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Pile and piled raft bridge foundation design and construction subject to mine subsidence effects

Concept et construction de fondations de ponts avec pieux et avec radier sur pieux, soumis aux effets de l'affaissement minié

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ABSTRACT: This paper presents a case study of the foundation design using pile and piled raft foundations for a twin-viaduct bridge (Bridge BW10) constructed over incised valleys subject to mine subsidence effects in the future. The original bridge design considered a piled foundation design with permanent casings for piles at Pier 2 to deal with the ground movement induced by the incised valley closure resulting from the far-field mine subsidence. During construction a weak rock seam identified at a shallower depth at Pier 1 required a piled raft foundation to deal with the mine subsidence induced ground movement. 2D and 3D numerical modelling were undertaken to assess the piled raft performance after a semi-analytical assessment has confirmed its feasibility. The bridge and foundations were successfully constructed and the monitoring results are within the predicted.

RÉSUMÉ: Cet article présente une étude sur le design de fondations avec pieux et avec radier sur pieux pour un pont à double viaduc (pont BW10) construit sur des vallées incisées sujettes à des effets de subsidence minière. La conception initiale du pont considérait une fondation sur pieux avec des enveloppes permanentes autour des pieux au pilier 2 afin de prendre en compte le mouvement du sol induit par la fermeture de la vallée incisée résultant de l'affaissement minier. Au cours de la construction, une couche de roche faible identifiée à une faible profondeur au pilier 1 nécessitait une fondation avec radier sur pieux pour gérer le mouvement du sol provoqué par l'affaissement minier. Des modélisations numériques 2D et 3D ont été entreprises pour évaluer les performances du radier sur pieux après qu'une évaluation semi-analytique ait confirmé sa faisabilité. Le pont et les fondations ont été construits avec succès et les résultats de la surveillance se situent dans les prévisions.

Keywords: Piles, Piled Raft, Bridge Foundation, Mine Subsidence

1 INTRODUCTION

The $1.7 billion, 40-kilometre Hunter Expressway, a dual carriage freeway, was designed to ease congestion and accommodate long-term development and growth in the Hunter Region in the coming decades, and to reduce travel times between Newcastle and the Upper Hunter in New South Wales, Australia.

The 13-kilometre eastern section was undertaken by Alliance team of Roads and Maritime Services (Formerly Road and Maritime Authority, RTA), New South Wales, CPB (formerly Thiess), WSP (Formerly Parsons Brinckerhoff) and Arcadis (formerly Hyder Consulting). This section of roads involved in two major grade separated interchanges, light and heavy vehicle rest areas, three twin-viaduct bridges, 23 additional bridges and mine subsidence foundation treatment. Precast segmental balanced cantilever methodology was

adopted to construct the viaduct bridges up to 45 metres high and up to 300 metres long, and a launching gantry to erect the deck segments to 75 meter long bridge spans.

This paper discusses Bridge BW10 (also named Viaduct 2) which is a twin-viaduct bridge with 3 piers and 4 spans, each having 52m, 75m, 75m and 52m respectively. The original design considered piled foundations for abutments and pier foundations, with only Pier 2 piles being sleeved down to about 5m to accommodate the ground movement through the identified "shear weak plane" induced by the far field old mine subsidence. A piled raft concept was proposed for Pier 1 when a "weak zone" was identified at around 5m depth by the additional borehole drilling during construction stage. This paper describes the design and construction process of pile and piled raft foundations.

2 GEOLOGICAL SETTING AND INVESTIGATIONS

The geology at the three viaducts along the Hunter Expressway alignment is underlain by Adamstown subgroup of the Newcastle Coal Measures as shown in the regional geology map of NSW. The subgroups comprise interbedded sandstone, siltstone, claystone (tuffaceous) and coal.

Concept stage investigations for Bridge BW10 were completed by the RTA in September 2004. Additional investigations consisting of ten (10) boreholes were proposed for the detailed design of the foundations at the bridge piers and abutments as required by RTA design requirements. Between October and November 2010, a total of four (4) boreholes have been completed with six (6) boreholes for Piers 1, 2 and 3, as shown in Figure 1, outstanding due to site access and environmental constraints. As such these boreholes would have to be drilled during construction stage to satisfy the RTA requirement that at least one borehole at each pier location. In addition, extensive site investigations were undertaken for old coal mine working grouting and verification.

3 GEOTECHNICAL MODEL AND DESIGN PARAMETERS

Subsurface conditions at Bridge BW10 typically comprise very stiff to hard silty clay to depths between 1.5m to 3.7m and underlain by the siltstone and sandstone, with extremely weathered to depth of approximately 5m below ground surface level. Highly to moderately weathered, low to medium strength sandstone and coal layers are encountered below the extremely weathered siltstones and sandstones. The rock profile is variable across the site. It is noted that horizontal ground movement above an inferred shear plane around RL28m to RL30m, as shown in Figure 2, was considered for bridge foundation design. This horizontal ground movement is due to "valley closure" movement associated with far field old mine collapse.

Rock classification is based on HEA rock classification criteria, which are determined by a combination of the unconfined compression strength (UCS), the defect spacing and percentage of seam of a concerned rock mass, which is based on the same principles by Pells et al (1978, 1998). The rock mass was divided into five classes, with Class I (R1) being the best and Class V (R5) being the worst in quality. The rock class R1 has been further subdivided into R1a and R1b.

A summary of adopted geotechnical design parameters for each class of rock is shown in Table 1. Note the contribution from soil has been ignored in the pile and piled raft foundation design.

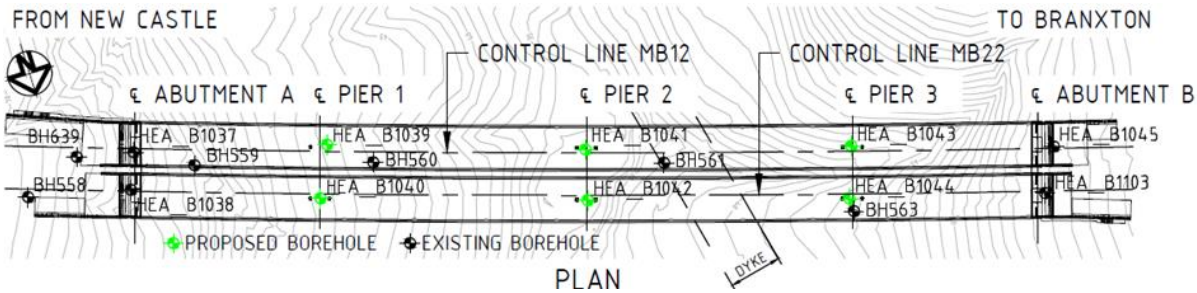

Figure 1. Plan of Bridge BW10 layout and proposed borehole locations

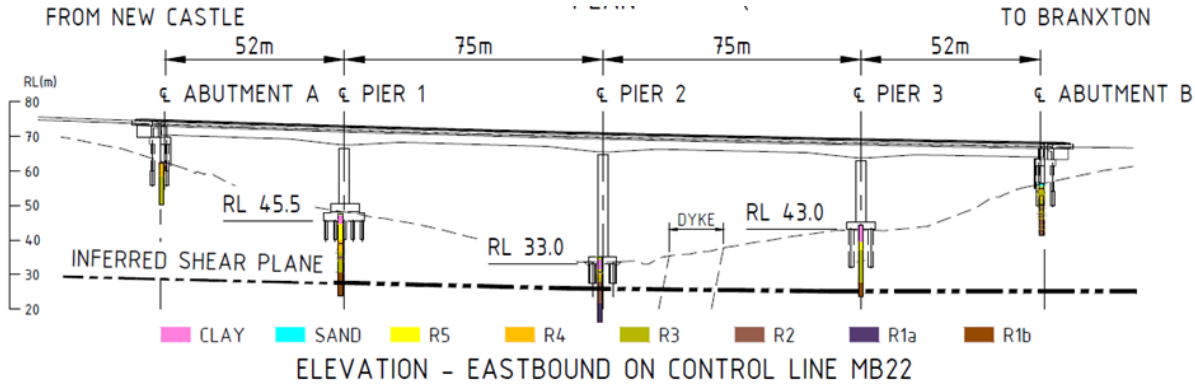

Figure 2. Geological cross-section with bridge shown

Table 1. Adopted geotechnical parameters for different class of rock

Rock Class	Uniaxial Compressive Strength Range		Design Mass Modulus		Design Side Shear Resistance		Ultimate End Bearing Resistance		Allowable End Bearing Resistance	
	qu, (MPa)		Ed, (MPa)		fs, (MPa)		qbu, (MPa)		qba, (MPa)	
R5	0.5	1	100	150	0.14	0.3	1.25	2.5	0.5	1
R4	1	2	150	200	0.3	0.4	2.5	5	1	2
R3	2	7	200	400	0.4	0.8	5	17.5	2	7
R2	7	20	400	675	0.8	1.6	17.5	50	7	20
R1	20	30	675	825	1.6	2.4	50	75	20	30

4 OLD MINE WORKING SUBSIDENCE EFFECTS

The alignment of Bridge BW10 (Viaduct 2) is traversing over a natural valley with the proposed road surface reduced levels varying between RL67m to RL74m, and the valley floor being around RL33m, as shown in Figure 2.

Bridge BW010 (Viaduct 2) alignment, as shown in Figure 3 and Figure 4, is one of the three viaducts, underlain by the old coal mine workings and subject to mine subsidence effects in the future. The depths to the mined coal seam below the bridge foundation is varying between 65m to 110m as shown Figure 4 (Pakharel et al, 2012). To limit the vertical settlements of the proposed bridge structures and foundations the horizontal extent of grouting beyond the footprint of the structure for old mine working in plan, as shown in Figure 3, was taken to be greater than half the overburden depth for this project.

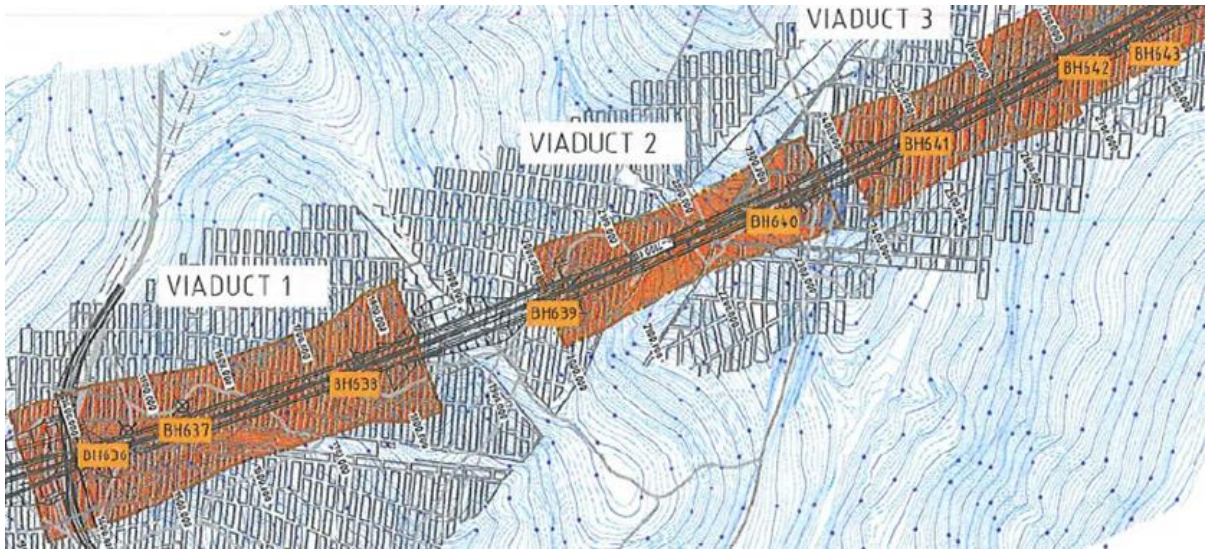

Figure 3. Extent of grouting and old mine working layout below bridges

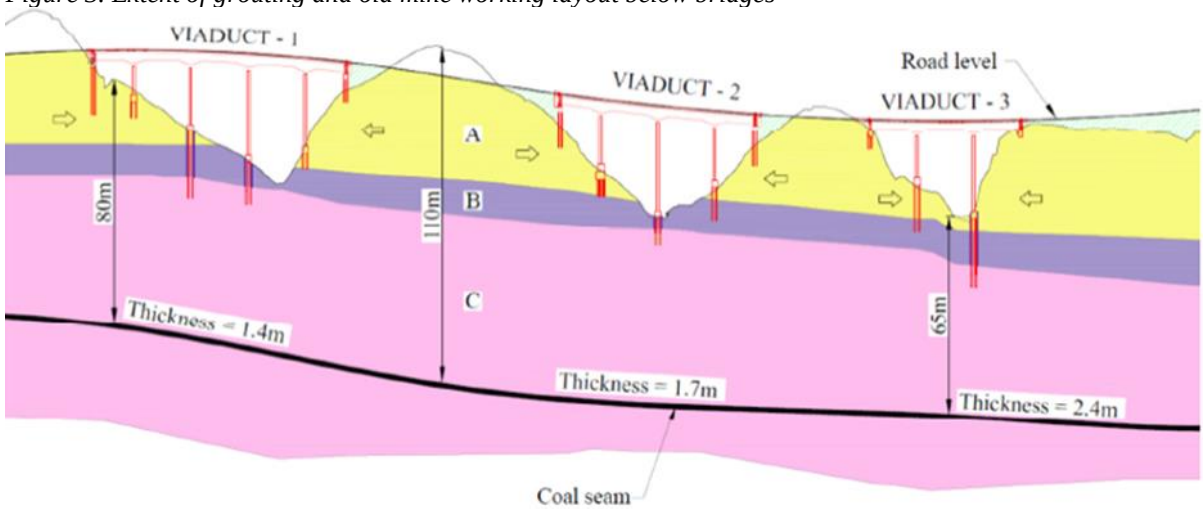

Figure 4. Elevation of proposed viaducts and depth to coal seam (after Pakharel et al).

After grouting the old mine voids, the predicted maximum lateral ground movement anticipated is approximately 100mm due to the slope terrain movement induced by far field old mine working collapse. It has been advised that this 100mm lateral ground movement is independent of the mine void grouting and is expected to occur through the identified "weak layer (Unit B)" at a depth of 2m to 10m below the valley floor , as shown in Figure 4. The global direction of the lateral movement occurs down the slope towards the valley floor.

Vertical differential settlement between adjacent piers will depend upon the spacing of piers and other ground conditions, including the subsurface rock profile. Vertical settlemetn due to far field old mine working collapse after ground treatment at BW10, as shown in Figure 3, has been advised to be a maximum vertical upward movement of 25mm at Pier 2.

5 BRIDGE FOUNDATION DESIGN

5.1 Structural Design Criteria

The design criteria for the bridge structure are as follows: 1) Differential vertical movement between adjacent piers is not greater than 25 mm; 2) Tilt of foundation in any direction is to be 1 in 1000; 3) No horizontal ground movement for Abutments and Pier 1 and Pier 3; and 4) A horizontal ground movement of 100mm and the upside movement 25mm for Pier 2.

The critical design cases were "loss" of segment during construction and the seismic load case and post failure deformation to ensure the bridge foundation will not require major repair works in the event of earthquake and rock pillar collapse of old mine workings beyond the grouted zones.

5.2 Bridge Foundation Design

The original design of Bridge WB10 was separate twin viaducts, with one eastbound and the other westbound, each bridge to be founded on piles. The abutments and Piers 1, 2 and 3, as shown in Figure 5, were all to be supported by pile foundations except Pier 1 is a piled raft. The piles at Pier 2 at the bottom of the valley were provided with oversized steel casing to provide adequate annulus void of 100mm for future movements induced by valley closure associated with far field pillar collapse of old mine workings beyond the grouted zones.

The Pier 1 was originally supported by two separate piled foundations, with each being formed by 6 number of 1.8m diameters piles of about 15m length down to about RL30m, which is above the identified shear plane. Following the review of the additional borehole information obtained during construction stage, the originally design using piled foundation would not be feasible due to the likely shearing movement along the extremely weathered zone (partly fissile) around RL40m, resulting from the potential valley closure movement induced by rock pillar collapse of old mine working. The geotechnical team was requested to investigate the possible options to deal with the potential large shear at this depth.

Three options of alternative foundations were investigated, including pile foundation with enlarged sleeved casing, massive concrete caisson of about 6m depth and piled raft. The caisson option was not considered practical due to the greater depth of excavation, the substantial amount of temporary excavation support and disposal of large volume of soil and rock spoil as well as environmental impacts on the existing conditions. The pile foundation with permanent casing was not considered due to access to the pier location and long-term maintenance issue. At the brainstorming workshop the piled raft was preferred both from design and construction perspectives.

6 PILED RAFT CONSIDERATION

6.1 Concept Design Development

The primary design criterion for the piled raft foundation is to achieve the acceptable horizontal movement at the raft level and the rotation of the raft such that the predicted horizontal movement at the bridge deck level will be within tolerable limit for the fixed bearing point under earthquake loading condition.

The original designed pile caps were separate for the viaduct piers of eastbound and westbound bridges, with the concrete for the pile caps having a characteristic strength of 50MPa. The new pile caps were structurally connected to a common reinforced piled raft of 12m x 25m x 2.5m founded on R5 rock, as shown in Figure 6 and Figure 7. The piled raft with the top level at RL48m is supported by a pile group of five rows of nine piles with a little socket within R4 rock.

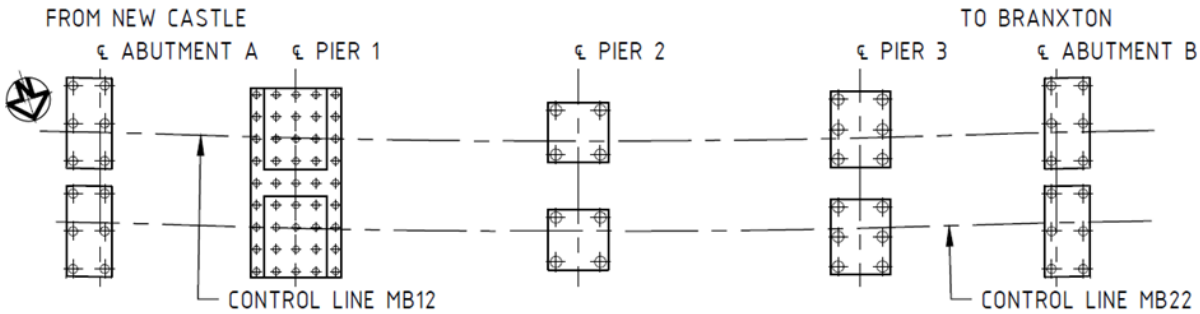

Figure 5. Layout plan of bridge foundations

All piles are 900mm in diameter, 5.5m long, made of reinforced concrete with characteristic strength of 40MPa, with the cut off level at RL45.5m and the toe level at RL40m. The pile centre-to-centre spacing is 2.6m in longitudinal direction and 2.9m in transverse direction. All piles are expected to be through R5 rock with tips being slightly keyed in the underlying R4 rock while pile heads are connected to the underside of the reinforced pile raft at RL45.5m. This option is defined as the base case for the modelling and assessment.

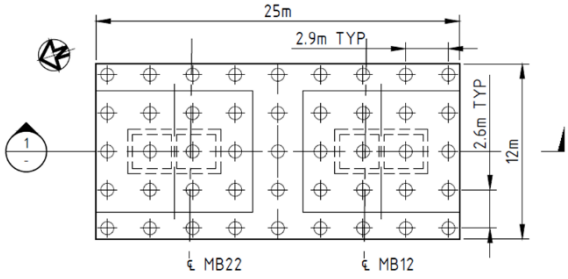

Figure 6. Layout plan of piled raft foundations

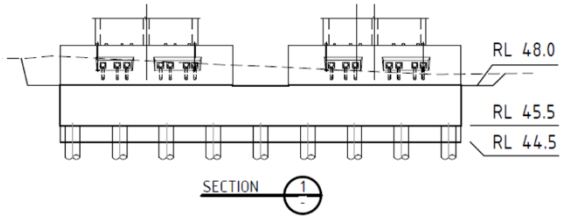

Figure 7. Piled raft plan and transverse cross-section

6.2 Methods of Piled Raft Analysis

The initial analysis of the raft only resulted in excessive rotation of columns supporting the bridge due to presence of the hard clay and very low strength rock (R5) immediately below the raft between RL44.5m and RL45.5m. Subsequently a preliminary analysis was carried out using Poulos-Davis-Randolph method (Poulos, 1980, 2001 and Randolph, 1994) for load distribution and hand calculations. The assessment indicated that it is likely that the rotation of the piled raft will be within the required range of the allowable movement at the bridge deck level. The structural loads acting on the top of each of the two pile caps are: vertical load = 22MN, horizontal shear = 8.5MN and bending moment =145 MNm.

Figures 8 show the Plaxis 2D model adopted for piled raft analysis. Part of the pile cap was modelled as concrete block with an unconfined compression strength (UCS) of 50 MPa while the concrete UCS of the raft and piles were modelled as 40 MPa reinforced concrete. The pile was modelled using a pile element. The loading from the two pile caps were smeared according to the pier spacing and the loadings. Note that the Plaxis 2D model also considered the 1m thick lean mix concrete for replacemtn of the hard clay immediately beneath the raft.

Figure 9 presents the material model in Plaxis 3D for the piled raft foundation. The piled raft was modelled by a concrete block for raft with the piles being simulated as pile elements. There was a layer of 1m thick lean mix concrete below the base of raft. This lay was considered necessary to achieve a relatively R5 founding material for raft to minimize its rotation.

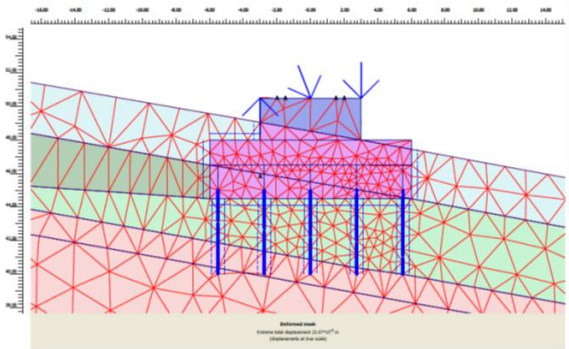

Figure 8. Piled raft Plaxis 2D model

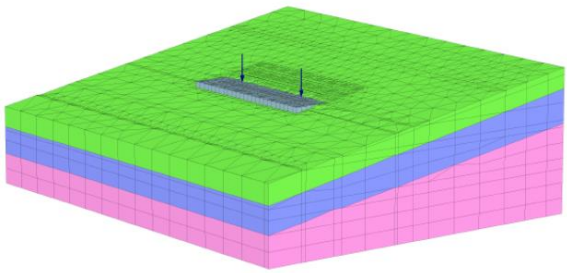

Figure 9. Piled raft Plaxis 3D model

6.3 Results of Piled Raft Analysis

Results of further analysis using program Plaxis 2D suggested that the rotation and settlement of the piled raft would satisfy the settlement and rotation limits under the ULS cases. To have a full confidence in the final design Plaxis 3D analysis was also undertaken. The key output parameters of piled raft from both Plaxis 2D and Plxis 3D are presented in Table 2.

It can be seen that the calculated maximum vertical displacement of piled raft is ranging from 16mm to 18mm while the calculated lateral displacement is approximately 15mm for both Plaxis 2D and 3D analyses. The calculated rotation difference between the two analyses was less than 2.5%. The calculated axial load from Plaxis 2D is 2328 kN which is higher than 2090 kN from Plaxis 3D. As expected, Plaxis 3D would yield more uniform distribution of pile axial load. Similarly, the calculated tension from Plaxis 2D is higher than that from Plaxis 3D, indicating more uniform distribution of load among the piles. The calculated maximum shear force from Plaxis 2D is 513 kN which 53% higher than that from Plaxis 3D while the maximum bending moment from Plaxis 2D is 28.1% higher.

Sensitivity analysis results using Plaxis 3D for the founding material of R4 and R5 rock at the toe of piled raft are found to be acceptable. It is noted that the calculated movements and bending moment in pile would increase by approximately 11% to 13% while the axial compression, tension and shear would decrease by about 5% when founded on R5. From a practical engineering perspective this level of increases or decreases are reasonably acceptable.

Figure 10 presents the vertical stress contour from Plaxis 2D analysis, with a maximum vertical stress at the toe of piles being about 780 kPa, which is even within the allowable bearing capacity of R5 sandstone.

Table 2. Summary of key parameter output from Plaxis 2D and Plaxis 3D

Key Output Parameters	Plaxis 2D	Plaxis 3D	Changes
Maximum Vertical Piled Raft Displacement (mm)	16.7	18.1	+8.4%
Maximum Lateral Piled Raft Displacement (mm)	15.1	14.6	-3.3%
Rotation of Piled Raft (deg)	0.120	0.123	+2.5%
Maximum Axial Compression Load on Pile (kN)	2328	2090	-10.2%
Maximum Axial Tension Load on Pile (kN)	882	735	-16.7%
Maximum Shear Force on Pile (kN)	513	241	-53.0%
Maximum Bending Moment in Pile (kN)	374	269	-28.1%

It was concluded from the above analyses that the proposed piled raft foundation is adequate for the loads on the Pier 1 when the pile toe is founded on R4 rock.

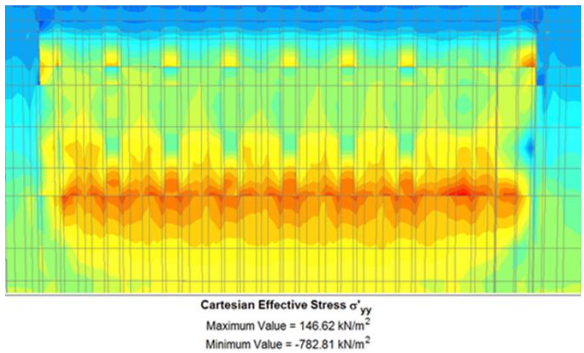

Figure 10. Vertical stress contour of piled raft

7 CONCLUSIONS

The Alliance team has successfully dealt with design and construction of bridges crossing steep valleys subject to future mine subsidence induced horizontal movement and upward movement at valley floor. The unexpected geological conditions at Pier 1 of Bridge BW10 revealed during construction was managed by the means of piled raft foundation. It was found the Poulos-Davis-Randolph method was useful in the concept design development. The numerical analyses using 2D and 3D modelling has confirmed the piled raft was achieving the design criteria. The results of Plaxis 2D and Plaxis 3D are comparable except for the shear and bending moment in the piles from a geotechnical perspective. The use of 1m thick lean mix concrete during pile raft construction enabled mobilisation of raft contribution while the piles are kept within the capacity. The bridges were successfully constructed, and the monitoring results indicate movements are within the predicted.

8 DISCLAIMER

The author, contributors and their respective organisations do not make any representation or warranty as to the accuracy, completeness or suitability or otherwise of the information contained in this paper and shall have no liability to any person in connection therewith.

9 ACKNOWLEDGMENTS

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