

DESIGN OF SLOPE STABILIZING PILES FOR REINFORCED SOIL WALLS ON HUNTER EXPRESSWAY

Henry Zhang¹, Simon Yau², Robert Kingsland³, David Och⁴

¹Principal Geotechnical Engineer, Parsons Brinckerhoff, GPO Box 5394, Sydney, NSW 2001; PH +61(0)2 92725100;

email: hezhang@pb.com.au

²Technical Director, Hyder Consulting (Formerly Roads and Maritime Services), North Sydney NSW 2060; PH +61(0)2 89079148

³Technical Executive, Parsons Brinckerhoff, GPO Box 5394, Sydney, NSW 2001; PH +61(0)2 92725100

⁴Principal Engineering Geologist, Parsons Brinckerhoff, GPO Box 5394, Sydney, NSW 2001; PH +61(0)2 92725100

ABSTRACT

The Hunter Expressway will provide a 40 km long four-lane divided carriageway motorway between the F3 Interchange at Newcastle and the New England Highway at Branxton, New South Wales Australia. The project is due to be opened by the end of 2013. The Hunter Expressway Alliance (HEA), comprising Roads and Maritime Services (RMS), Thiess Pty Ltd, Parsons Brinckerhoff and Hyder Consulting, is responsible for the design and construction of the 13 km eastern section of new freeway and local road adjustments. There are 28 bridges and major culvert structures and 29 Reinforced Soil Walls (RSWs). This paper discusses the design challenges faced by the RSW designers and the innovative engineering solution developed for RW17, a 120 m long RSW up to 10m in height on sloping ground with foundations containing bands of low strength tuffaceous claystone. To achieve the minimum design factor of safety (FOS) of 1.35 for the overall slope stability of the RSW as stipulated in RMS Specification R57, three rows of 450 mm/750 mm diameter and one row of 1500 mm diameter bored piles were designed and adopted at various selected sections along the 120 m long slope. Both the limit equilibrium program Slope/W and the finite element program PLAXIS were used to assess the FOS for the global stability of the RSW, ground movements during and after RSW construction and forces in the piles. Two inclinometers were installed to monitor the field lateral ground movements during and after construction to verify the design assumptions. This paper describes the challenging ground conditions, the development of the stabilising pile design, the analytical models used and the results of the construction phase monitoring of the completed RSW.

Keywords: slope stabilizing piles, reinforced soil wall (RSW), tuffaceous claystone, instrumentation

1 INTRODUCTION

The Hunter Expressway is a 40-km long four-lane freeway that will form a new east-west link between Newcastle and the Lower Hunter. It is the largest single road project to be built in the Hunter Region and is scheduled for completion in late 2013. The \$1.7 billion Hunter Expressway is jointly funded by the Australian and the New South Wales (NSW) Governments. The new expressway will reduce the travel time between Newcastle and the Hunter Region by 28 minutes and relieve traffic congestion between Newcastle and the towns of Thornton, Maitland and Rutherford.

The project is being constructed under two contracts, with the western 27 kilometre section from Branxton to Kurri Kurri being built under a design and construct contract and the 13 kilometre eastern section between Kurri Kurri and the F3 Freeway (Figure 1) being built by the Hunter Expressway Alliance (HEA), which comprises Roads and Maritime Services (formerly RTA), Thiess Pty Ltd, Parsons Brinckerhoff and Hyder Consulting. This section of the expressway traverses a floodplain and rugged terrain via a series of bridges, tall viaducts, and extensive earthworks. Former underground coal mining activities and poor ground conditions present exceptional engineering and construction challenges to the designers. Aryal et al. (2013) discusses the unique design and construction challenges associated with earthworks and mine subsidence for the project.

Other design and construction challenges include soft ground improvement for embankments at Buchanan Swamp (Zhang et al., 2012) and bridge foundations affected by mine subsidence. There are 28 new bridges and culverts, including three viaducts up to 787 m long and 47 m high above the gully floor through the Sugarloaf

Range. Eight of the bridges are subject to potential mine subsidence due to presence of old mine workings underneath the project site. Zhang and Choi (2012) discussed the foundation design challenges and innovative foundation design solutions for one of the viaducts.

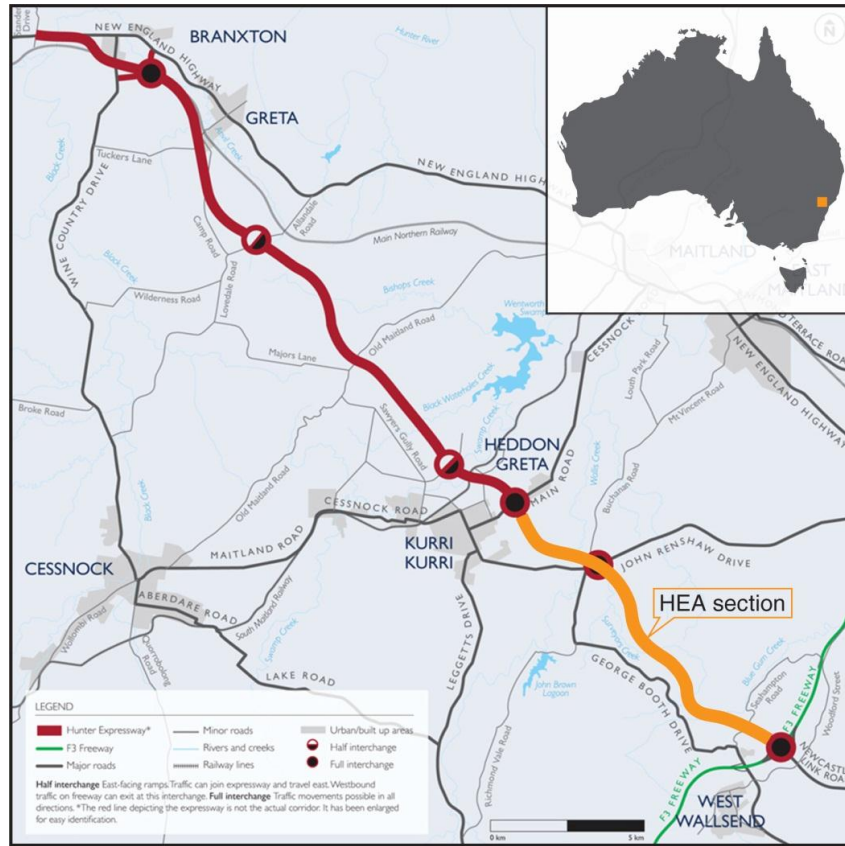


Figure 1: Project location

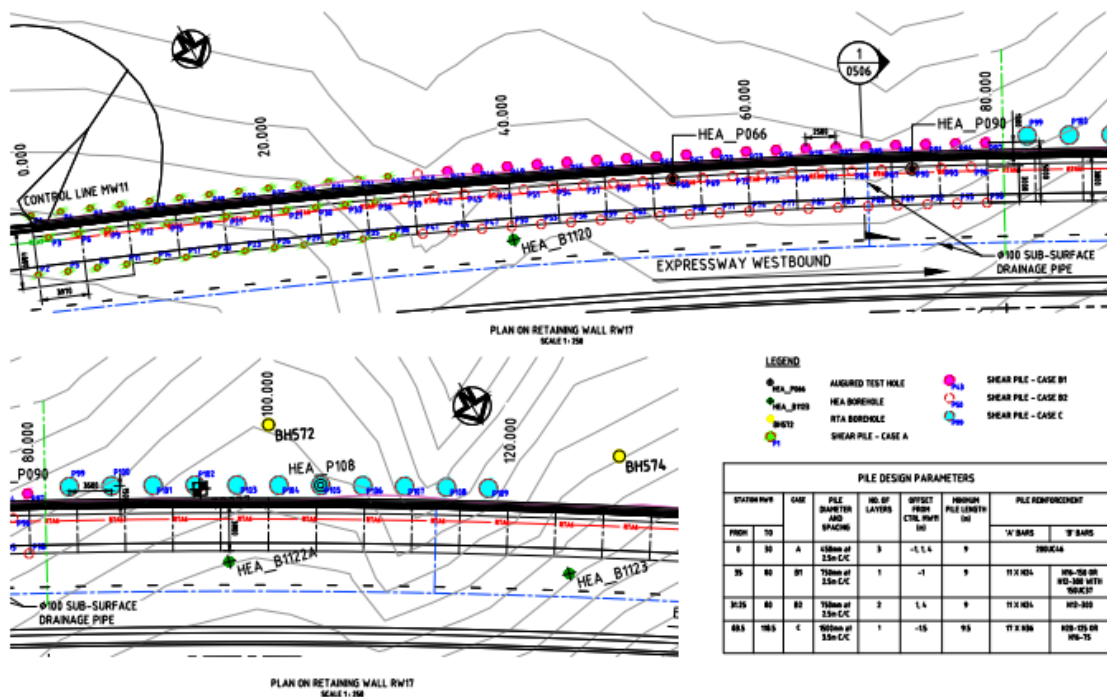


Figure 2: RW17 site layout plan

This paper discusses the design challenges faced by the RSW designers and presents the innovative engineering solution developed for RW17, which is located from Chainage Ch3225 to Ch3425 near Cut 9 in the Sugarloaf area. Refer to Figure 2 for the RW17 site layout plan.

2 SUBSURFACE CONDITIONS

Through the Sugarloaf Ranges the expressway route traverses the upper formations of the Newcastle Coal Measures (NCM), namely the upper part of the Adamstown Subgroup (Australasian Coal underlain by the Charlestown Conglomerate Member) and basal part of the Boolaroo Subgroup (Mount Hutton Formation) that are rocks of Late Permian Age (Hawley et al. 1994, Hawley and Brunton, 1995). The upper part of the NCM are sequences of raised mires, coarse-grained fluvial, siliciclastic sediments (Lindsay and Herbert 2002) with distinct beds of finer-grained (claystone) pyroclastic ash deposits all deposited in the Sydney Basin. This upper formation comprises sandstones, siltstones, conglomerates, tuffs and claystones; that are dominated by tuffaceous, coal and carbonaceous sedimentary rocks. The tuffaceous claystones are low strength rocks that are particularly susceptible to weathering and are highly reactive. After the construction phase test pitting works had revealed the presence of bands of very low strength tuffaceous claystone, a more detailed site mapping and 4 additional boreholes were then conducted along the wall alignment to better define the ground profiles. These weak seams were identified at representative sections at wall chainages Ch20, Ch40 and Ch95. These seams were typically encountered between 2.0 m and 7.0 m depth below ground level near the RSW block and daylighted the slope in front of RSW wall. The distribution of these weak seams would induce slope instability if the ground were not treated. The slopes appeared stable before earthwork was started in late 2011. After temporary excavation for drainage works and heavy rainfall, a few local slip failures occurred. Slickensides in weak seams were observed indicating movement along a slip planes. The presence of the slickensides also indicates that the weak seams may have been at residual strength values. Following the construction of RW17 RSW block further assessment were undertaken during the reassessment of RW18 and the materials on the slope behind RW17. As access to the upper levels of the slope was possible it was found that the colluvial cover was much thinner in many parts of the upper slope and the thicker colluvial cover was due to a small filled valley (Och et al. In Prep). Therefore, the RW17 modelling reflected the worst case assessment (Figure 4).

3 DESIGN CHALLENGES

The following challenges were faced by the RSW designer:

- To determine the strength and deformation parameters for the clay seams within the weathered tuffaceous claystones
- To meet the stringent global slope stability requirements of RMS Specification R57
- To assess the forces and deflections of the slope stabilizing piles
- To verify the design

3.1 SHEAR STRENGTH

The evidence of slickensides exposed in foundation excavations for RW17 indicates that the tuffaceous clay seams have in places experienced shearing movement. Therefore it was prudent for the designer to adopt the residual strength in order to achieve a suitable design for the RSW for its 100 years' design life. Assessment of the residual strength of the weak seams became critical.

3.2 SLOPE STABILITY

RMS Specification R57 Clause 4.7.3 requires: under load combination F (all load factors set to 1.0 except 50% of live load and 75% of earthquake load), the minimum overall factor of safety achieved must be 1.35 for RSWs not supporting bridge abutments. This load combination proved to be critical for the design of slope stabilizing piles that support RW17.

3.3 DESIGN METHODS FOR PILES

There is no RMS approved method for the design of slope stabilizing piles. Poulos (2011) recommends a general design approach that involves in three steps:

- (1) Evaluating the total shear forces needed to achieve the desired FOS of the study slope
- (2) Evaluating the maximum shear force that each pile can provide to resist sliding soil mass caused by slope instability
- (3) Selecting pile type, size and number and the most suitable location in the slope

To assess the bending moments in the piles within an unstable slope, as illustrated in Figure 3 (Poulos, 1995), it is required to have the distributions of lateral soil modulus and limiting soil pressures with depth, and the free-field horizontal soil movements. It is not difficult to estimate the distribution of soil modulus and limiting soil pressures (e.g. Broms, 1965); however the free-field horizontal movements are difficult to obtain for this project because there were no inclinometers installed before this issue occurred. Furthermore the construction of RW17 was on critical path and it was too late to install any instruments to obtain such results. Assumed free-field lateral soil movements may be an option; however with the reinforcing piles in place, the slope would move much less thus this approach may result in an overly conservative design of the piles.

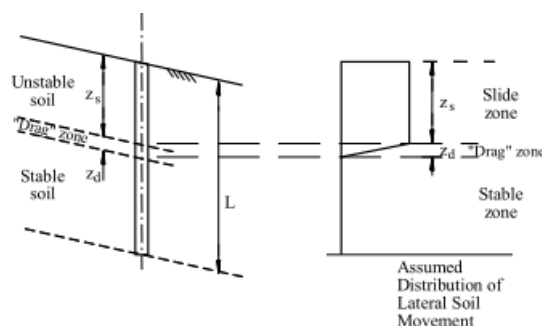


Figure 3: Basic problem of a pile in an unstable ground (source: Poulos, 1995)

3.4 DESIGN VERIFICATION

Due to the complexity and uncertainty of the horizontal soil movements and pile design, verification of the design adequacy was another challenge.

4 DESIGN DEVELOPMENT

In order to reduce the risk and uncertainties of slope instability due to the high RSW loading and low strength clay seams and to ensure a sustainable design solution was developed, the following key strategies and innovations were adopted.

4.1 RESIDUAL STRENGTH OF WEAK SEAMS

Tri-axial tests were carried out for R5 materials to determine the peak shear strength parameters. Two ring shear tests were conducted on an adjacent cutting to determine the residual strength of the weak seams. The samples for testing were collected from the weakest seam of tuffaceous claystones (logged as clay seam). Published data and engineering judgement and the ring shear test results were utilised to determine the residual effective friction angle of the weak seams, which ranged from 15 to 17.5 degrees. Design parameters for the soils and rocks are shown in Table 1 below.

Table 1: Design parameters for soils and rocks

Material	Unit weight (kN/m ³)	Modulus E _{ref} (kN/m ²)	Poisson's Ratio ν	Cohesion c' (kPa)	Friction Angle ϕ' (degree)
Engineered. Fill	20	30,000	0.3	1	34
Colluvial	18	30,000	0.3	36	16.5
R5	22	50,000	0.3	36	16.5
R4	23	200,000	0.3	25	29
R3	24	250,000	0.3	75	34
Seam	22	10,000	0.3	2	15-17.5

4.2 SLOPE STABILITY ANALYSIS AND PILE FORCES

A limit equilibrium method (LEM) using Slope/W program was initially adopted to check the FOS of global slope stability with and without stabilising piles. A typical model is shown in Figure 4. Steel reinforcements inside the RSW block were modelled as a horizontal load in accordance with R57. The calculated FOS without piles was in the range from 0.9 to 1.2 which were below the required 1.35 as discussed in Section 3.2. Thus a decision was made to install stabilising piles in front of the RSW. The pile lateral resistance was modelled as a concentrated point load applied at the seam location. The concentrated load was calculated from the limiting pressures using Broms (1965) method. Various slip surfaces were checked - circular, user defined, wedge. It was found that user defined slip surfaces along the weak seams generally gave the lowest FOS.

LEM analyses (PLAXIS) indicated that with the inclusion of the piles the required minimum FOS could not be achieved for selected critical RSW sections. In addition, the SLOPE/W software is not designed to calculate the bending moments in the piles which were required for structural design. Therefore it was assumed that the shear force in Slope/W model was applied at a cantilever height of half the depth from ground surface to the weak seam level. However this may lead to excessive bending moments if weak seams were encountered at a greater depth. In addition, LEM ignores stress-strain behaviour of the ground and the pile-soil interaction and cannot calculate the pile movements. These serious limitations required the designer to consider the use of a more rigorous finite element (FE) method. An engineering decision was subsequently made to run the Slope/W model at all representative chainages; should FOS be found less than 1.35, a two-dimensional (2-D) FE analysis would be carried out to check for the FOS, pile deflection and forces.

The finite element method (FEM) overcame such limitations and calculated the pile shear forces and bending moments by modelling staged construction of the RSW block. It also predicted the overall FOS by Shear Strength Reduction Technique (ϕ -c reduction method in PLAXIS), which reduces $\tan(\phi)$ and c' at the same rate until the program cannot converge, in other words, a failure mechanism has formed and the slope fails. The ultimate bending capacity of the piles was input in such calculations and the calculated ultimate shear force was then checked for structural adequacy. It was found that the critical failure surface was predominantly determined by the location of the weak seam layers which to a certain extent appeared to be more realistic than that arbitrarily defined from Slope/W applying a user specified method. FOS from PLAXIS analyses were all above the minimum required 1.35 except at Section Ch95 where the computed FOS was slightly less than 1.35. PLAXIS predicted the behaviour of both the RSW and reinforcing piles and overall slope stability both during construction and in the long term.

It was found that different FOS was calculated from Slope/W and PLAXIS and this difference could be as high as 20%. This is not surprising as these two engineering programs use different computation methodologies and different slip surfaces were assumed in the two programs. Figure 5 shows the FE model of the same section as illustrated in Figure 4. To address the difference between the two programs, the calculated shear force from Plaxis was input in the Slope/W model. The predicted FOS was found to have improved with a difference of generally less than 10%. This gave the confidence to the designer with the adopted design.

Staged construction was modelled in PLAXIS so that RSW movements, pile deflection and forces were calculated at each stage during the construction and in the long term. FOS of overall slope stability was checked for the critical stage during construction, i.e. when the RSW has reached full height and in the long term. The maximum pile forces during construction and in the long term were treated as working load; these were multiplied by 1.5 to work out the ultimate loads and compared with those from stability calculation; the greater of the two was considered as the maximum ultimate loads for structural design. The predictions are tabulated in Table 2 below.

Table 2: Design parameters for soils and rocks

Chainage*	Max deflection ⁺ (mm)	FOS	
		Plaxis	Slope/W
0-35(20)	12	1.48	1.56
35-85(40)	20	1.56	1.47
85-120(95)	7	1.32/1.37 [‡]	1.37

Notes: * number shown in bracket is the section chainage for Slope/W and PLAXIS analyses

⁺ horizontal displacement during construction

[‡] FOS for the case with seam effective friction angle of 15° and 17.5° respectively; 15° adopted all other Cases.

4.3 DESIGN OUTCOME

The details of the final design of slope stabilizing piles are summarised in Table 3.

Table 3: Reinforcing pile design details

Chainage	Pile Number	Pile diameter (mm)	Pile spacing (m)	Pile length (m)
0-35	3	450	2.5	9.0
35-83.5	3	750	2.5	9.0
83.5-120	1	1500	3.5	9.5

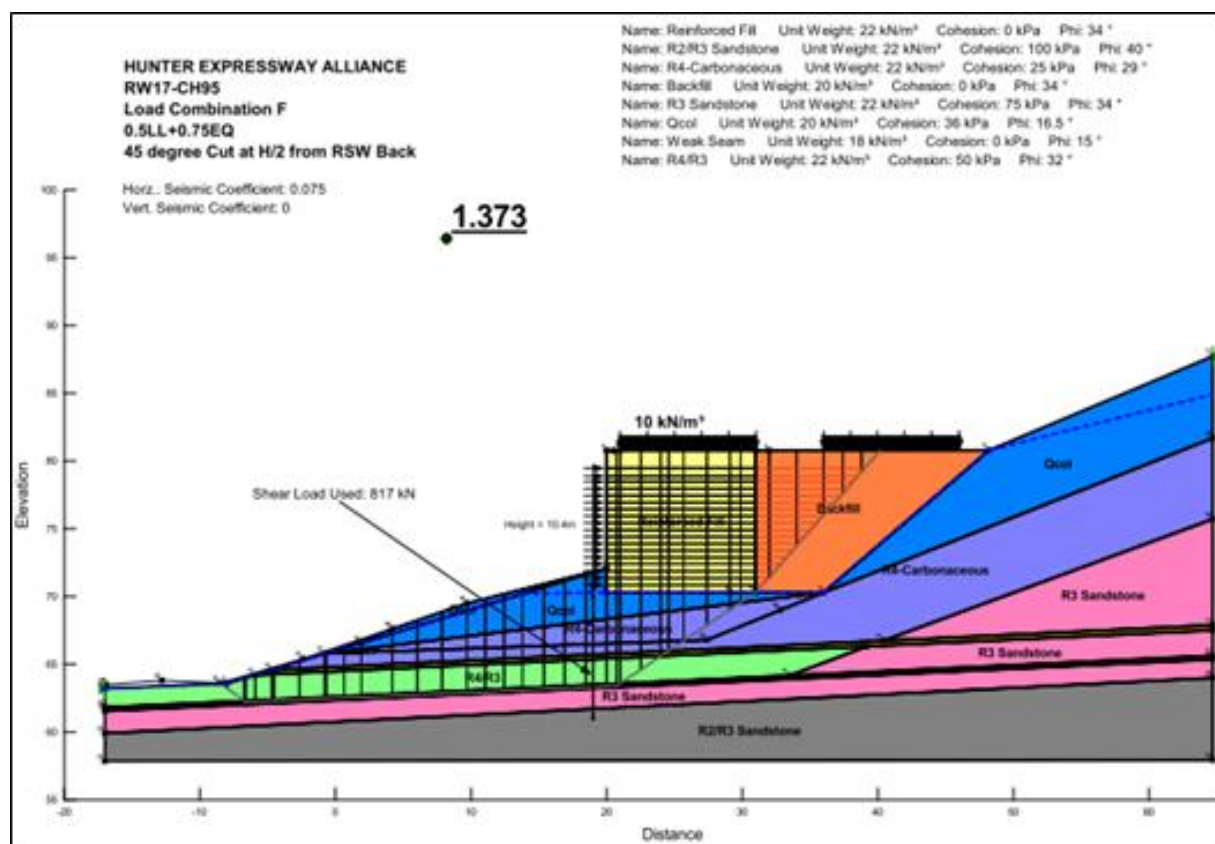


Figure 4: LEM model using Slope/W

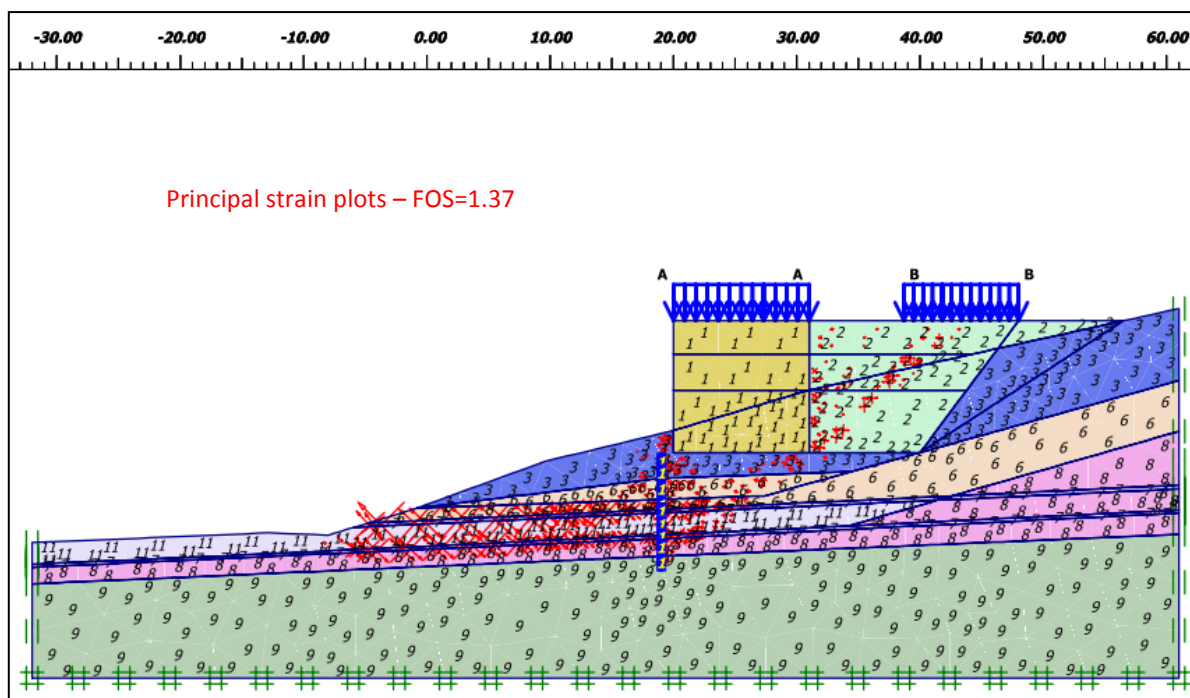


Figure 5: FEM model using PLAXIS

4.4 DESIGN VERIFICATION AND OBSERVATIONAL METHOD

The observational method was adopted to verify the design predictions and ensure construction safety. Two inclinometers were installed at about at Ch40 and Ch95, approx. about 1-2m in front of the RSW. Full height of the RSW was reached in early September 2012 followed by concrete slab pouring. The maximum horizontal movement of about 5mm was recorded by the inclinometer at Ch40 in late February 2013, as shown on Figure 6. This is well below the predicted 20mm. Although the monitored maximum deflection is much less than the predicted, the two deflection profiles are similar. The inclinometer at Ch95 was damaged during construction. Readings taken after the repair showed about the maximum 3mm deflection near the existing ground surface, which is less than the predicted 7mm at the same location from 2-D FE model.

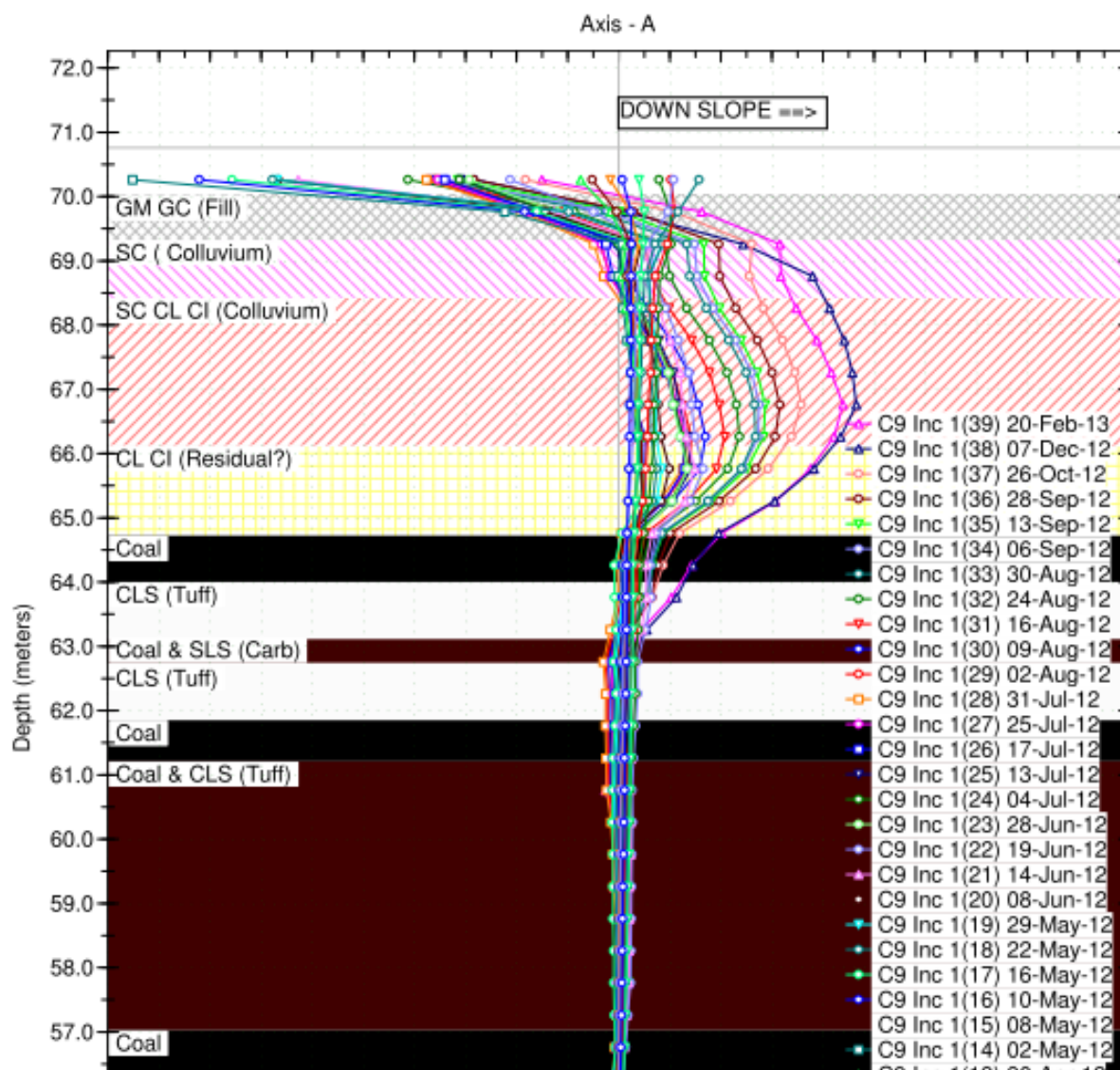


Figure 6: Inclinometer data at Ch40

5 DISCUSSIONS

Stabilizing piles were modelled as a 1-D plate element in PLAXIS. Smeared properties were adopted for the piles in the 2-D model with axial and flexural stiffness of EA/S and EI/S respectively, where E , A , I , S are Young's modulus, sectional area, moment of inertia and spacing of the piles respectively. Based on the first author's experience, use of smeared properties in a 2-D model tends to over-predict pile deflections and bending moment as compared with a 3-D model if the pile spacing is greater than 3 times the pile diameter. Thus it is probable that pile deflections and forces at Ch20 and Ch40 may be overestimated from the 2-D FE analysis.

For stability calculations, the phi-c reduction method in PLAXIS is relatively new to the industry compared to more conventional limit equilibrium method such as Slope/W. The former analysis might be affected by mesh density and geometry, and material properties and loads. Sound engineering judgement is needed so as to come up with sensible predictions. In this case study, ultimate bending moment was input to the LEM program and the maximum shear forces were calculated from the program and subsequently used for structural design. The calculated shear forces may be unrealistic due to the limitations of PLAXIS in which shear capacity cannot be input directly. Furthermore, forces calculated from phi-c reduction may not be conclusive because at the end of the calculation equilibrium is not achieved. To overcome this limitation, c' and ϕ' slightly higher than those at the end of the calculation should be adopted to ensure that equilibrium is achieved.

In spite of the above limitations, phi-c reduction in PLAXIS is a promising calculation tool for stability analysis especially for the cases where complex soil-structure interaction is involved and displacements required. Although LEM Slope/W predicted a lower factor of safety for some design sections, the slip surfaces may not be realistic due to the fact that Slope/W itself does not consider stress-strain behaviour and soil-structure interaction. PLAXIS overcomes these fundamental limitations, thus the predicted slip surfaces are more reliable.

6 CONCLUSIONS AND RECOMMENDATIONS

A case history on slope stabilizing piles for high reinforced soil walls is discussed. Both LEM and FEM were adopted to evaluate the FOS of the global slope stability according to RMS Specification R57. The adopted phi-c reduction method in PLAXIS overcomes the limitations of Slope/W and successfully resolved the design issues of the stabilizing piles. The construction of the study RSW was on a critical path and any delay would have significant implications on the overall project cost and program. The monitored horizontal displacements are comparable to the predictions from 2-D FE analyses, demonstrating that the design is safe and justified.

Phi-c reduction is a relatively new technique and has its limitations as discussed in Section 4. However with Plaxis becoming more and more popular for routine design in the industry, it is expected that phi-c reduction will be used more and more for stability design especially when complex soil-structure interaction is involved. More education is needed for practising engineers who frequently use finite element programs in their day-to-day design work. Further research, refinement in the analytical skills and case studies are required from academics, practising engineers and contractors to improve and master this technique.

8 ACKNOWLEDGEMENTS

The authors wish to thank NSW Roads and Maritime Services for their approval to publish this case study. The authors also acknowledge the support and contribution of their Alliance colleagues: Kenny Lam, Joseph Mak, Geoff Russell, Ben Wallace, Athena Sooklall, Sudar Aryal, Jim Yang, Iain Hespe and Garry Ewen.

9 REFERENCES

- Aryal, S., Kingsland, R., Rees, P., Russell, G., Stahlhut, O., and Wheatley, D. (2013), *Hunter Expressway, Australia: dealing with poor ground and subsidence*, Proceedings of the Institution of Civil Engineers, Civil Engineering Special Issue 166, May 2013, Issue CE5, pp 22-27, ICE Publishing
- Broms, B. (1965), *The Lateral Resistance of Piles in Cohesive Soils*, J. Soil Mech. Found. Div., ASCE, Vol. 90, pp. 27-63.
- Hawley S. P. and Brunton J. S. (1995). *Notes to the accompany the 1:100,000 Newcastle Coalfield regional Map* (The Newcastle Coalfield). Coal and Petroleum Geology Branch, Department of Mineral Resources, Sydney.
- Hawley S. P., Glen R. A. and Baker C. J. (1995). *Newcastle Coalfield Regional Geology 1:100 000, 1st edition* (1st edition edition). Geological Survey of New South Wales, Sydney.
- HEA Technical Memo on RW17, 10/02/2012.
- Lindsay G. and Herbert C. (2002). Coal and conglomerate in the Newcastle Coal Measures--coeval facies or temporally unrelated? *International Journal of Coal Geology* **51**, 169-184.
- Och, D. J., Kingsland, R., Aryal, S. Russell, G. and Rodgers, C. (2013) Case studies of post investigation geological assessments: Hunter Expressway. (In press – accepted in IAEG to be published in 2014).
- Poulos, H. G. (1995). *Design of reinforcing piles to increase slope stability*. Canadian Geotechnical Journal, **32**(5), pp. 808–818.
- Poulos, H. G. (2011), *Applications of Piling to Infrastructure Development*, International Conference on Advances in Geotechnical Engineering, Perth, Australia, Nov.7-9.
- PLAXIS. (2007). *Finite element code for soil and rock analyses*, Version 8. PLAXIS b.v.

- SLOPE/W - Slope Stability Analysis, (2007), <http://www.geo-slope.com>, Calgary, Canada: Geo-Slope International.
- Zhang, H. and Choi B. (2013), *Foundation Design Challenges at Hunter Expressway Alliance Project in Australia*, International Symposium on Coastal Engineering Geology (ISCEG-Shanghai 2012), September 19-20, Shanghai. *New Frontiers in Engineering Geology and the Environment*, Springer Geology, 2013, Volume 9, pp.177-180.
- Zhang, H., Chandrasekaran B. and Lin K. (2012), *Preloading of Soft Ground for a Highway Embankment in Newcastle*, 11th Australia New Zealand Conference on Geomechanics (ANZ 2012), 15-18 Jul 2012, Melbourne, pp. 247-252.