equipment.

In 2011, several slips were identified in the hinterland areas of Peachester and Maleny in South East Queensland. Temporary emergency works were required to support a number of slips with TMR seeking new and innovative solutions that were quick to install with minimal impact on the local communities. As a result, several sites were stabilised using micropile A-Frame structures with equipment that could work within the confines of a single lane of traffic such that the road could remain open.

Since 2011, several other sites have been stabilised using micropile A-Frame structures for both permanent and temporary solutions following the design method published in the *FHWA NHI-05-039 Micropile Design and Construction* manual (FHWA NHI-05-039 December 2005). Where the micropile A-Frame arrangements were used as permanent stabilising measures, their approval for use were on the basis of a trial. As such they were instrumented and are currently being monitored by TMR. The two candidate sites where permanent A-Frame micropile stabilising structures were installed were the Gatton Clifton Road and Kin Kin Road slips. At the Kin Kin Road slip, in addition to stabilising the deep slip failure, the A-Frame micropiles were required to support the loads from a 1.5m high insitu concrete retaining wall cast directly onto the A-Frame capping beam.

The retaining wall on top of the capping beam introduced additional axial, shear and bending loads which are not covered by the FHWA design method. These applied forces required additional analysis highlighting some limitations in the assumptions made within the FHWA method particularly in relation to the function and geometry of the capping beam.

This paper will compare the design of two permanent A-Frame micropile structures and discuss various limitations of the FHWA design method. Particular focus will be on the design of the Kin Kin Road A-Frame structure and the additional steps and assumptions that had to be addressed due to the applied forces from the retaining wall stem above the capping beam.

## 2.0 DISCUSSION OF CURRENT A-FRAME DESIGN PRACTICES

The design approach for micropile A-Frame structures is well documented by the FHWA in chapter 6 of their micropile design guide released in December 2005. The design process follows twelve steps which involve modelling the slope, determining the additional shear resistance required to achieve an acceptable factor of safety, configuring the A Frame to provide this resistance e and determining the axial loads within the piles.

The resulting micropile A-Frame structure generally consists of two or more rows of downslope and upslope raking micropiles extending for a sufficient depth below the slip plane and terminating at ground level within a reinforced concrete capping beam. Figure 1 is an illustration from the FHWA design manual depicting the general cross section of a micropile A-Frame structure.

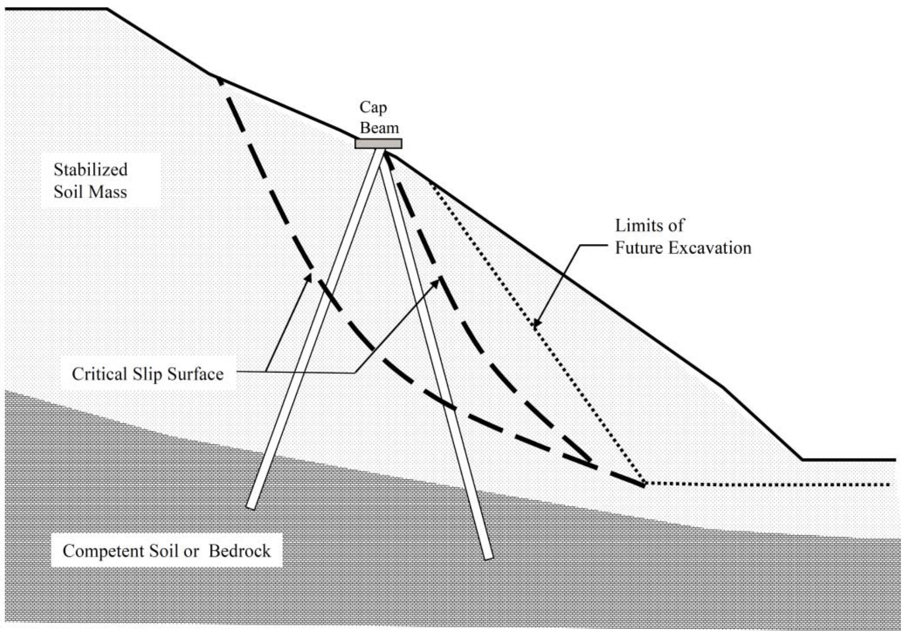


Figure # 1 FHWA depiction of general cross section of an A Frame micropile structure

The FHWA design procedure focuses on solving the geotechnical aspects of the slope stability problem. Although it gives some guidance on the potential size of the capping beam, its scope is limited to determining the required shear resistance to achieve a suitable factor of safety for a deep slip and leaves the structural design of the capping beam mostly up to the structural designer.

The assumption is that the maximum axial load is at the critical slip interface and the "axial loads transferred to the ground surface should be small" (FHWA NHI-05-039 December 2005). As such, the FHWA approach for the design of the capping beam only addresses differential settlements along the length of the capping beam from variations in the soil and for any anchor forces in the event that ground anchors are required.

As a consequence, once a solution for slope stability has been determined using the steps in the FHWA process, additional steps are required for any external applied forces to the structure.

It is interesting to note that section 6.7.10 of *FHWA NHI-05-039 December 2005* suggests a typical capping beam is in the order of 2m wide and 1m deep. However, the document is silent on the requirement for the A-Frame structure to consider either the self-weight of the capping beam to prevent surcharging the slope nor is there mention of potential lateral earth forces acting on the capping beam. The function of the capping beam in the FHWA method is one of pile head fixity only.

As such, it may be prudent on some slopes, for the designer to check for the load case during construction when the concrete has not yet cured and is unsupported by the micropiles to ensure it does not cause localised instability. Particularly when using a capping beam in the order of magnitude suggested by FHWA.

A secondary check for completeness may also include the potential shear and overturning moment induced into the piles from the earth pressure acting on the upslope vertical edge particularly on relatively steep slopes or when external surcharge load from sources such as vehicles are relatively close to the capping beam.

For the majority of structures, the effects of earth pressure forces on the capping beam or the self-weight of the capping beam itself are small and insignificant, however, the designer should be mindful of the limitations of the FHWA design method and use their engineering judgement as to when external forces interacting with the capping beam or induced by the capping beam require additional design steps to be considered.

## 3.0 DESIGN CONSIDERATIONS FOR APPLIED FORCES TO THE CAPPING BEAM

The addition of external applied loads to the capping beam requires consideration of the geometry of the micropile to pile cap connection and pile layout to determine how the external forces are transferred into the micropiles and how these forces impact on the maximum load within each of the rows of micropiles.

### 3.1 Gatton Clifton Road

In late 2013 a site near the town of Gatton in the Darling Downs region west of Brisbane was rectified with an A-Frame positioned on the edge of the road near the crest of the slope. Figure 2 illustrates the location of the Gatton A-Frame directly beneath the metal crash barrier.

Figure 2 – Completed A Frame at Gatton

The FHWA design method was used for the design of the Gatton Clifton Road project. Due to the potential for localised erosion and minor downslope slips below the A-Frame, PLAXIS 2D was used to check for deflections for load cases involving the temporary loss of passive resistance below the structure resulting from future erosion or shallow slips within the downslope. Limits were set by the Queensland Transport and Main Roads Department (TMR) on the maximum allowable deflection for the structure for serviceability.

The PLAXIS 2D modelling introduced an additional step in the design process. The A-Frame was divided up into six different design sections with varying micropile layouts to adjust the stiffness of the structure in order to meet the deflection limits set by TMR. Figure 3 below illustrates the PLAXIS 2D model used on one of the critical design sections with a maximum calculated deflection of 132mm.

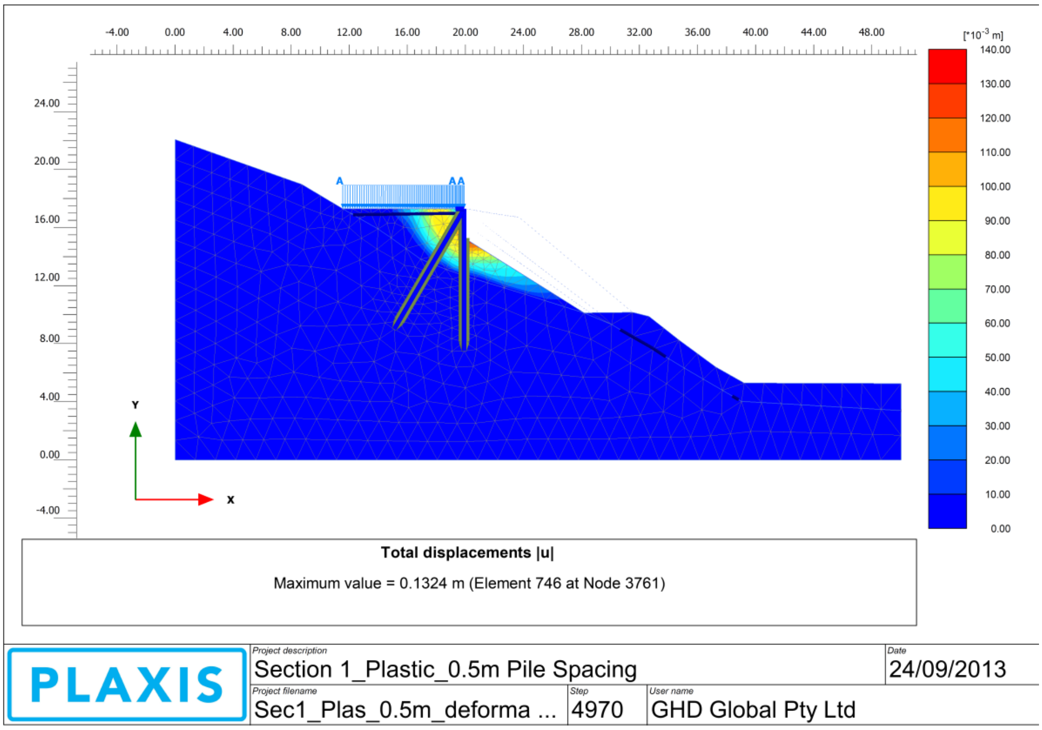


Figure 3 – Gatton Clifton Road Plaxis 2D modelling of critical section

The PLAXIS 2D modelling raised some interesting questions during the design process in relation to the geometry of the micropiles and their interaction with the capping beam which are not addressed in the FHWA design process.

The FHWA design process simply considers the capping beam as a way of pinning the pile heads together to form the A-Frame structure. Although this is a suitable assumption for the basis of the overall A-Frame design, by ignoring the contribution of the capping beam connection, the calculated deflections in the PLAXIS 2D modelling resulted in a much higher maximum deflection of 159mm as compared to the 132mm at the ultimate limit state in the final model which included some stiffness from the capping beam.

This difference in deflection for the two models was not critical for the Gatton Clifton Road project. The A-Frame structure was in place to solve the deep slip condition with the potential calculated deflections only being possible in the temporary condition caused by potential future erosion or slips in the downslope prior to remediation works being undertaken. However, it did highlight the importance of correctly modelling the capping beam and micropile connection on projects where load transference into the micropiles from applied loads on the capping beam is a permanent structural requirement. This will be discussed further in the following section relating to the A-Frame structure on Kin Kin Road.

### 3.2 Kin Kin Road

In early 2015, a site on Kin Kin Road in the Sunshine Coast Hinterland north of Brisbane, Australia received funding from the Queensland Government to rectify a potential deep slip which had developed tension cracks near the centreline of Kin Kin Road adjacent a local creek.

To reduce the risk of future downslope slips, the Kin Kin A-Frame was positioned beyond the crest of the slope at a level 2.5m below Kin Kin Road. As a means of widening the road corridor, a 1.5m high cast insitu reinforced concrete retaining wall was integrated into the capping beam. Figure 4 illustrates a typical cross section through the Kin Kin structure detailing the position of the A-Frame structure with the integrated concrete retaining wall.

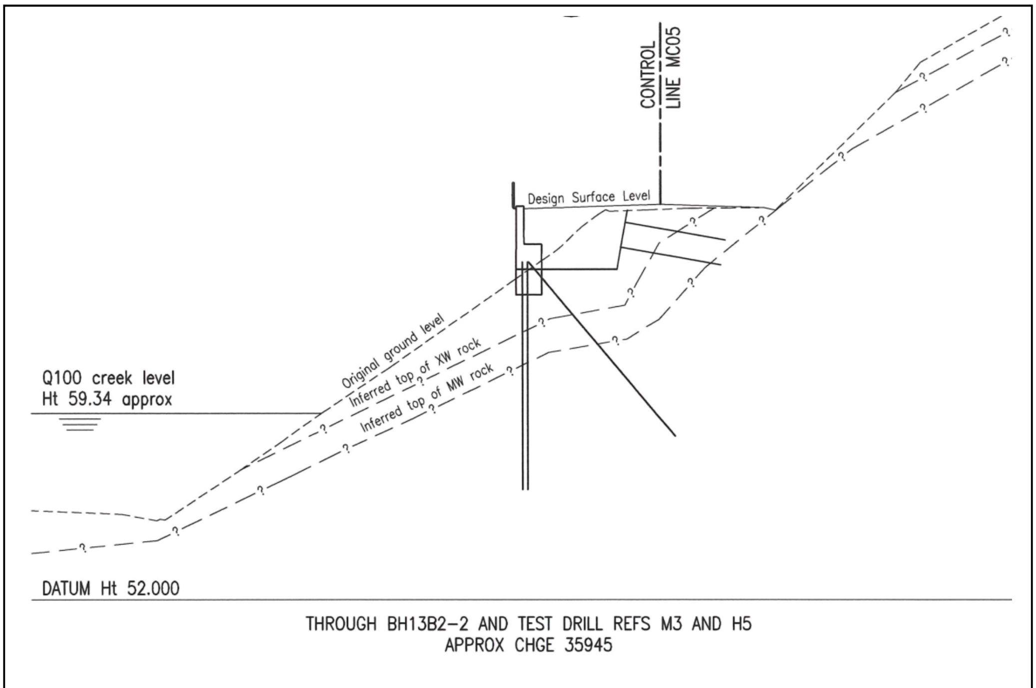


Figure 3 – Location of downslope Kin Kin A-Frame

The addition of the retaining wall to the capping beam required a unique design approach involving the following procedures;

1. **Step # 1 – Designing the A-Frame for Slope Stability**. This step involved the design of the A-Frame structure to stabilise the deep slip in accordance with the FHWA design method.
2. **Step # 2 – Designing the A-Frame piles to carry the applied load from the 1.5m high retaining wall.** This involved taking the horizontal shear at the base of the wall due to lateral earth pressure and the resulting overturning moment about the toe and including these forces in the A Frame micropile design.
3. **Step # 3 – Overall deflection check.** Using PLAXIS 2D to ensure the serviceability deflections were within required design limits.
4. **Step # 4 – Check pile forces for combined axial and bending.** Using PLAXIS 2D, pile forces were determined to enable a check for combined axial and bending for the serviceability load case.

Each of these steps are discussed in further detail below.

***Step # 1 – Designing the A-Frame for Slope Stability***

Following the procedures in the FHWA design procedure, it was determined that the slip circle failure plane was at a depth of 3.5m. This soil consisted of a layer of medium dense gravelly sand fill with lenses of firm to stiff sandy silt overlaying residual soil consisting of medium dense to dense sandy gravelly silt.

Ischebeck Titan 40/20 hollow bar micropiles with 90mm drill bits were selected for use in the project. Using proprietary effective skin friction values for the material above the slip, an ultimate pile load ($P_{ult}$) was calculated to be 164kN.

Slope W modelling was undertaken to determine the required shear resistance per lineal metre run of capping beam to provide an acceptable factor of safety, for both the extreme ground water condition and the normal ground water condition (1.3 and 1.5 respectively). It was determined that a shear contribution per pile of 105kN/lin.m of capping beam was required.

The battered pile group was modelled with the rear (upslope) micropiles raking back into the slope at 40 degrees from vertical while the front (downslope) row of micropiles remained vertical due to the fairly steep downslope batter. The micropile group was analysed for its shear capacity ($H_{ult\ pair}$ = 125kN) which resulted in a spacing of 600mm per pile group with a factor of safety for soil flow of 3.2.

**Step # 2 – Designing the A-Frame piles to carry the applied load from the 1.5m high retaining wall.**

The applied loads on the capping beam of the A-Frame from the 1.5m high retaining wall were calculated to produce the following design actions;

$M^*_{ot}$ = 59kNm/m

$V^*_{sliding}$ = 71kN/m

To resolve these forces into the micropiles, the distance between point of entry of the hollow bar micropiles into the underside of the capping beam was deemed to be the moment arm for the purposes of calculating the reactions required to resist the applied moment.

As such, a distance of 500mm between the front and rear rows of micropiles resulted in pile forces of 71kN per pile of compression for the front row of micropiles and 137kN per pile of tension for the rear row of micropiles.

To determine the maximum pile force, an approach was taken which assumed the full structural load is transferred through the 3.5m deep slip plane to be developed within the bonded length below the slip plane. As such, a maximum pile force was calculated as 301kN per micropile by adding the maximum applied tensile force (137kN) to the $P_{ult}$ value of 164kN per pile from the FHWA modelling.

It could be argued that since the micropiles were not de-bonded through the upper 3.5m of soil that some pile load would be distributed above the slip plane. However, an assumption was made in the event of an active slip, the full applied load from the retaining wall above the capping beam would need to be developed within the bond zone below the slip plane.

The combination of load for both geotechnical and structural forces within the pile group using static analysis is believed to be much more complex than simply adding the two maximum forces together. This is because at the point of maximum geotechnical load, the ultimate geotechnical force in the pile requires the soil to move along the length of the pile. As it does so, the forces acting on the retaining wall are changing as the structure begins to deflect.

As such, further modelling was conducted to check how the structure would deflect through finite element modelling (FEM) using PLAXIS 2D. FEM was also used to check the pile forces during the serviceability load condition.

***Step # 3 – Overall deflection check***

The first of two FEM checks was to determine serviceability and ultimate deflections of the A-Frame and retaining wall. Similar to the Gatton Clifton Road project, the A-Frame structure was required to meet deflection criteria for the ultimate load case which included temporary loss of passive pressure on the downslope from erosion or future downslope slip.

Initially, the structure was modelled assuming;

a) A Mohr-Coulomb model for soil and rocks
b) The micropiles and capping beam were modelled as plate elements
c) The micropile intercept was coincident at a node centred on the underside of the capping beam ignoring the transverse spacing of the front and rear rows of micropiles within the capping beam.
d) Median strength values for soil parameters were used
e) Surcharge loads were applied directly behind the retaining wall assuming no road shoulder

The PLAXIS 2D model is illustrated in Figure 4 below.

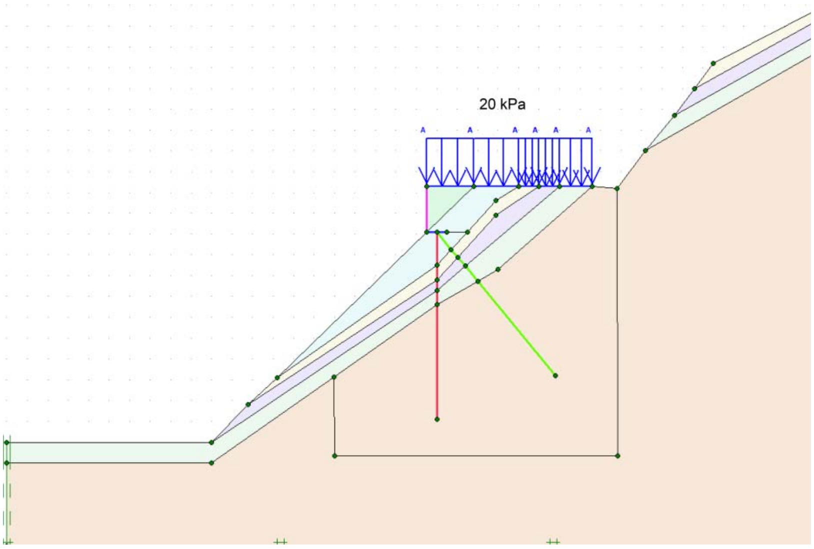


Figure 4 : Initial PLAXIS 2D model

The results of this initial model are summarised in Table 1 below.

| Loading Stage | Displacements (mm) |
|---|---|
| Place new fill and construct pavement (Serviceability condition) | 17 |
| Remove partial soil (Fill and Residual) on the passive side of the micropile wall | 21 |
| Apply traffic loading equivalent to 20kPa surcharge without removing passive support | 34 |
| Apply traffic loading equivalent to 20kPa surcharge after removing passive support | 52 |
| Apply Q100 flood level and groundwater on ground surface (Ultimate condition) | 55 |

Table 1 : Initial PLAXIS 2D results for overall deflection of the A-Frame structure

Analysis of the results of the initial modelling led to a refined approach being undertaken which took into account the transverse spacing of the front and rear rows of micropiles within the capping beam. This lead to a much stiffer structure and the deflections reduced accordingly.

The refined model can be seen in Figure 5 below.

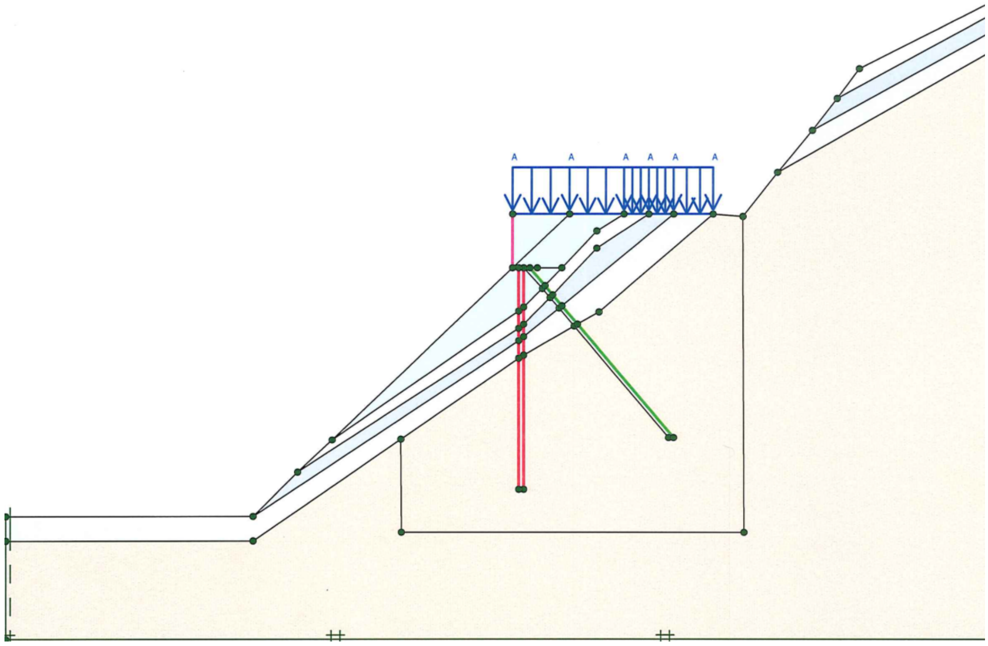


Figure 5 : Refined PLAXIS 2D model with transverse micropile spacing

With the transverse micropile spacing included in the model, the ultimate condition under full Q100 flood level and high water table reduced from 55mm in the initial model to 3.5mm in the refined model.

The deformed PLAXIS 2D model is contained in Figure 6 and demonstrates the nature of the deflection of the structure in the ultimate load case where passive resistance is potentially lost directly in front of the wall.

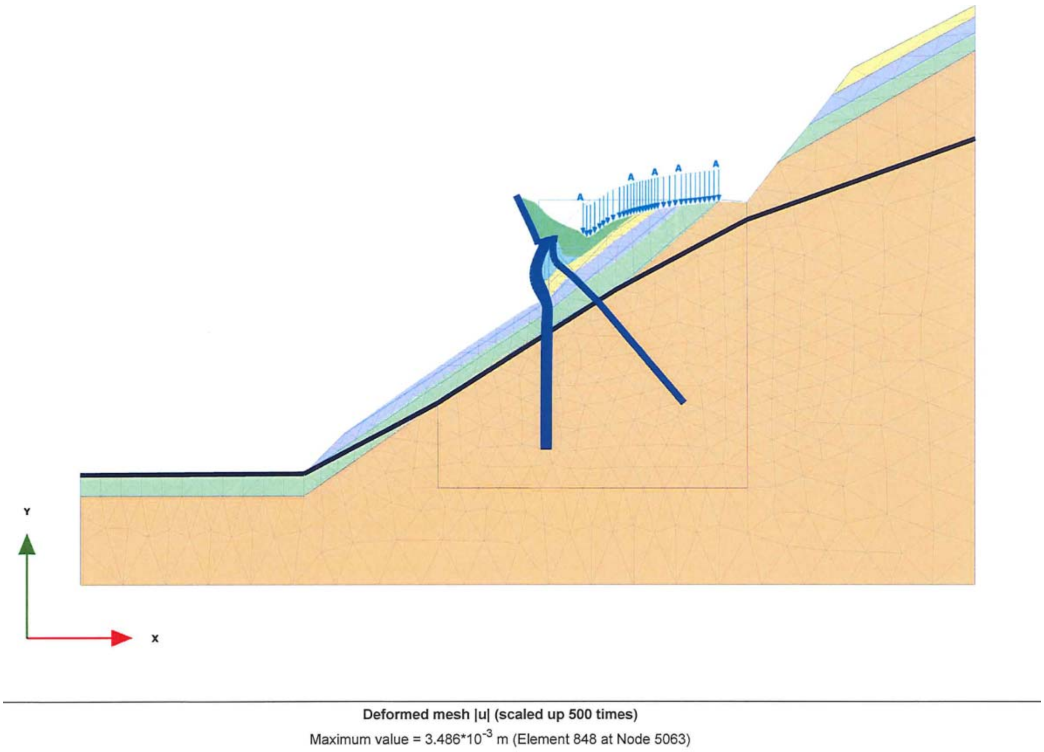


Figure 6 : Refined PLAXIS 2D model illustrating deflections in the ultimate load condition

**Step # 4 – Check pile forces for combined axial and bending.**

The model in Figure 6 indicated that some deflection would occur in the front vertical row of micropiles between the underside of the capping beam and the entry point into the ground.

Although the deflection of the structure was well within limits required by the Department of Transport and Main Roads Queensland, the designers were interested in checking the micropiles for combined axial and bending capacity due to the relative slender nature of the micropile elements.

Although the micropiles had been designed for the maximum combined load of $P_{ult}$ from the FHWA geotechnical analysis along with the ultimate applied load from the retaining wall, this was a maximum tensile force calculated from the upslope raking pile applied for simplicity to all of the micropiles.

For the combined axial and bending check , the refined PLAXIS 2D model was used to determine the axial and bending forces within the individual micropiles. Due to the nature of the PLAXIS 2D being a geotechnical modelling tool, the model was limited to providing serviceability forces only due to the inability to include the ULS applied structural loads combined with the un-factored soil properties. A global factor of 2 was applied to relate ULS to SLS forces

To provide the stiffness required for limiting deflection of the A-Frame, a micropile layout including a single upslope row of raking micropiles combined with a double downslope row of vertical micropiles was selected. Figure 7 below demonstrates the micropile layout selected in the final design with the raked piles being on the upslope and the vertical piles being downslope.

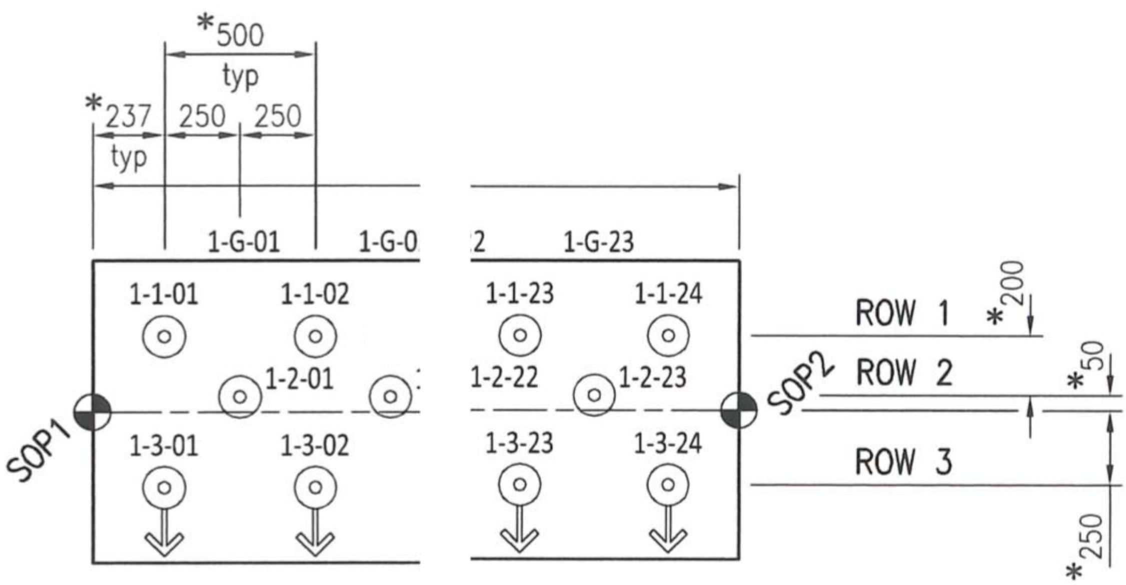


Figure 7 : Final micropile layout

Due to the capping beam having a high relative stiffness compared to the micropiles with fixity of the micropiles potentially being neither fully pinned or fully fixed, the fixity of the micropiles was modelled for both a pinned and fixed head condition to compare the effect on pile forces. The results are illustrated graphically below in Figures 8 and 9.

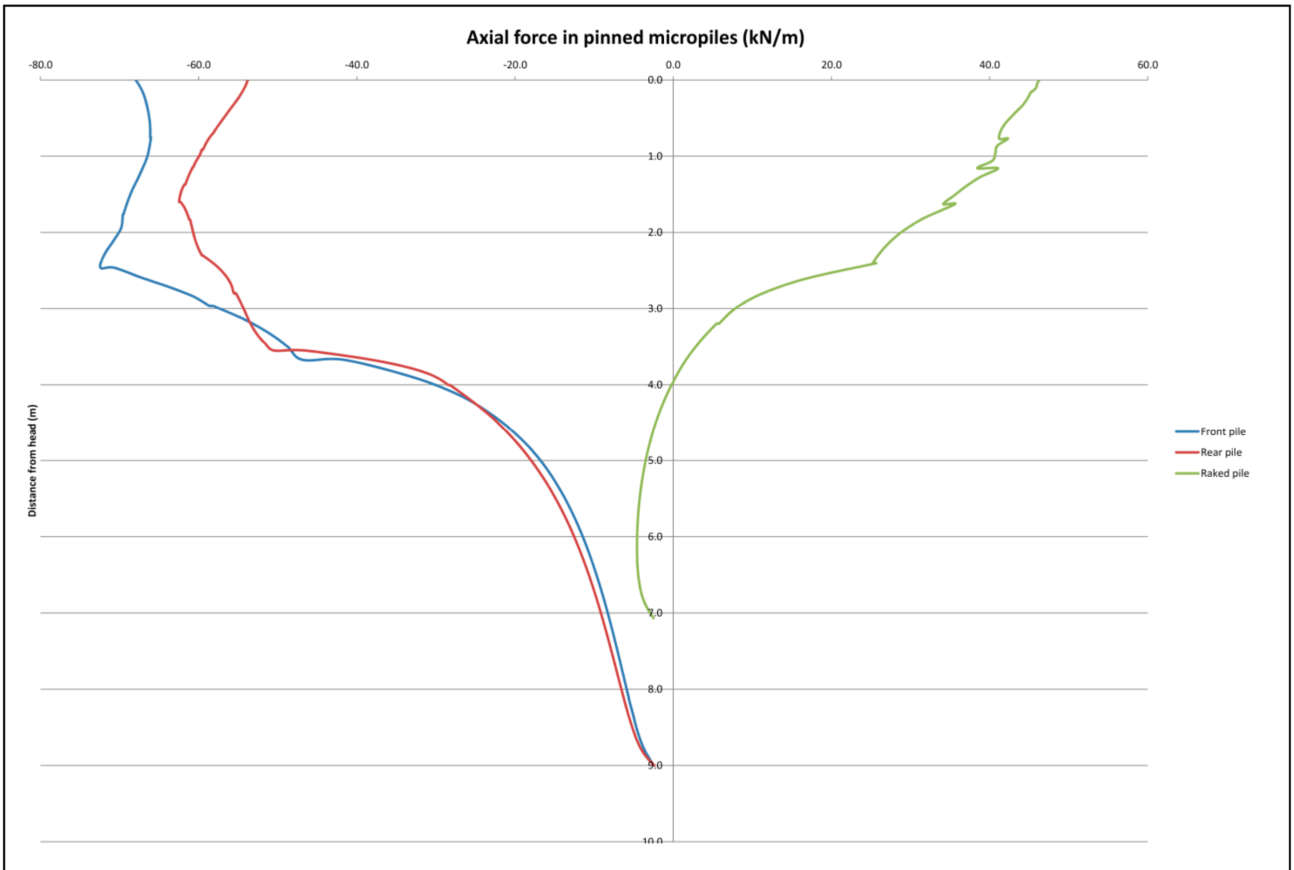


Figure 8 : Axial force for pinned head condition

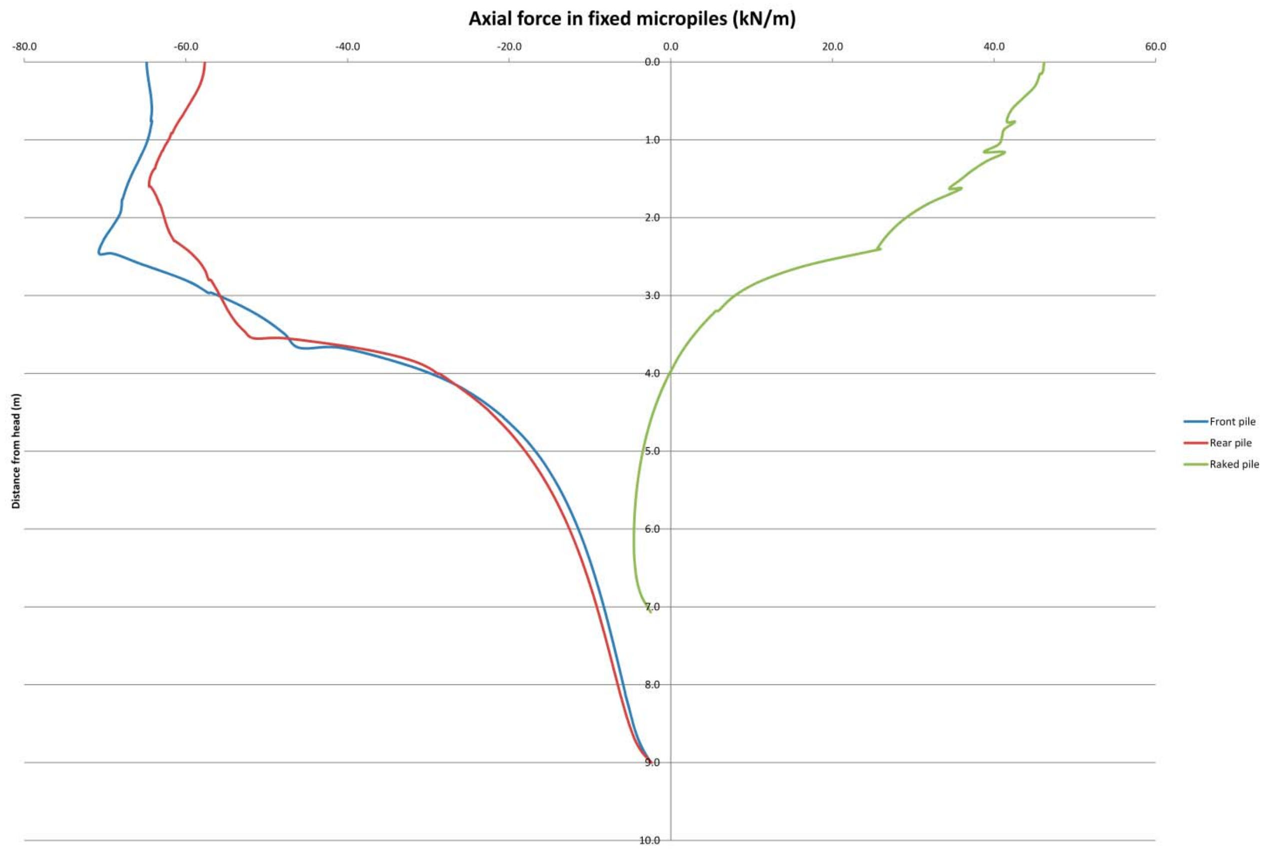


Figure 9 : Axial force for fixed head condition

From the analysis, the unfactored axial force in the micropiles was similar for both fixed and pinned head conditions with the pinned head condition producing a slightly higher force per micropile.

A factor of 2 was then used to factor the force up for the limit state condition used in the combined axial and bending check in accordance with Australian Standard "*AS4100 – 1998 Steel Structures*".

The results of the combined axial and bending checks for both the tension and compression piles are contained in Tables 2 and 3. It should be noted that a reduction factor of 0.5 was used on the bending capacity from the LPILE analysis for the semi-exposed outer vertical downslope pile. This is due to the potential loss of restraint for the outer piles during the worst case scenario for potential future downslope slip or erosion of soils providing passive resistance.

| Raked Pile (Tension) | M* (kNm) | N* (kN) | $\phi$Ms (kNm) | $\phi$Ns (kN) | $\phi$Mrx [1] (kNm) | Result |
|---|---|---|---|---|---|---|
| 40 / 20 bar | 0.49 | 47 | 2.9 | 382 | 2.5 | OK |

Table 2 : Combined axial and bending results of the micropile in tension

| Vertical Pile | M* (kNm) | N* (kN) | $\phi$Ms (kNm) | eff length (mm- limit) | $\phi$Nc (kN) | $\phi$Mi [1] (kNm) | |
|---|---|---|---|---|---|---|---|
| 40 / 20 bar | 0.32 | 72.5 | 2.9 [2] | 1385 | 81 | 0.32 | OK |
| 40 / 20 + grout col | 0.32 | 72.5 | 3.0 [3] | 2770 | 81 | 0.32 | OK |

Table 3 : Combined axial and bending results for the micropile in compression

1 Combined axial compression and bending AS4100 cl 8.4.2
2 METHOD 2 as section 11.4.1
3 0.5 x LPILE value

The micropiles were found to be OK for combined axial and bending forces. This was the final check in the design process.

## 4.0 FINAL REMARKS

Although providing a solution for the Kin Kin Road slip remediation, the process of designing the A-Frame micropile structure with an integral concrete retaining wall proved to be challenging due to the interaction between the structural and geotechnical components.

The FHWA design method for the A-Frame considers the shear resistance required by the structure and does not require the capping beam to do any more than simply fix the pile heads together. The potential for applied forces to act upon the capping beam and be resisted by the micropiles is not covered as they are not required for a simple slope stability solution.

The PLAXIS 2D design checks were also beneficial for checking the potential serviceability deflections of the structure but there were limitations in the benefits of PLAXIS 2D when it came to the structural design when incorporating the applied static loads. The two processes didn't quite work together as well as they could due to complexities when comparing the static calculations of the structural design to the PLAXIS 2D geotechnical model which allows the structure to deflect.

The PLAXIS 2D design checks were also beneficial for checking the potential serviceability deflections of the structure but there are inherent limitations using a geotechnical package to determine structural forces.

The combination of the static structural analysis and the forces obtained from the geotechnical model proved to be more complex than originally thought which may have resulted in the final design being somewhat conservative. That said, inclinometers within the structure will be used for further analysis and refinement which should result in a more refined approach for similar structures in the future.

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