Foundation Design Challenges at Hunter Expressway Alliance Project in Australia

H. H. Zhang and B. Choi

Abstract

The Hunter Expressway is a road infrastructure project that will provide a 40 km four lane carriageway between the F3 Interchange at Newcastle and the New England Highway at Branxton, New South Wales Australia. It 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 constructing one of two sections consisting of 13 km of new freeway and local road adjustments. Eight bridges (including three viaducts) are affected by mine subsidence due to past mine workings and future mining works. This chapter describes the foundation design challenges due to subsidence movements and innovative engineering solutions at one of the viaducts. In particular, the subsidence induced horizontal movements, strategies for managing subsidence risks, design methodologies for piled foundations and superstructure to accommodate the anticipated subsidence movements will be discussed. Rock socketed pile design using the Rowe and Armitage Method and pile group analysis will also be covered.

Keywords

Piled foundation • Mine subsidence • Rock socketed piles

1 Introduction

The Hunter Expressway is a new highway consisting of 13 km of new freeway construction with various local road adjustments, including 20 new bridges and three culverts to provide crossings across major valleys and creeks. Figure 1 shows the route of the Hunter Expressway.

Of the 20 bridges, eight bridges including three viaducts will be constructed in areas of prior mine workings and proposed mining works. Thus, one of the major issues and significant challenge of the project is to ensure the foundation of the bridges accommodates the anticipated mine subsidence effects.

This paper will present the design of one of the viaducts, BW010, a twin bridge with 3 piers and 4 spans. The foundation issues due to the mine subsidence effects, the innovative foundation designs and the outcome of the design.

2 Bridge Design Loads and Articulation

The design loads associated with the bridge structure consists of the design vehicular loading as specified by the Scope of Works Technical Criteria (SWTC) and in accordance with the Australian Standards AS5100. Construction loading was considered along with temporary loading associated with the delivery of precast segments and the launching of the erection gantry.

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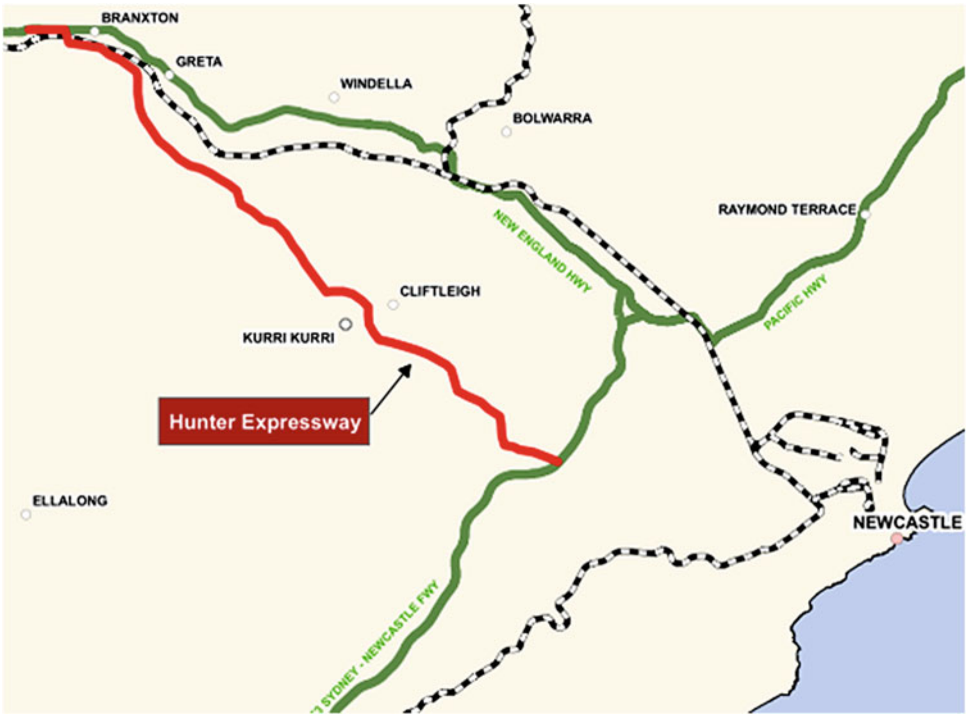

Fig. 1 Hunter expressway plan view

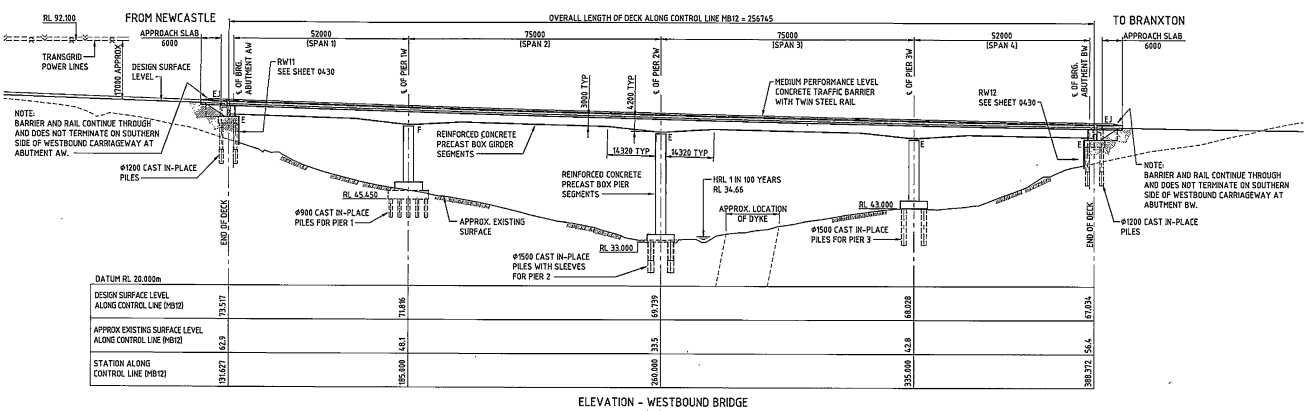

Fig. 2 BW010 cross section view

BW010 is classified as a Type III bridge for earthquake design in accordance with AS5100. The bridge earthquake design is categorised as BEDC-3, based on the acceleration coefficient and the site factor. Based on the SI data available and in accordance with AS1170.4, a site factor (S) of 1.0 is adopted for earthquake design and a ground acceleration coefficient (α) of 0.11 for the design of the structures. No SLS EQ load case was considered for the design of the structures.

BW010 is restrained horizontally by a fixed shear key at Pier 1 and a guided shear key at Pier 3. This allows rigid movement of the bridge deck during a horizontal mine subsidence event resulting in no additional stresses in the deck. All horizontal loads are transferred through shear keys only. All bearings are free sliding, transferring vertical load only (Fig. 2).

3 Foundation Design Challenges

There were two challenges for foundation engineers. The first was soil-structure interaction and the integration of superstructure and foundation design. Structural programs were used to determine the forces and behaviour of the whole bridges under various load combinations; however such programs are not good at soil-structural interaction and

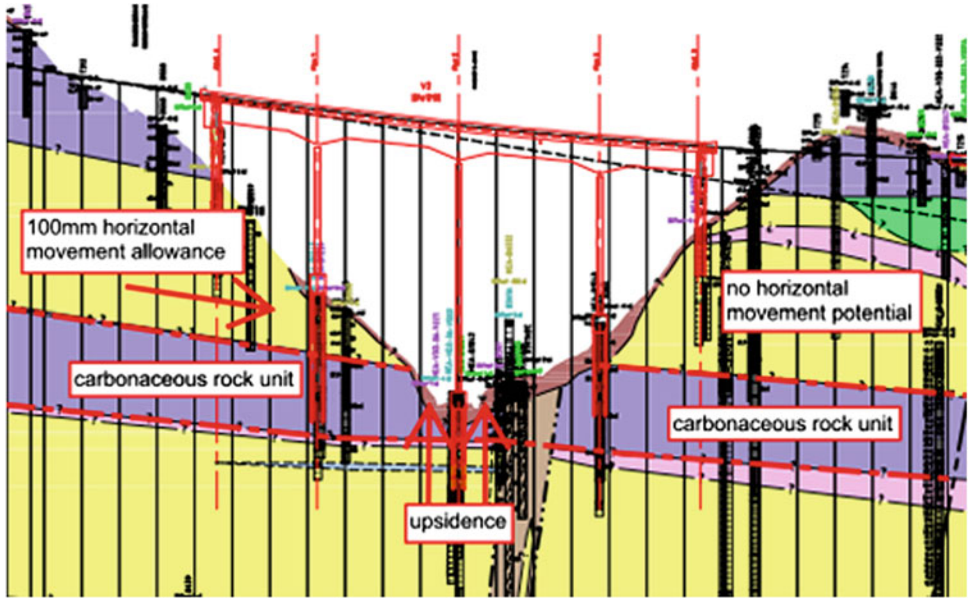

Fig. 3 Horizontal movements and carbonaceous rock unit for BW010

rely on simplified linear springs to simulate this effect. To overcome such limitations, geotechnical programs were adopted to assess soil-structure interaction and estimate the springs for superstructure model. Thus there was a gap between geotechnical analysis and structural analysis.

The second and the biggest challenge was the mine subsidence. Subsidence movements associated with the sudden collapse of old mine workings and proposed future mining have potential to impact on bridge BW010. Following the mine void grouting, the nominated ground movements for BW010 were as follows: (1) Vertical Movement = 25 mm differential settlement; (2) Tilt = < 1 mm/m; (3) Horizontal Movement (Abutment A, Pier 1 and Pier 2) = 100 mm; horizontal Movement (Abutment B and Pier 3) = 0 mm; (4) Upsidence Movement (Pier 2) = 75 mm.

Low strength bedding planes associated with tuffaceous claystone units within the carbonaceous rock unit occurs at and below the base of the valley which BW010 crosses. This carbonaceous rock unit or potential shearing zone is shown in purple in Fig. 3. The strata are dipping to the southwest which equates to a westerly dip along the alignment.

The mechanism and approach to assessing the mine subsidence movements are discussed in SCT Letter Report (2011) and are beyond the scope of this chapter.

4 Innovative Foundation Design

4.1 Soil-Structure Interaction Analysis

Repute 2.0 was adopted to analyze soil-structure interaction at all pier and abutment locations. Pile group effect under combined vertical, horizontal load and bending moment was analyzed for both serviceability and ultimate limit states. Other programs Plaxis 3D Foundation and Piglet were also used to calibrate the results from Repute. These three programs gave comparable results.

To close the gap between geotechnical analysis and structural analysis, the following procedure was adopted: step 1—preliminary soil/rock springs (=similar magnitude of Young's modulus) were used in structural program to determine forces of a pile group. Step 2—these forces were then input to Repute to calculate the pile movements and forces. Step 3—if the difference between two programs was acceptable (within 20 %) then stopped and used the highest loads form two programs to design the individual piles; otherwise repeated step 1 and 2 with springs back-calculated from Repute analysis until the difference was acceptable.

4.2 Individual Pile Geotechnical Design

Once the pile forces were determined, two checks were made: axial and lateral geotechnical capacity.

4.2.1 Pile Socket Design for Axial Load

Rowe and Arimitage (1987) Elastic Method was used to check the rock socket length. The ultimate bearing capacity was also checked against the criteria in AS2159-2009: $\phi_g R_{ug} \geq S^*$, ϕ_g was taken as 0.5 based on AS2159-2009. For piles under tension load, only the shaft resistance was considered; both piston pull-out failure and cone lift-out failures were checked.

4.2.2 Pile Socket Design for Lateral Load

Broms (1964) method was used to check pile lateral geotechnical capacity. Rocks were assumed as clay with ultimate lateral pressure of $9.0C_u$ (or $4.5q_u$), where Cu is undrained shear strength and q_u uniaxial compressive strength of intact rock.

4.3 Pile Solution for Mine Subsidence

The strategies adopted for managing the subsidence risk included both mine fill and bridge design components. The bridge structures have been designed to accommodate the low levels of vertical subsidence that could not be prevented by mine filling alone and the potentially horizontal ground movements caused by pillar collapse at adjacent areas.

The design philosophy for bridges subject to horizontal movement was to provide a flexible structure with the foundations isolated from the horizontal ground movements either through double-sleeving of piles that penetrate through the shear zone or footings founding above the potential shearing zone. The double-sleeving of piles provides an annulus of free space between the outer and inner sleeves to accommodate potential horizontal movements across the basal shear plane.

4.3.1 Pile Raft Solution at P1

The design of the foundation for Pier 1 consisted of a combined piled raft foundation that terminated above the potential shear zone. The 25 × 12 m thick raft slab is sat on a grid of 45, 0.9 m diameter piles and a 1.0 m nominal thick plain concrete layer. The piles were founded on the top of the potential shearing zone and on R4 rock. The raft slab was founded on plain concrete which transfers the compressive bearing stress through the residual soil layer to the top of the R5 rock. FE program Plaxis 2D and 3D were used to analyze the piled raft behavior and individual pile forces.

4.3.2 Sleeved Pile Solution at P2

Double sleeved piles were adopted for the P2 foundation to avoid the potential horizontal and vertical subsidence movements. P2 piles were founded in the R2 layer below the identified coal seams. The pile cap soffit level was located above the finished ground level to cater for potential upsidence movement.

4.3.3 P3 and Abutment B Piles

Pier P3 piles were founded in R2/R3 layer within the coal seam strata with no sleeves. At this location, evidence from the mine subsidence investigation and mine void grouting indicated that there was no potential for ground movement.

4.3.4 Abutment A Piles

The piles of this foundation were founded above the shear plane and not sleeved. It was assumed that during mine subsidence event, piles and pile cap would move with the surrounding ground.

4.3.5 Monitoring of Mine Subsidence

The monitoring system for double-sleeved pile foundations included inspection pipes installed within the pile caps to allow the inspection of piles sleeves via an endoscope camera. Inclinometers were installed at pier locations through the anticipated zone of shearing to monitor the horizontal movements.

5 Conclusion

A case history of bridge foundation design for mine subsidence is presented. Innovative engineering solutions for pile foundations to accommodate mine subsidence are presented: P1 piled raft foundation that terminated above the potential shear zone and double sleeved piles at P2 to avoid the potential horizontal movements. The incorporation of monitoring into foundation designs will permit detection and an appropriate response to movements should they occur.

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