TREATMENTS OF EXISTING ABUTMENT RSW USING PERMEATION GROUTING AND HORIZONTAL SOIL NAILING

Bosco Poon[1], Kim Chan[2], Greg Saenger[3] and Daniel Jones[4]
[1]Principal Engineer, [2]Senior Principal and Service Line Leader, [3]Principal Engineer, [4]Service Group Manager Geotechnics, GHD Pty Ltd

ABSTRACT

The construction of Bangalow Interchange on the Pacific Highway Upgrade Project was undertaken using a staged approach. All traffic was initially diverted onto the northbound carriageway of the existing Pacific Highway whilst the southbound carriageway was decommissioned and excavated down to a maximum of 6.7m below the existing fill embankment surface. A temporary soil nail retaining wall was required to support the vertical cut face at the median. The temporary wall and the northbound carriageway would be decommissioned after the interchange construction work.

One challenge to the temporary design was the presence of a Reinforced Soil Wall (RSW) at the abutments of the existing twin bridges over Bangalow Road. The proposed temporary wall alignment was to cut through the middle of the RSW block in order to allow the eastern half of the RSW block, along with the southbound bridge, to be removed. The diverted traffic was to travel over the existing northbound bridge, which would be supported by the remaining western half of the RSW block. One further challenge was that the existing RSW abutments were designed to support the bridge sill beams without any pile foundations.

This paper focuses on the design treatment of the existing bridge abutment subjected to partial demolition of the abutment RSW. Initially, the adopted strategy was to inject low pressure permeation grout into the existing RSW block prior to excavation. A grouting trial was conducted, which indicated that because of the presence of the relatively high fines content within the existing RSW, the injected grout was unable to permeate through the RSW fill to form a coalesced grouted mass. An alternative support system using horizontal soil nailing was subsequently adopted through the vertical cut of the abutment RSW. The design of the horizontal soil nails would not only need to satisfy the stability requirements but also require controlling the movements of the RSW to within a tolerable limit. A comparison of the survey monitoring results with the design predictions of the wall movements is outlined

1 INTRODUCTION

Prior to the construction works undertaken at Bangalow Interchange, the southbound carriageway of the existing Pacific Highway on top of a fill embankment had to be decommissioned. All traffic was diverted onto the existing northbound carriageway whilst construction works began at the Interchange site. At the end of the construction, all traffic was redirected onto the new main alignment followed by the decommissioning of the existing northbound carriageway.

During construction, the southbound carriageway was excavated down to a maximum of 6.7m below the existing embankment surface. The vertical cut face at the median was supported by soil nails (see Figures 1a and 1b), whilst live Pacific Highway traffic was operational above the wall. The design of this retention system was not only to satisfy all the stability requirements, but also to limit the wall movements such that the deformation of the pavement above the retaining wall is within a tolerable limit.

Another challenge to the temporary retaining wall design was the presence of the existing reinforced soil walls (RSW) at the abutments of the twin bridges over Bangalow Road. The proposed temporary wall was aligned along the median of embankment approaching the bridge abutments, such that the southern half of the RSW block and the southbound bridge structure was proposed to be removed. One further challenge was that the RSW, constructed in early 90's, had been designed to support the bridge sill beams without any piles support. The initial preference was to not have to soil nail the RSW block due to the presence of the steel straps that could potentially impede the installation of soil nails and or be damaged by the soil nail installation.

This paper presents the details of the design treatment of the existing bridge abutment subjected to partial demolition of the RSW and bridge sill beams. A comparison of the survey monitoring results with the design predictions of the wall movements is outlined.

2 GEOLOGICAL CONDITIONS

The geological conditions of the site were assessed based on the available borehole and test pits information at the subject wall location. The assessed geological profile is provided in Figure 2. In summary, the existing fill embankment heights at the southern and northern bridge abutment are up to about 7m and 5.5m, respectively. The embankment fill is underlain by in excess of 10 m thick very stiff residual soil. Groundwater was not encountered in the underlying residual soil layer during the borehole drilling. Except for the RSW, the general fill materials typically comprise stiff, very stiff and hard red-brown clay with high to very high strength basalt cobbles and boulders (core stones) commonly encountered. The high incidence of oversize rocks, in the general fill may have been exacerbated in the median, where core-stones encountered could have been preferentially removed from the pavement area and pushed to the side to be disposed of in the fill in the median and in the batters. Two test pits (up to about 2.6m deep) were excavated at the existing RSW abutments. It was observed that the RSW select fill is generally fine to coarse grained gravel, sub-angular to sub-round, and with sand and clay. The results of the particle size distribution testing conducted on six test pit samples indicate that the fines content ranges from 12% to 18%.

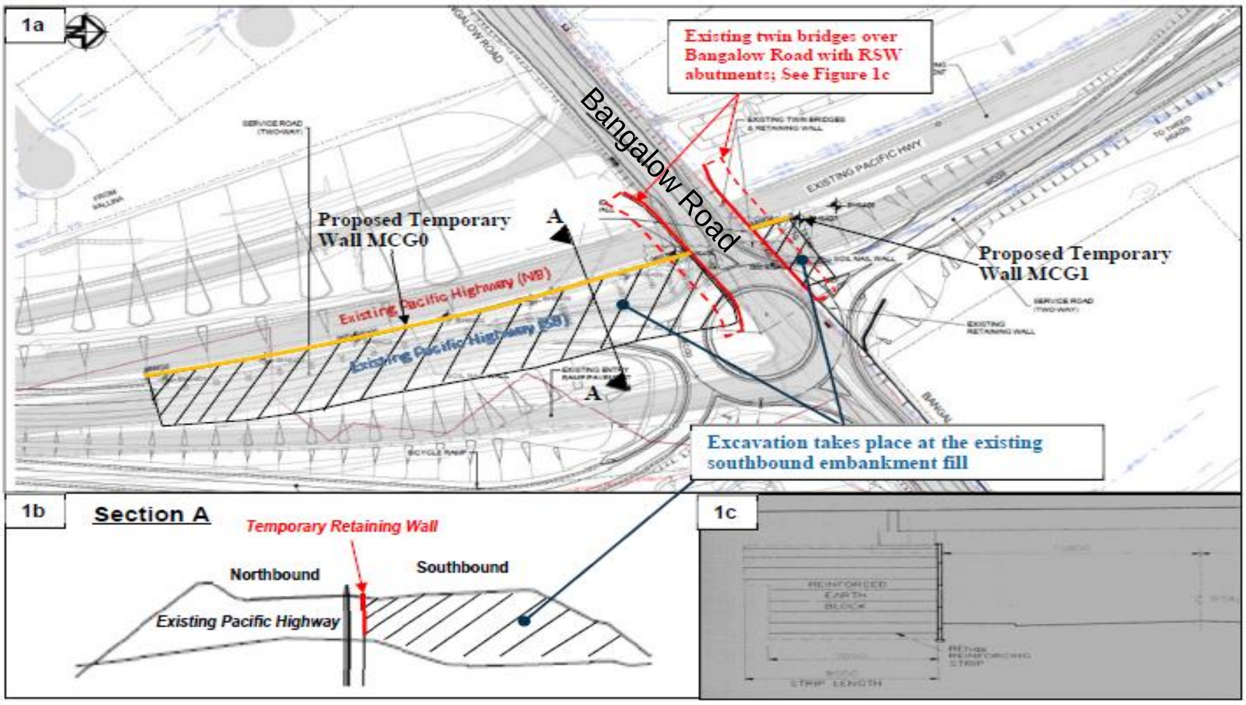


Figure 1: Temporary retaining wall at Bangalow Interchange

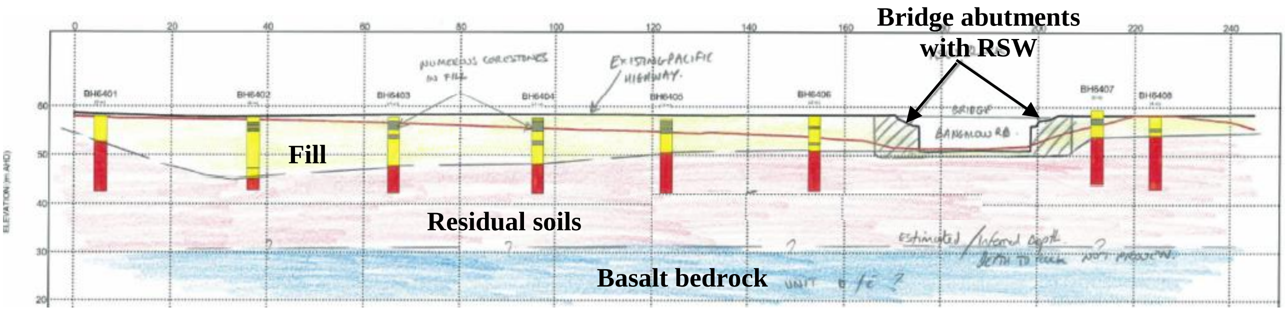


Figure 2: Geological longitudinal section

3 GROUTING OF EXISTING RSW BLOCK

Permeation grouting of the existing RSW blocks was initially proposed in order to strengthen the granular RSW fill prior to the excavation through the middle of the bridge abutments. The grouted RSW block will form a gravity structure to retain the embankment fill behind, as well as to support the bridge sill beam.

Permeation grouting is the injection at low pressure of a low viscosity liquid grout that infiltrates the soil pores and locks the soil particles in place as it cures. The advantages of permeation grouting over other high pressure grouting techniques such as jet grouting is that it entails minimum disturbance on the existing RSW and the bridge structures. The high pressure jet grouting, however, reworks the soils by breaking down the soil formation using a very high-velocity (jet) energy, removing spoil to the surface via return flow, and introducing a binder in the form of a grout. Therefore, the jet grouting will cause significant disturbance to the existing bridge abutment structures, in particular the steel straps that are an integral part of the RSW block.

Generally, unconfined compressive strength (UCS) of the grouted soil (14 days) is in the range of 300kPa to 2.0MPa. For the grout block design and to provide adequate bearing capacity for the support of the bridge sill beam load, the specified minimum ultimate UCS of the grouted soil has been taken as 0.7MPa.

A grouting trial was conducted in the RSW at the southbound abutment prior to excavation. The grout was injected via an injection pipe that was inserted into a vertical pre-drilled hole. It was observed that significant grout loss occurred and a coalesced grout mass was unable to form within the RSW block. The unsuccessful grouting trial was due to:

- The particle size distribution results from the test pit investigation indicated well graded select materials within the existing RSW. There was about 60% to 80% of gravel size material in the select fill while the fine content (< 75μm) was between 11% and 18% (see Figure 3). In particular, the fines content was considered high for the application of permeation grouting since difficulty in grout starts at between 5% and 10% of fines (see Figure 4); with 10% to 20% falling in the ‘marginal’ category, and soils that contain 20% fines or greater generally considered ‘non-groutable’ (Karol, 1990).
- Excavation within the RSW after the grouting trial revealed that the grout permeated the soil matrix in the coarse sand/gravel, but simply displaced the soil in the more fine sand/silty backfill. The heterogeneity in the RSW select materials has caused preferential flow of grout which was difficult to control and the results were unpredictable.

Due to the unfavourable grouting attempt, an alternative support system using horizontal soil nailing was proposed in consultation with the Construction Team, as outlined in the sections below.

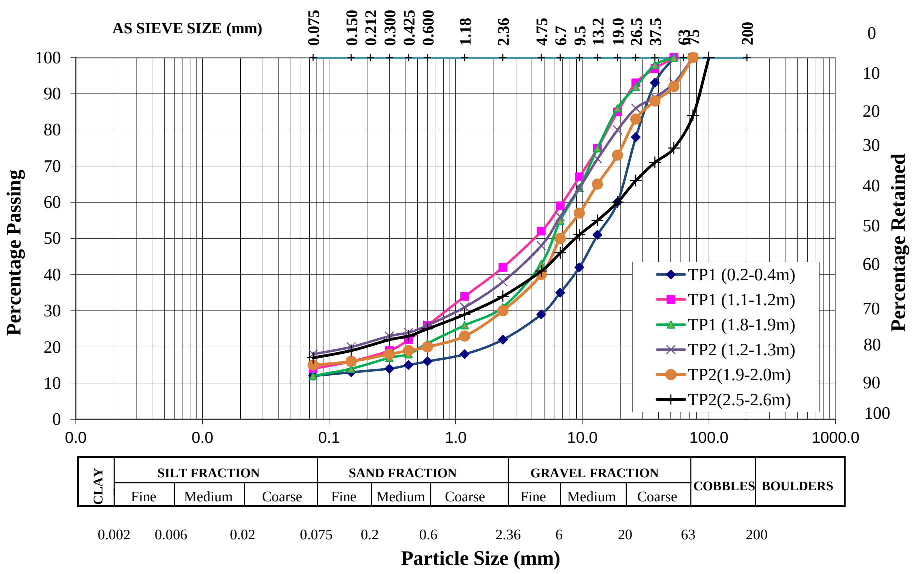


Figure 3: Particle size distribution results for the existing RSW select fill materials

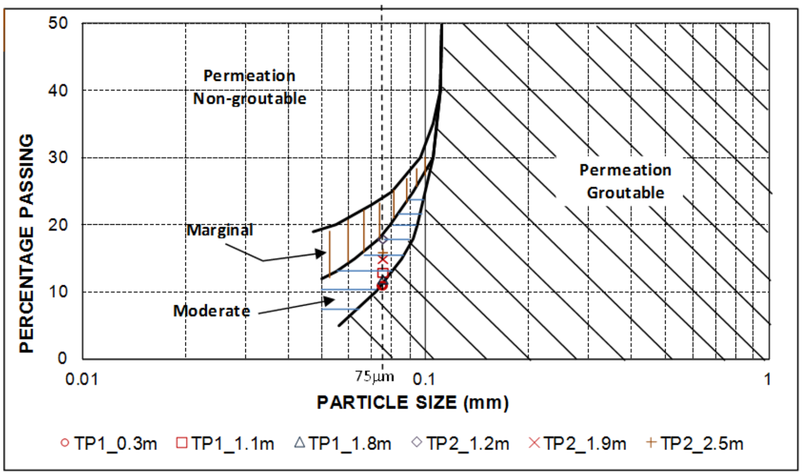


Figure 4: Measured fine contents in relation to published permeation grouting data of Karol (1990)

4 HORIZONTAL SOIL NAIL DESIGN

For the soil nail option in the RSW wall, the existing RSW straps at the northbound carriageway were to remain to provide lateral support for the abutment wall face, whilst the stability of the wall excavation across the RSW had to be provided by installing horizontal soil nails through the spacing of the RSW reinforcing straps that are perpendicular to the proposed nails, as depicted in Figure 5. Note that the existing RSW, along with the reinforced straps, was aligned at an oblique angle of 15° from the normal to cut face along the median strip of fill embankment. Realignment of wall excavation at the southern abutment was considered necessary in order to avoid the reinforcing straps of the RSW poking out of the cut face. For the northern abutment, the reinforced straps were directed away of the wall excavation; therefore wall realignment was not required. Figure 6 shows the revised wall alignment at the southern abutment, which was parallel with the RSW straps thus minimising the disturbance of the straps during excavation. Figure 6 shows a 3D view of the wall realignment, which indicates that wall excavation along the median strip ended behind the headstock wing walls. A 2 m wide cross bench was provided over the lower-tier wall that was normal to the RSW face.

The RSW wall heights immediately beneath the sill beam structures were 4.5 m and 3 m for the southern and northern abutments, respectively. The soil nail requirements in the vicinity of the bridge sill beam structures were up to 5 rows of 9 m long horizontal soil nails at 0.75 m vertical and horizontal spacing. Away from the sill beams, the soil nail length was reduced to 6 m and the horizontal spacing increased to 1m while the vertical spacing remains at typically 0.75 m so to avoid intercepting the existing RSW straps. Hollow core self-drilling anchor bars with a nominal outer diameter of 30 mm have been used for soil nailing. The grout-hole diameter created by the drill bit was 115 mm. During installation, the sacrificial drilling bit was threaded onto to the hollow soil nail bar so that the bar also served as the drilling rod. After drilling to the design length, grout was pumped via the hollow tube to fill up the drilled hole from the base of the hole, until clean grout appeared at the collar. Thus, upon completion, the void both outside and inside of the hollow tube was fully grouted.

As indicated in Figure 7, the facing structure consisted of two shotcrete layers that were applied sequentially within a cycle of two lifts of soil excavation and soil nail installation, with each lift of excavation limited to no more than 1.2 m height. The first shotcrete layer was a 50 mm thick temporary sealing layer that was applied immediately after each excavation lift and the installation of strip drains. This was to prevent moisture change and unravelling at the excavated face. The second shotcrete layer was a 100mm thick final layer with steel mesh reinforcement, which was applied to provide structural support to the face. It was applied after the installation of the soil nail row at the second excavation lift. The shotcrete was to have a minimum 28 days compressive strength of 32 MPa. All soil nails were installed when shotcrete strength of the temporary shotcrete layer has achieved 10 MPa. The face plates of the soil nails at the first and the second rows were not required to be tightened until the final 100mm shotcrete facing took place at the end of the two-stage installation cycle. This installation cycle was repeated until design cut level was achieved.

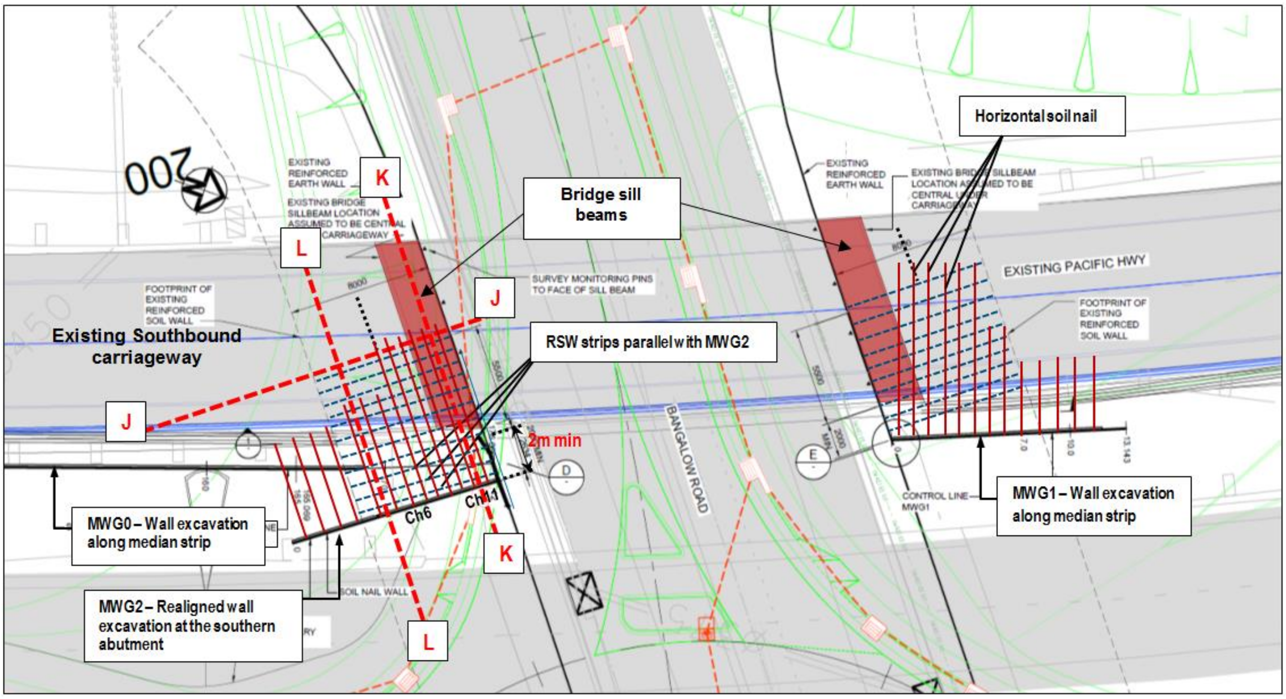


Figure 5: Horizontal soil nailing for stabilisation of wall excavation across existing RSW bridge abutment.

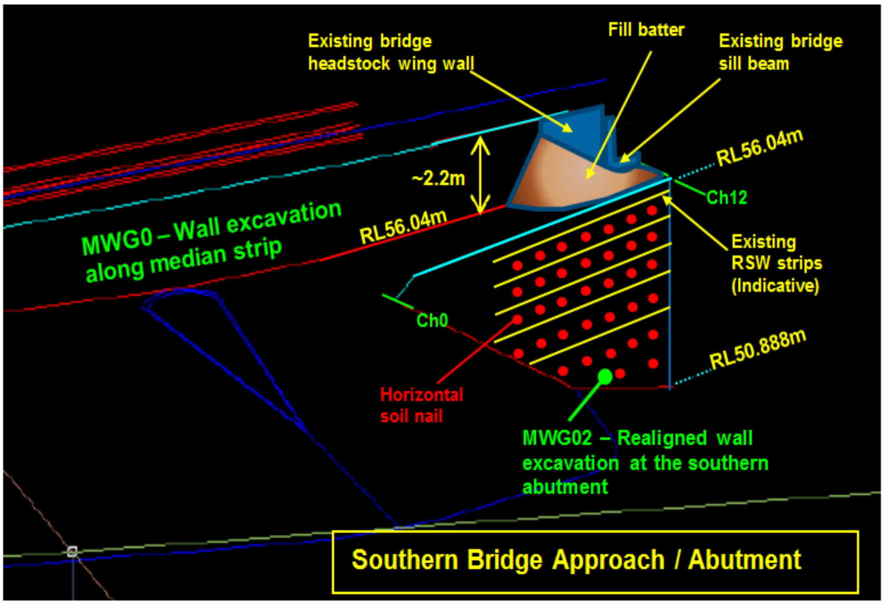


Figure 6: 3D view of soil nail wall alignments

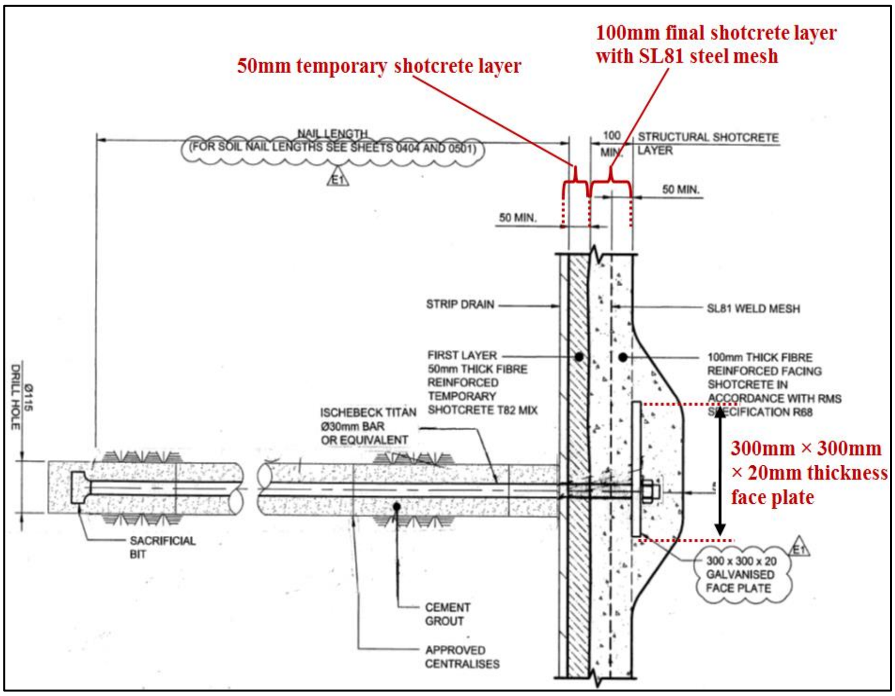


Figure 7: Soil nail wall facing details

5 STABILITY ASSESSMENT

The stability of the wall excavation across the RSW abutment and its influence on the existing bridge sill beam structure were of paramount importance for the temporary wall design. Three critical design sections, Sections JJ, KK and LL, have been considered for stability assessment purposes (see Figure 1 for locations). Section JJ is a longitudinal section adopted for the assessment of the RSW stability in accordance with the current RMS R57 requirements, noting that the RSW was built in the early 1990s before R57 was available. However, it was not our intent to consider any upgrade of the existing RSW to meet the R57 requirements. Sections KK and LL were the transverse sections (perpendicular to the soil nail wall face) adopted for stability design of the wall excavation and soil nail requirements. The design of the soil nail wall has been based on the limit state design principle in accordance with AS5100.3-2004.

The existing RSW at the southern abutment (section JJ) was about 5m high plus 1m of wall embedment below the original ground surface. The RSW block had up to 9 rows of reinforced straps at 0.75 m vertical and horizontal spacing, which were connected to face panels of 1.5m square. The width of the RSW block was up to 8m. The stability assessment has indicated that although the RSW block width was satisfactory for external stability requirements of R57 (i.e. sliding, overturning and bearing), it fell short of pull-out resistance to meet the current requirements for internal and global stabilities. In particular, the stability against pull-out failure of the existing reinforcing straps was only marginal. The computed stress-strength ratios against pull-out for the 5^{th} and 6^{th} rows of the reinforcements were slightly less than 1.0. The computed factor of safety (FoS) against global failure was 1.45, which was less than the required minimum of 1.6 for RSW supporting bridge abutment as required by R57. Although the assessment indicated that the stability was marginal under the current R57 standard, there were no obvious signs of distress of the wall prior to wall excavation (e.g. bulging of the RSW panels, cracking of the road pavement above the RSW). Moreover, the RSW would be decommissioned at the end of the interchange construction work after 1 -1.5 years, therefore it was not considered necessary to carry out any upgrade of the existing RSW to meet R57 requirements.

The stabilities of the soil nail wall at the locations immediately beneath (section KK) and at an offset distance (section LL) from the sill beam structure have been assessed in accordance with AS5100.3-2004 using appropriate load factors for the imposed soils and bridge loads. The adopted geotechnical reduction factors for the soil shear strength and for the bond resistance of the grouted nail were 0.55 and 0.5, respectively. It is noteworthy to point out that the global stability of the soil nail wall at Section K-K underneath the sill beam structure is a three dimensional problem; however, this is beyond the capability of the conventional limit equilibrium approach using method of slices, which is based on plane strain condition without the consideration of load spread within the soil mass. It is therefore necessary to apply a dispersed design pressure, as opposed to the actual bearing pressure at the underside of the sill beam, in the global stability analysis. A common way to assess the load spreading is to adopt the 2:1 load spread method in which the imposed sill beam load spreads out longitudinally along the line with a vertical-to-horizontal slope of 2:1 as shown in Figure 8 until it intercepts the potential critical slip surface associated with the wall excavation (generally occurred towards the base of the wall as indicated by slope stability analysis results). It is noted that the adopted 2V:1H load spread was considered conservative for the compacted RSW fill. It can be demonstrated using elastic solution or other more sophisticated finite element methods that the angle of load spread will increase (up to 1V:1H) as the stiffness of the soil increases.

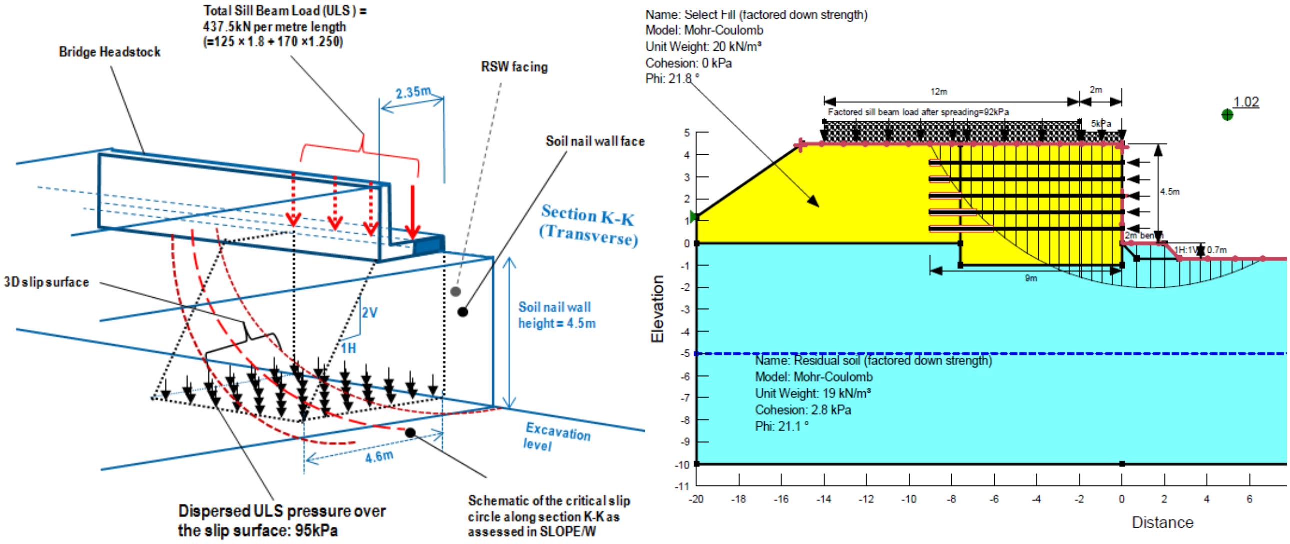


Figure 8: Load spreading for slope stability analysis

6 DEFORMATION ASSESSMENT

The proposed excavation with soil nails at the existing RSW and its impact on the overlying bridge sill beam structure is a three dimensional engineering problem. For practical assessment of the wall/sill beam deformations without recourse to the full 3D FEA, a simplified 2D FE approach using PLAXIS has been employed that comprised firstly an estimation of the current load state and deformation of the existing RSW under the maximum serviceability limit state (SLS) sill beam load, followed by the prediction of additional deformations caused by the excavation of the abutment fill. The two-stage analysis process is outlined below:

Stage 1: Carried out a 2D FEA for the longitudinal section J-J perpendicular to the existing RSW abutment as shown in Figure 9. The predicted stress distribution and displacement at the underside of the sill beam structure were induced by (i) the dead weight of the sill beam, (ii) the back filling of the sill beam, (iii) the application of the un-factored maximum bridge load (SLS), and (iv) the imposition of traffic load on top of the embankment. This design section did not include the modelling of the soil nail wall excavation, but was used to calibrate the Stage 2 FE model as outlined below.

Stage 2: Carried out a second 2D FEA for the transverse section K-K in order to assess the deformation caused by the excavation of the fill abutment (up to 4.5m high wall excavation plus 0.7m over-excavation) and the accompanying soil nail wall construction, as well as to predict the induced nail head load for the design of face plates. Note that the applied sill beam load on this 2D section under plane strain condition is assumed to extend infinitely in the out-of-page direction. It is therefore unable to model correctly the actual sill beam load that is in reality applied over a finite sill beam width of 2.35 m. To overcome this limitation of 2D model, the following strategy has been adopted:

- Prior to the excavation of the abutment fill and the installation of the supporting soil, an equivalent sill beam load simulated by artificial surcharge has been applied over the RSW fill to match the predicted vertical displacement of the sill beam structure as obtained from Stage 1 FEA for the orthogonal Design Section J-J. This applied equivalent sill beam load on Section K-K was considered to have accommodated the combined effect of various loading conditions in the orthogonal Section J-J.
- All displacements of the FE model were reset to zero after the application of the equivalent sill beam load. The cumulative displacements in the subsequent excavation stages gave the predicted sill beam/wall movements caused by the proposed excavation.

The effect of fill compaction within the existing abutment fill was considered significant on the deformation prediction. This was especially the case for fill excavation since the compaction of the RSW fill may have locked in higher lateral pressure, thereby causing greater stress relief and horizontal displacement upon excavation. For the RSW construction in the Stage 1 FEA (Section J-J), an artificial surcharge of 100kPa has been applied and subsequently removed at each lift of RSW filling to simulate the lateral effect of compaction load. For the excavation analysis in Stage 2 FEA (Section

K-K), the locked-in horizontal stress due to compaction has been incorporated in the initial stress state prior to the fill excavation by multiplying the vertical fill load with an appropriate coefficient of earth pressure K_0. Seed and Duncan (1986) has proposed the following empirical equation for the estimation of Poisson's ratio of compacted fill:

$$\nu = \frac{4-3sin\phi}{8-4sin\phi} \quad (1)$$

The K_0 was then calculated based on elastic theory:

$$K_0 = \frac{\upsilon}{1-\upsilon} \quad (2)$$

For the modelling of the soil nail in Stage 2 FEA, an equivalent axial stiffness, E_{eq}, has been used to account for the combined stiffness of both grout cover and reinforced steel bar. E_{eq} can be calculated by:

$$E_{eq} = E_n\left(\frac{A_n}{A}\right) + E_g\left(\frac{A_g}{A}\right) \quad (3)$$

where E_g is the modulus of elasticity of grout material; E_n is the modulus of elasticity of nail; E_{eq} is the modulus of elasticity of grouted soil nail; A is the total area of the grouted soil nail calculated based on the diameter of the drill hole D; A_n is the cross-sectional area of the steel bar; and A_g is the cross-sectional area of the grout (the hollow tube also filled with grout) calculated by $A_g = A - A_n$. For the grouted soil nails that were spaced horizontally at s_h, the equivalent axial rigidity $EA_{eq\ (2D)}$ that was used in the 2D FE model could be determined by:

$$EA_{eq(2D)} = \frac{E_{eq}\times A}{s_h} = \frac{E_nA_n+E_gA_g}{s_h} \quad (4)$$

Note that Equation 4 calculates the equivalent EA of the grouted nail without allowing cracking of the grout. As the soil nails were subject to tensile axial force, any cracking of the grout will reduce the rigidity of the nails. In the FE model, cracking of the grout has been considered by the following equation:

$$Cracked\ EA_{eq(2D)} = \frac{E_{eq}\times A}{s_h} = \frac{E_nA_n+\Phi_\mu E_gA_g}{s_h} \quad (5)$$

where Φ_μ accounted for the degree of cracking and has been adopted to be equal to 0.5. It is noted that in the FEA cracked EA has been used for the prediction of wall deformation and sill beam movement, whilst full EA (uncracked) was used conservatively for the prediction of the nail head load for face plate design.

For Stage 2 FEA and for the prediction of wall and sill beam movements, staged excavation with 50mm thick shotcrete took place in advance of the soil nail installation in Stage 2 FEA in Figure 9. This analysis sequence allowed for soil relaxation, thus predicting greater movements but lower soil nail force. Conversely, for the prediction of nail head force, soil nails were conservatively assumed to be installed at the same time the excavation was taken place. This analysis sequence gave higher soil nail force as no soil relaxation was allowed prior to soil nail installation.

Interface elements have been used to model the interaction between the reinforcing straps of the RSW and the surrounding fill, as well as at the interface of soil nail and fill. In which the pull-out resistance of the reinforcement was adjusted by a strength reduction factor, R_{int}, such that the maximum shear stress, τ_{max}, at the interface is:

$$\tau_{max} = \sigma'_n \times R_{int} \times tan\phi'_{soil} \quad (6)$$

where σ'_n is the stress normal to the interface elements; and ϕ'_{soil} is the soil friction angle. R_{int} has been set to 0.8 for deformation assessment, and 1.0 (at the interface of soil nail and fill only) for nail head load prediction.

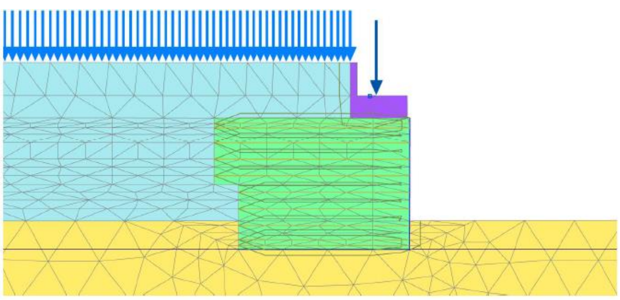
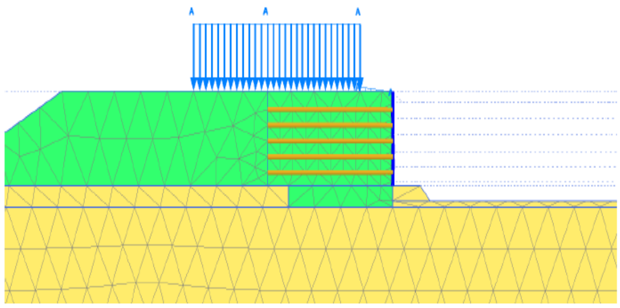
Stage 1 FEA (on Section J-J)	Stage 2 FEA (on Section K-K)
	

Figure 9: Existing RSW construction in Stage 1 FEA and wall excavation in Stage 2 FEA

Figure 10 presents the predicted displacements and stresses at the underside of the sill beam structure from Stage 1 FEA for section J-J. The predicted vertical displacement of the sill beam was about 50mm (Figure 10a). Part of this settlement however occurred during construction. The settlement due to the operational (live) bridge load was assessed to be about 18 to 20 mm. The predicted horizontal displacement of the RSW was up to about 25mm at the underside of the sill beam level (Figure 10b). Part of this displacement occurred during construction, the maximum wall displacement (at the underside of the sill beam) due to the bridge live load was assessed to be about 15mm. The predicted bearing pressure across the base of the sill beam varies from about 80kPa to a maximum of 187kPa (Figure 10c). This agreed well with the maximum design bearing pressure of 180kPa as given in the as-built RSW drawing.

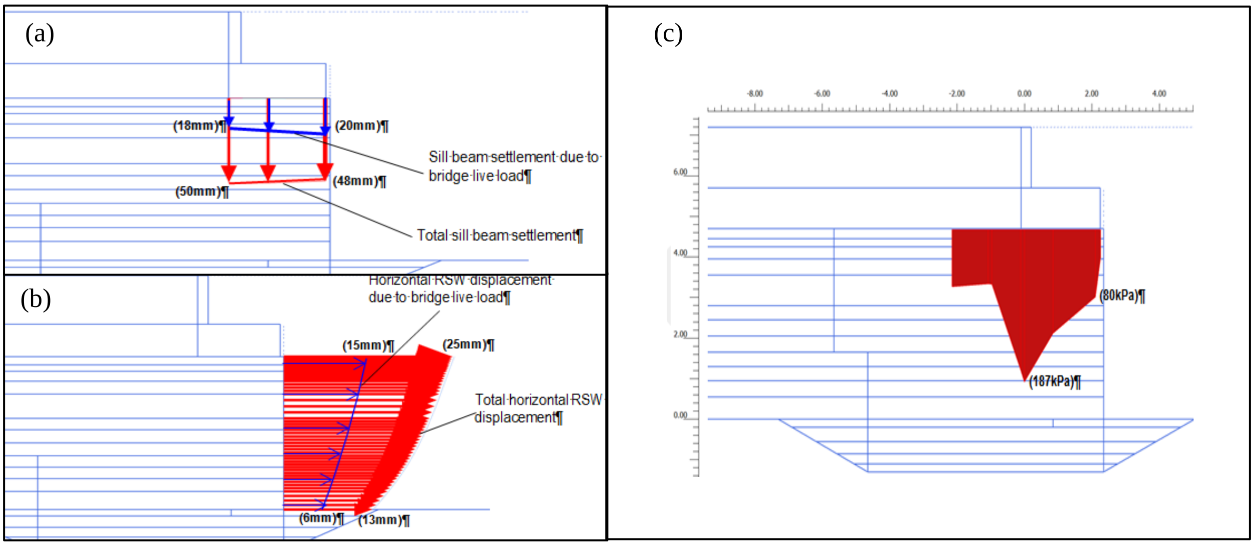


Figure 10: Predicted displacements and stresses at the underside of the sill beam from Stage 1 FEA for Section J-J

Figures 11a and 11b present the predicted vertical and total horizontal displacements for Section K-K. The predicted results have been based on FEA using a cracked EA of the soil nail and a reduced shear strength (i.e. R_{int} = 0.8) at the soil to nail interface. Also, soil relaxation was allowed prior to the soil nail installation to allow for greater predicted movements. The analysis results have indicated that the maximum settlement at the top of the soil nail wall face was up to 7mm (Figure 11d), which is about 0.15% of the retaining wall height. The predicted horizontal displacement at the top of the wall face was 15mm (Figure 11e), which is about 0.3% of the retaining wall height. Figure 11c indicates that the induced fill settlement at the underside of the sill beam was less than 5 mm. Some minor heaving (<1mm) was predicted at about 4 m from the sill beam edge, which was likely to be caused by unloading following excavation in front of the soil nail wall face.

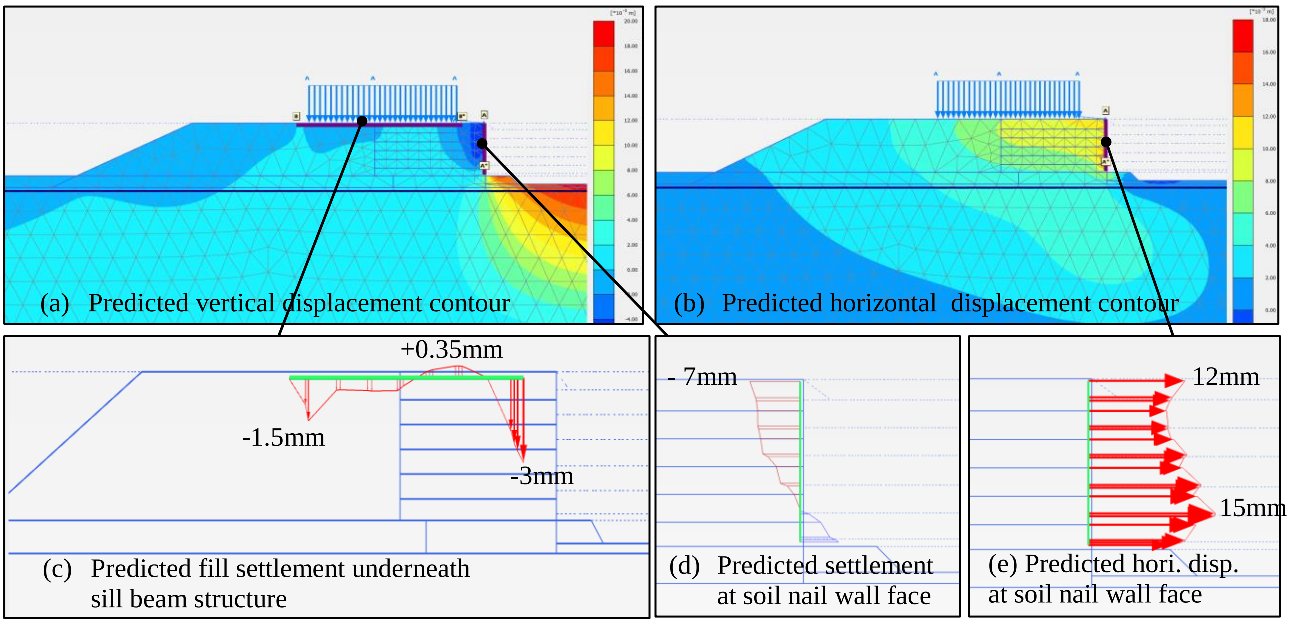


Figure 11: Predicted displacements of wall excavation from Stage 2 FEA for Section K-K

The variation of the axial forces along the soil nails was predicted based on FEA using (i) uncracked EA of the soil nail, (ii) soil to nail interface reduction factor, R_{int}, of 1.0 (i.e. no strength reduction) and (iii) no soil relaxation was allowed prior to the soil nail installation. The predicted maximum nail head force was 47kN that occurred at the bottom soil nail row. This assessed value formed the basis for face plate design.

7 FIELD MEASUREMENTS

Survey markers were installed at the soil nail wall face for the monitoring of the wall movements during and after construction. Figure 12 shows the locations of the installed survey markers. Figure 13(a) shows that the measured settlements at the wall face were up to about 2 mm, which is less than the predicted maximum value of 7 mm given in Figure 11b. Figure 13(b) indicates that up to about 5 mm of horizontal displacement has been registered at the survey markers on the wall face. This is less than the predicted maximum value of 15 mm given in Figure 11c. In general, the predicted movements were 2 to 3 times greater than the measured values, which might be attributed to the following reasons:

- It was of design intent to be erring on the side of caution when undertaking the prediction of wall deformation. In particular, considerations relating to construction induced wall movements have been taken into account in the design assessment. This included the effects of (i) relief of the locked-in horizontal stress within the compacted fill of the RSW; and (ii) soil relaxation during each lift of wall excavation prior to the installation of soil nails.
- The use of the cracked axial rigidity *Cracked* $EA_{eq\ (2D)}$ of the grouted nail and a reduced friction angle at the interface between soil and grouted nail might have underestimated the axial stiffness for the short term.
- It is well known in literature that it is difficult to achieve reliable predictions of lateral movements because of the anisotropy of the soil stiffness (or the stiffness of the RSW fill in this case) and effect of dilation within the RSW fill materials.

8 CONCLUSIONS

This paper presents the design treatment of the existing bridge abutments subjected to partial demolition of the abutment RSW. Initially, the adopted strategy was to inject low pressure permeation grout into the existing RSW block prior to excavation. However, it was observed from a grouting trial that significant grout loss occurred and a coalesced grout mass was unable to form within the RSW fill. Note that the fill was well graded and had fine content of up to about 18%. It may be concluded that permeation grouting is better suited for relatively homogeneous soil with a low fine content. In heterogeneous soil, such as the well graded RSW fill encountered in the present case study, permeation grouting causes preferential flow of grout that is difficult to control and the results are unpredictable.

Owning to the unfavourable grouting attempt, an alternative support system using horizontal soil nails was proposed. In this approach, the existing RSW straps were to remain to provide lateral support for the abutment wall face, whilst the stability of the wall excavation across the RSW had to be provided by installing horizontal soil nails through the spacing of the RSW reinforcing straps that are perpendicular to the proposed nails. The design of the horizontal soil nails would not only need to satisfy the stability requirements but also require controlling the movements of the RSW to within a tolerable limit. Furthermore, the proposed excavation at the RSW beneath the bridge sill beam structure is a three dimensional engineering problem. For practical assessment, some simplifications have been adopted in order to circumvent the 3D problem without recourse to full 3D design analyses.

The analysis predictions have been compared with field measurements. In general, the predicted movements were 2 to 3 times greater than the measured values, which might be attributed to the following reasons:

- It was of design intent to be erring on the side of caution when undertaking the prediction of wall deformation. Some of the design assumptions adopted might have underestimated the axial stiffness of the soil nails for the short term.
- It is difficult to achieve reliable predictions of lateral movements because of the anisotropy and the dilatational effect within the RSW fill materials.

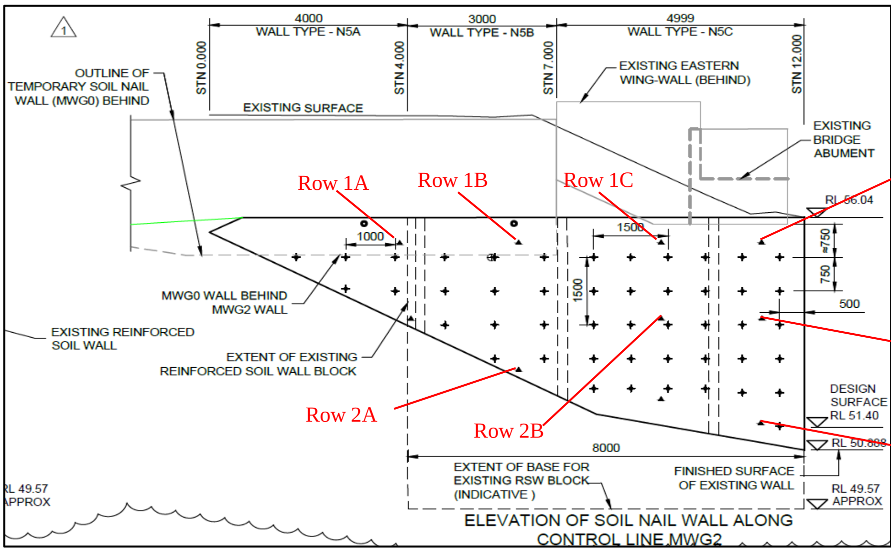


Figure 12: Survey markers at wall excavtion across RSW at the southern abutment

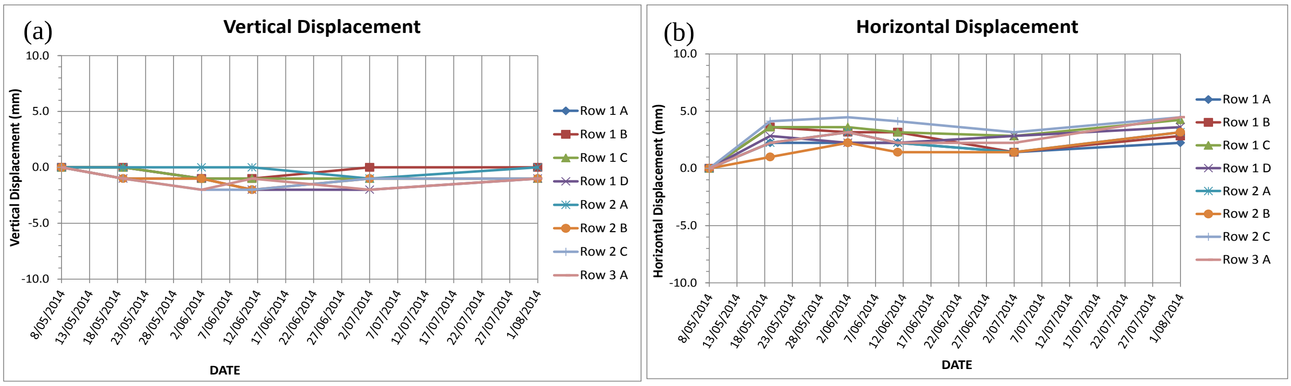


Figure 13: Survey monitoring results of (a) vertical and (b) horizontal displacements at the southern abutment

9 REFERENCES

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