Building New Bridges on the Hunter Expressway over Abandoned Coal Mines

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ABSTRACT

The industrial development of our societies over the past century used coal as the main source of energy which was mined from deep underground seams leaving voids below the ground surface. New urban development and transportation networks planned to meet the demand of future generations require roads and bridges to be built over these historical mining areas. Substantial mine related ground movement has been observed at the surface level above mining areas where standing pillars have become destabilised as a result of rising water levels within the mine, earthquakes, adjacent mining activity and the effect of sustained load in its supports. Building new bridges in such areas is a challenge for engineers.

This paper discusses the details of three prestressed concrete segmental balanced cantilever bridges having a combined total length of 850m with internal span lengths of 75m and pier heights varying up to 30m above the ground. Up to 500mm vertical and 450mm horizontal ground movements are predicted in the event of pillar instability in bord and pillar workings in the seams ranging from 65m to 170m below the surface. Ground treatment works have been undertaken to reduce the predicted vertical movements to 25mm while horizontal movements of 450mm remain to be considered in the design. The bridges have been articulated to accommodate the predicted ground movements due to mining subsidence. Various types of foundation have been adopted.

KEYWORDS

Mine Subsidence, Viaducts, Bridges, Pile Sleeve, Pile Raft, Horizontal Shear Plane, Coal Seam

INTRODUCTION

The Hunter Expressway road project is located near Newcastle in New South Wales. A new 40km long, four lane road is being built from the F3 Freeway near Seahampton to Branxton. The Hunter Expressway Alliance, HEA is currently planning, designing and building the eastern 12km part of this project from F3 to Kurri Kurri. The project requires the construction of many bridges including three major twin viaducts. This project site is located in one of the major coal deposits in Australia as shown in Figure 1[1]. There are records of abandoned coal mines under these viaducts. Seaham No 1 Colliery mined in the area between 1890 and 1932 using bord (open voids) and pillar mining methods with full pillar extraction in some areas.

In those areas where the voids are left open, the pillars may deteriorate over time as a result of rising water levels within the mine, earthquakes, or additional loading associated with adjacent mining. The consequences of such collapse are the ground movements of overlying and adjacent strata directly affecting the structural foundations. Managing the risk associated with such ground movement has been the major challenge for the bridge engineers. There is ongoing mining in the area at some of the deeper seams.

A design guideline prepared by the Department of the Main Road, NSW [1] in 1981 outlines recommendations to deal with mine subsidence effects on bridges. It recommended simply supported beam and slab types of structures with low torsional stiffness preferably supported on elastomeric bearings and foundations without piles. Similar recommendation and guidelines are provided by the Highway Agency of the United Kingdom [2]. The Road and Maritime Services have built many other bridges in New South Wales dealing with similar issues. The majority of these structures are smaller bridges with a few major structures like the bridges over Mooney Mooney Creek and the Nepean River. The scale and complexities of the viaducts of the Hunter Expressway are beyond the simple solutions outlined in these guides. New innovative solutions are required incorporating the latest research in mine subsidence observations.

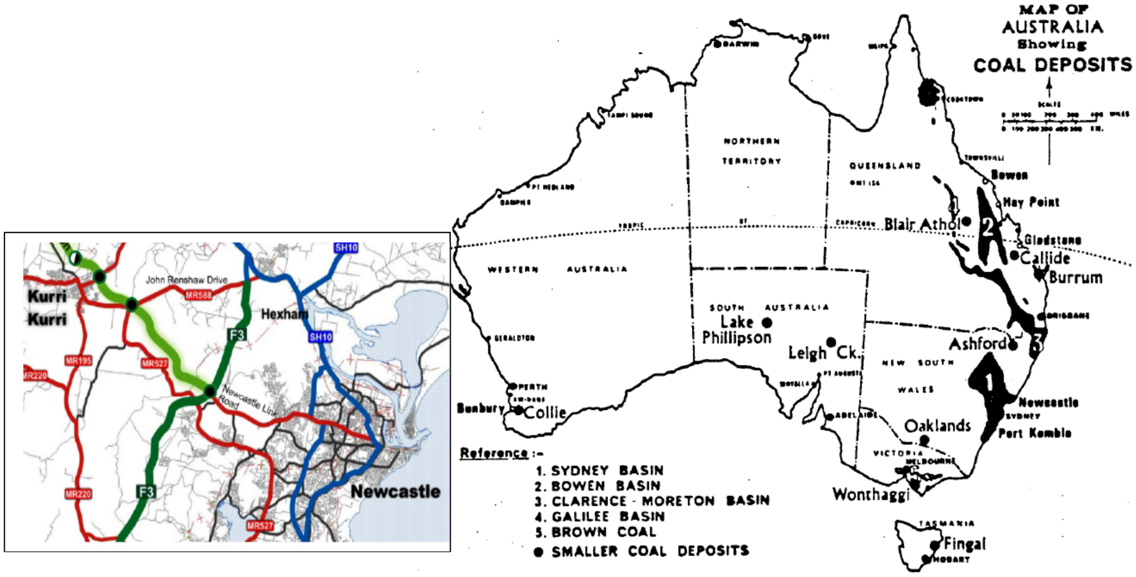


Figure 1. Project site and the Map of coal deposits in Australia [1]

This paper describes the characteristics of the ground movements at the site, and type and features of the bridges adopted in the design to accommodate and mitigate the potential ground movements anticipated. The technique includes a unique bridge articulation system, special types of movement joints and bearings, and a combination of various foundation types.

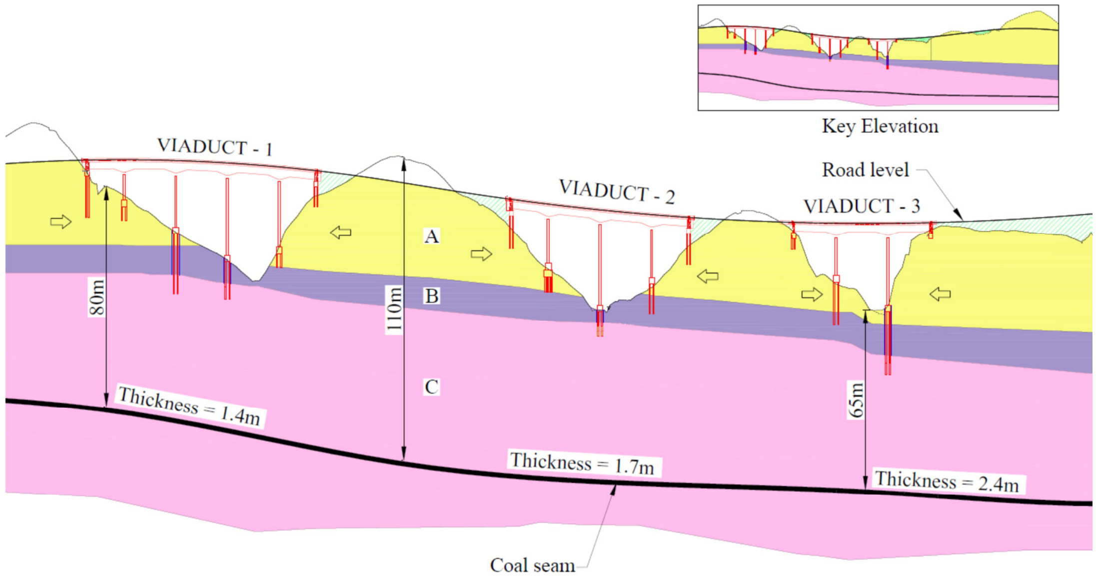


Figure 2. Vertical profile of coal seam and ground surface

SITE CONDITION AND THE ISSUE OF MINE SUBSIDENCE

The longitudinal section of this site in Figure 2 shows the ground profile, road level, and bridge arrangement. In this figure, the horizontal scale is reduced to accommodate all three viaducts. The lengths of these three bridges are 330m, 250m and 200m respectively. The deck level of the viaducts is up to 35m above ground surface. The coal Seam varies in thickness from 1.3m to 2.4m and is located from 65m to 170m below the ground surface in the close vicinity of the viaducts. Record tracings of the mine workings show varying degrees of coal extraction below the viaducts ranging from main heading pillars that are likely to remain stable at least in the medium term through standing pillars in bord and pillar areas, to full extraction with evidence of surface subsidence in other areas. The potential for further ground movement due to the failure of pillars was a major risk for the viaducts.

The maximum vertical subsidence is affected by several factors including the thickness of seam, degree of extraction, depth of seam from ground surface, types of rock above the seam and the location of collapse zone from the point of interest. For the design of viaducts it was estimated to be approximately 500mm.

In addition to the vertical subsidence there is potential for horizontal ground movements to occur within the subsided area but also well beyond the limits of the area that has subsided. These mining induced horizontal movements have been studied in detail around longwall mining operations [4]. They are assumed to be possible in a pillar collapse scenario and will almost certainly be possible under the influence of further mining in the vicinity of the viaducts. There are three main components of horizontal movements each with their own characteristics: systematic movements, horizontal stress relief movements and valley closure movements.

Systematic movements occur in a direction toward the subsiding area as the ground is dragged down into the void created by mining or pillar collapse. These movements occur predominantly within the area where there is subsidence and tend to have a magnitude less than about 150mm. The effect on bridge structures is effectively eliminated by mine filling within a distance of half the depth of the coal seam from the structures on ground surface.

A second component of horizontal movement is caused by stress relief. This stress relief causes low level stretching of the ground, typically less than 100mm over 1km, but can extend for several kilometres from subsiding strata. This component tends to be a characteristic of high stress environments and is not considered significant at the Hunter Expressway site.

The third component, valley closure movement, occurs wherever there is variability in the surface topography within the area that is subsiding and these tend to be significant for the structural design of bridge structures. As the rock strata subside, there is a tendency for the volume of the rock to expand laterally as differential rotation occurs and fractures form. In steep terrain such lateral dilation under the high ground is unrestrained in the direction toward the valley floor. This imbalance causes horizontal movement to occur in a downslope direction with stretching at the top of slopes often causing open cracks and compression or shortening in the base of valleys. Downslope movements around longwall panels are commonly observed to exceed 500mm in steep terrain subject to a metre of vertical subsidence.

Compression in the valley base leads to a phenomenon called upsidence whereby rock strata in the valley floor buckles upward relative to the general downward movement of the subsiding strata. The magnitude of upsidence is typically about half the magnitude of the valley closure.

The displacements observed on the surface are accommodated as shear movements along stratigraphic horizons typically located at or up to about 5-15m below the base of valleys, excluding infill material, depending on the geological setting. An example of a horizontal shear plane generated by subsidence movements is shown in Figure 3 where it has been exposed in a cutting.

Figure 3. Typical horizontal ground movement and shear plane by mine subsidence

The direction of such valley closure movement depends upon the topography of the site and is not necessarily in the longitudinal direction of the bridge. Figure 4 shows the topographical map of Viaduct 3 and the direction of horizontal ground movement anticipated if a subsidence event were to occur under the topographic high ground on either side of the valley. For the three viaducts, horizontal movement is estimated to range from 50mm to 300mm for the 500mm of vertical subsidence expected. This creates movements in both the longitudinal axis and transverse axis of the bridge.

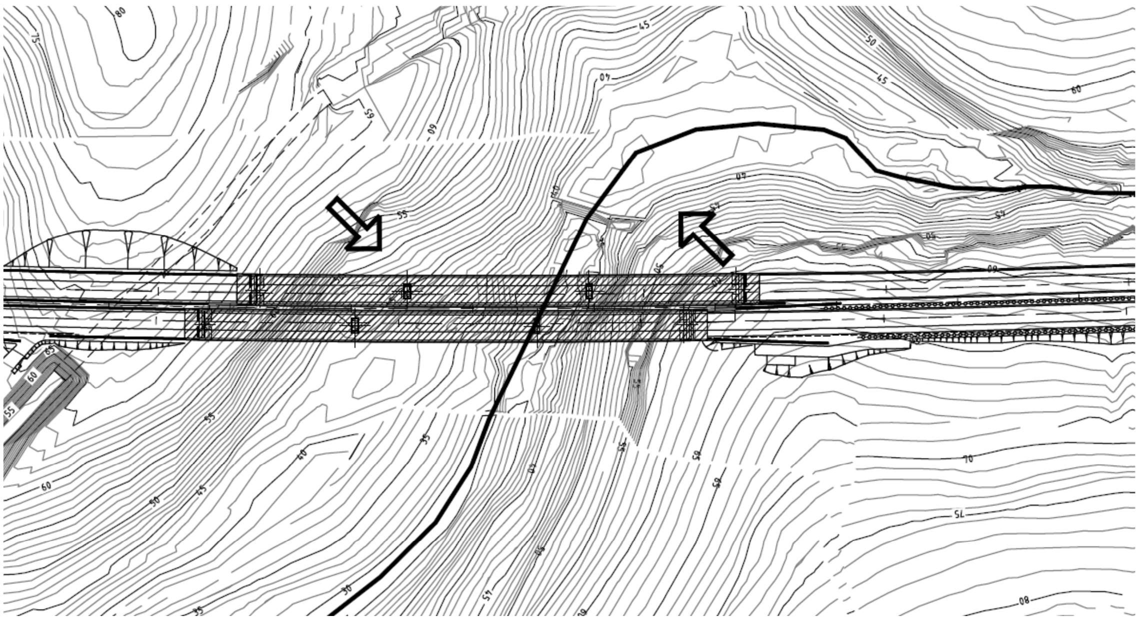


Figure 4. Topographic contours and the direction of ground movement

MINE VOID FILLING MAJOR MITIGATION MEASURE

Various measures have been adopted to mitigate the risk of potential mine subsidence from pillar collapse. The major component of the strategy is to fill up the voids with low strength, cement stabilised fly ash grout. This grout provides lateral support to the existing pillars to eliminate any potential for sudden pillar collapse. Then the vertical settlement over the grout fill is the effect of sustained load on the existing supports. In plan, the mine filling extends to a minimum distance of half the depth of seam layer from the foundation to reduce any potential for vertical ground movements of greater than 25mm. There remains some residual potential for low level elastic compression of the pillars and fill if a pillar collapse occurs outside the fill area, but this level of movement has been accommodated within the structural design of the bridges.

During the process of void filling, the condition of the ground has been assessed using a borehole camera. In some areas, there are standing pillars, in others the roof of the bords has collapsed but the pillars remain stable, and in others the pillars have also been extracted.

DESIGN PARAMETERS FOR THE BRIDGES

The predicted nominal design ground vertical settlement due to mine subsidence for the purpose of differential settlement between adjacent bridge foundations is 25mm which was reduced from 500mm as a result of grouting the mine voids within the zone of influence underneath the bridges.

While the vertical movement is reduced, the horizontal movements are not possible to reduce, as the displacement wave generated by the collapse of the pillars in far the distance will affect the bridge foundation, inducing lateral displacements. Therefore the nominated design lateral displacement remained unchanged which for the bridge in Figure-4 is 50mm on the left hand side of the valley and 300mm on the right hand side.

Tilting of the bridge foundation from its vertical axis at any direction is considered with an angle of 1:1000.

The current bridge design code AS5100.2 [7] requires mine subsidence effects to be checked in the superstructures with a load factor 1.0 for the SLS conditions and the foundations with a load factor of 1.5 for the ULS conditions. The effect of differential vertical settlement and the tilting of ground surface are considered as permanent effects for the assessment of the structural capacity of deck, piers and foundations to comply with the above requirements.

However the effects of horizontal movements are considered as transient effect as the bearings will be reset after the mine subsidence. The horizontal movement capacities for joints and bearings are determined using various combinations of thermal, earthquake and horizontal movement from the mine subsidence using different factors for three levels of limit state conditions: namely SLS, ULS and worst credible event(WCE).

The first level is the normal SLS conditions in which the nominated horizontal movements along the nominated direction are accommodated. In this case the bridge remains operational with minor distress in major elements and repairable damage may occur in minor elements like barrier joints. The second level is the managed ULS condition that may require significant repair including major elements such as the deck, lifting and adjusting bearing levels, replacing barriers and expansion joints. The bridge however can be readily returned to service. The third level where larger than expected mine subsidence movement occurs; a worst credible event the bridge must sustain without collapse and loss of life.

The factors adopted for the nominated horizontal movements due to mining subsidence are 1.0 for serviceability limit state condition, 1.2 for the managed ultimate limit state conditions and 1.5 for the worst credible event. Similarly potential variation in the direction of movement is ±30 degrees for serviceability limit state conditions and ±45 degrees for the ultimate limit state conditions.

SELECTION OF BRIDGE TYPE

During the early stage of the project parallel studies were undertaken for selecting the type of bridge and assessing the magnitude of mine subsidence. Table-1 below summarises the major types of bridges and span lengths considered while exploring the most suitable economical option. The first assessment was to select a suitable superstructure type to accommodate the vertical differential settlement. A multi-criteria selection process was adopted to compare various options in the preliminary study.

Two options were selected for detailed further analysis in the next stage. Option-1 is a single cell concrete box girder with internal span length of 75m to be built using pre-cast segmental balanced cantilever methods and erected using an overhead truss. Option-2 was an Incrementally Launched Box Girder with span length of 46m. Among the other options considered, short span deck solution consisting of multiple girders supported on elastomeric bearings had been considered undesirable due to the need of substantial maintenance work in the future associated with the bearings and joints and large capital costs to build the substructure. Long span continuous deck solutions using cast-in-situ balanced cantilever methods of construction resulted in large superstructure construction cost by the procurement of erection equipment to complete the project within the allocated time.

Continuous deck girders are preferred for riding comfort. The match cast balanced cantilever bridge option was adopted as the longer spans minimised the number of piers and better accommodated mining subsidence movements. The total cost of the preferred option was about 10 percent lower than the other option.

Table 1. Type of bridges considered for Hunter Expressway viaducts

Bridge Types	Span Lengths																		
	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120
Cast-in-situ BCM											■	■	■	■	■	■	■	■	■
Precast segmental BCM						■	■	■	■	■									
Incremental Launched Box Girder				■	■	■													
Simply supported Concrete Girder				■	■														
Super-T girder	■	■																	
Simply Supported Steel Box Girder					■	■													

STRUCTURAL ARRANGEMENT AND ARTICULATION OF THE BRIDGES

Each bridge superstructure is a continuous girder between two expansion joints located at each abutment. The span length was dictated by the typical size of overhead erection gantries and maximum segment weight. The longer span length was preferred to minimise the effect of vertical differential settlement and minimum construction cost of substructures. Vertical load is transferred through two free sliding spherical bearings at each pier or abutment. Longitudinally the deck is connected at one pier and transversely at two piers irrespective of their total length. That means there are two shear keys for each bridge deck, one fixed and another guided. A typical support arrangement with a guided shear key is illustrated in Figure 5. Such an arrangement allows all lateral movements including the tilting of the ground from mine subsidence to be accommodated with minimal stress on the superstructure as the displacement is transferred by the rigid body motion.

Figure 6 illustrates the effect of horizontal ground movement for Viaduct-3 indicating the location of the deck in its as-built position and its position after the ground movement. In plan lateral movement of the deck with respect to the substructure is shown which is zero at Pier-1 and at Abutment-B because of the shear keys. At Abutment-A and Pier-2 there is a transverse shift of the deck with respect to its support which has been considered for the movement capacity of bearings and expansion joints. Modular type expansion joints can accommodate large transverse movements but the barrier rail attached to the parapet spanning the joint will break easily. This creates misalignment of the parapet and barriers at the Abutments therefore there may be a need to break the parapet, widen the approach slab and rebuild the barrier at new position after a significant mine subsidence valley closure event. Mine subsidence is a once in a life time event for the bridge and therefore, if mine subsidence occurs, the risk event is passed and will never occur again. Such repairs are considered minor and acceptable as normal operation of traffic is possible with reasonable controls at the repair site.

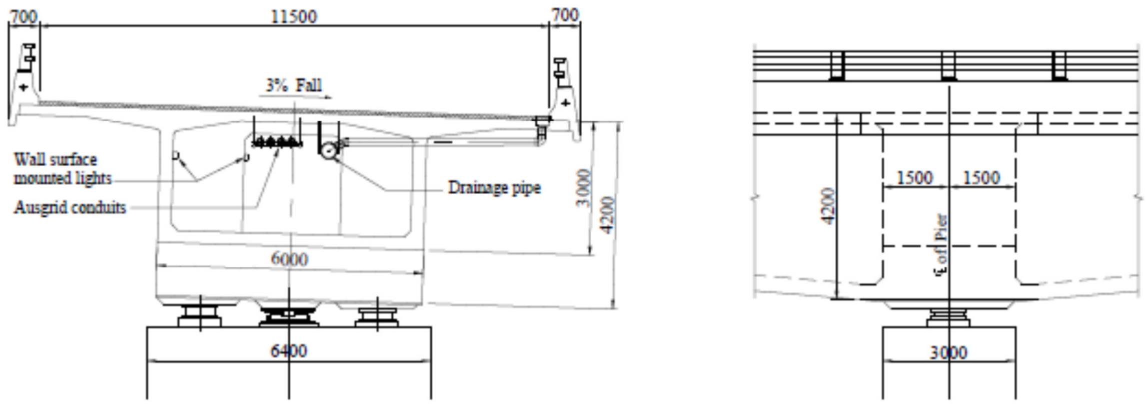


Figure 5. Support arrangement of bridge on piers

In elevation, longitudinal movements have been indicated which can be accommodated in the bearings, guided shear key and expansion joints as done normally in other bridges.

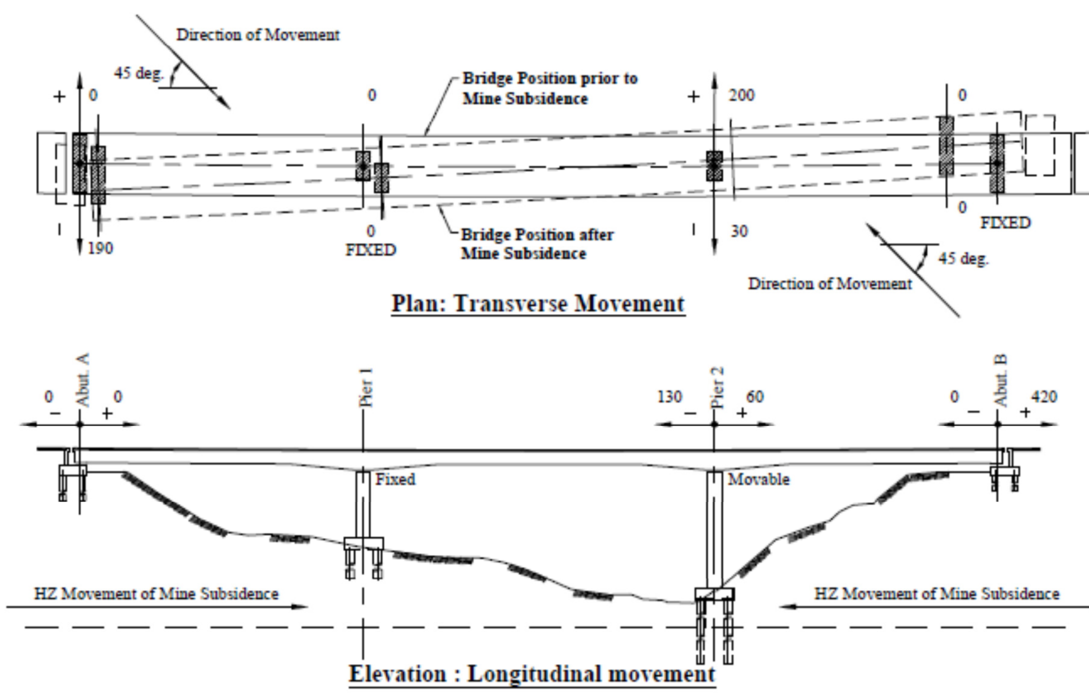


Figure 6. Movement of deck due to mine subsidence effect

A typical section of the spherical bearings is shown in Figure 7 which can accommodate lateral movements and tilting of the foundation without affecting the superstructure. It has a larger contact pressure between sliding plates compared to other types of bearings and, thereby requires relatively smaller attachment plates to accommodate the same movement. The fixed shear key shown in Figure 8 has a cylindrical steel piston attached to the deck and a female part on the pier head. This allows the transfer of horizontal force from deck to pier without transferring bending moment. The detail of the guided shear key shown in Figure 8 is different from that of the fixed shear key. A circular disk of PTFE surface is included on the bottom surface of the piston which also incorporates a guide bar connecting to a rigidly fixed attachment plate. This arrangement allows angular rotation of the deck in plan without creating any bending stress in the deck to suit the rigid body movement shown in Figure 6.

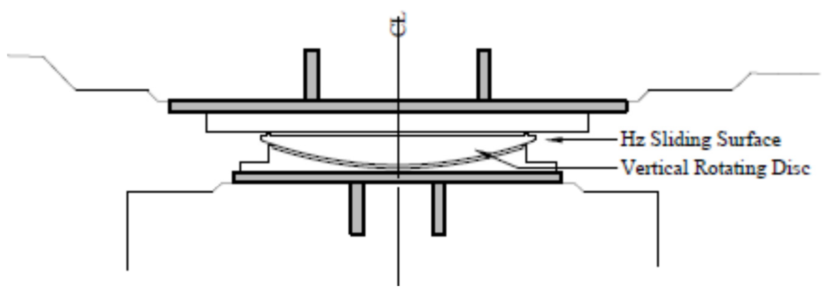
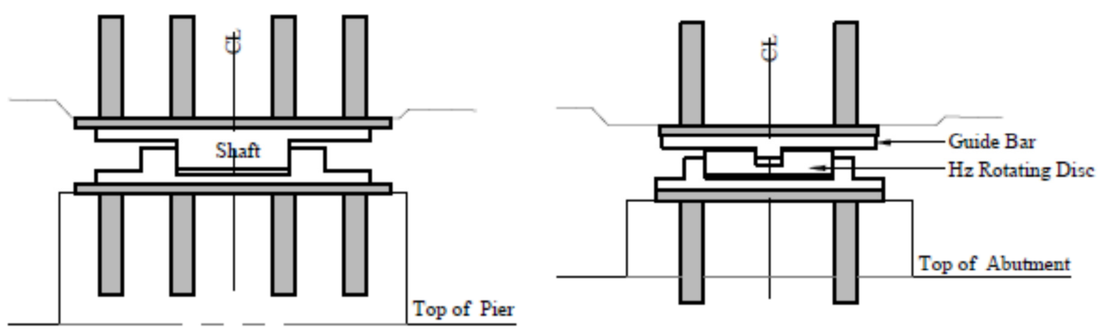


Figure 7. Typical section of free sliding Spherical Bearing



(a) Fixed Shear Key (b) Guided Shear Key

Figure 8. Typical sections of Shear Keys

The piers of these bridges are built using precast concrete segments with epoxy joints. The design procedure and details dealing with earthquake behaviours are given in reference-6.

SUPERSTRUCTURE DETAILS

The bridge superstructure is a single cell concrete box girder of 12m width which will initially carry two lanes of traffic. It can also accommodate three lanes of traffic as a provision for future widening of the Expressway to meet traffic growth. The depth of the box girder varies from 3m at the mid span to 4.2m at the support with a haunch at the quarter span as represented in Figure 2 and Figure 9. It will be built by precast segments with epoxy joints using the balanced cantilever method of construction by overhead gantry.

The post tensioning details consist of three types of tendon arrangements. Figure-9a and Figure 9b depict typical elevations for an end span and an internal span respectively. The internal tendons embedded inside the concrete of the top flange are mainly governed by the requirements for the erection of the segments. The internal tendons in the bottom flange of the box girder and the external tendons anchored at the diaphragms and deviators are governed by the requirements for the bending capacity at SLS conditions. Continuity of the tendons is maintained between expansion joints both at top and bottom flange to give extra robustness for the box girder. The external tendons are replaceable and the internal tendons are sufficient to maintain the necessary structural capacity to support the self-weight of the deck during the replacement of external tendons. The use of external tendons also significantly reduces the shear force carried by the webs.

In the case of excessive differential settlement, more than that predicted in the mine subsidence criteria, the superstructure maintains its stability. After the mine subsidence the position of the box girder can be adjusted by changing the level of the bearings and external tendons can be replaced to restore full capacity of the deck. A bridge monitoring plan and trigger levels for activating maintenance operations have been developed.

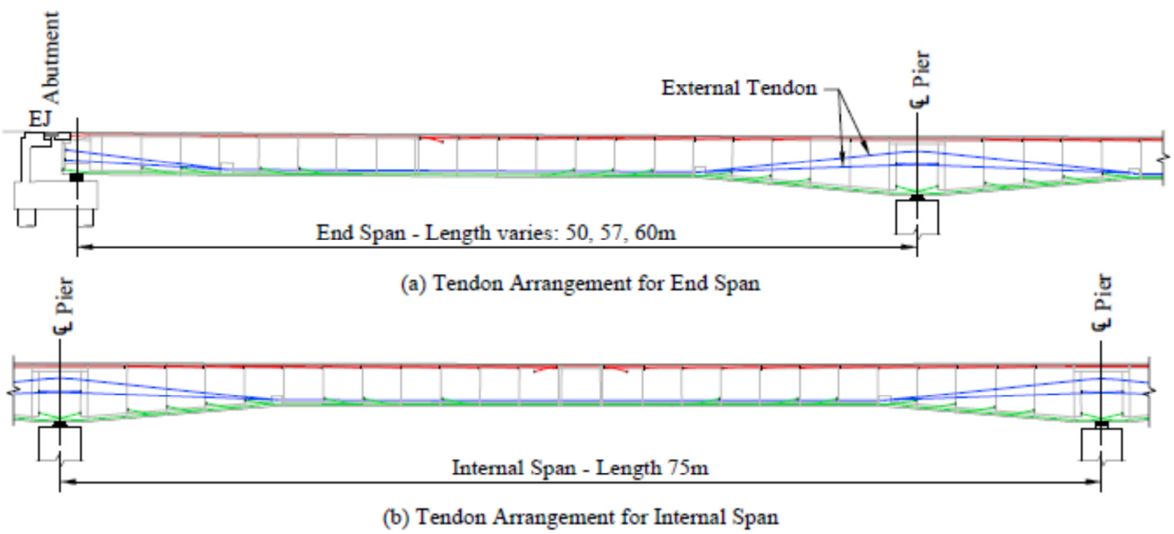


Figure 9. Typical tendon arrangement for deck

SUBSTRUCTURES AND FOUNDATIONS

The ground surface movement due to mine subsidence as discussed above and illustrated in Figure 2 has critical significance in selecting the type of foundation. The horizontal movement due to mine subsidence forces the soil mass A to slide over the mass block C somewhere at the weakest plane in layer B. A horizontal sliding shear plane is formed. Conventional reinforced concrete piles, if they cross the horizontal sliding plane, cannot resist the load induced by the horizontal ground movement. Therefore the design guides [1] generally recommend spread footing type foundations where possible. The problem with spread footings at the Viaduct locations is that the bearing pressure and settlement could not be managed by a reasonable size footing for the loads imposed. In difficult terrain like this there are cost, access and environmental impact issues which limit the economical size of excavation.

In Figure 2 we can identify three types of situations affecting the foundation. First is the case of conventional pile foundation where the thickness of rock mass A is sufficient for the toe of the pile to be significantly above the possible location of the horizontal shear plane as is case at the abutments and Pier-1 of Viaduct-1. The design of these foundations follows a standard pile capacity computation based on the friction and the end bearing capacity of the soil.

The second type is for those cases where the toe of the piles is just on top of the layer-B as in the cases of Pier-1 of Viadut-2 and Pier-4 of Viadut-1. These piles are not long enough to fully resist the design loads by the conventional pile design method. They are designed as a piled raft where part of the load is taken by the piles and part of the load is transferred to the pile cap by bearing on the soil/weak rock strata at the pile cap soffit. The third type of foundation consists of long piles crossing the horizontal shear plane and avoiding contact with the entire rock mass of layers A and B. These piles have a relatively large base capacity and a friction capacity sufficient for the tension capacity. Separation of the pile from the rock is achieved by casting the pile in a double sleeved casing, one sleeve in contact with the pile and the other in contact with the ground, thus permanently maintaining the required gap between them as shown in Figure 10. Relative position of the casings and the diameter of outer casing were determined by studying the magnitude and direction of the ground movement.

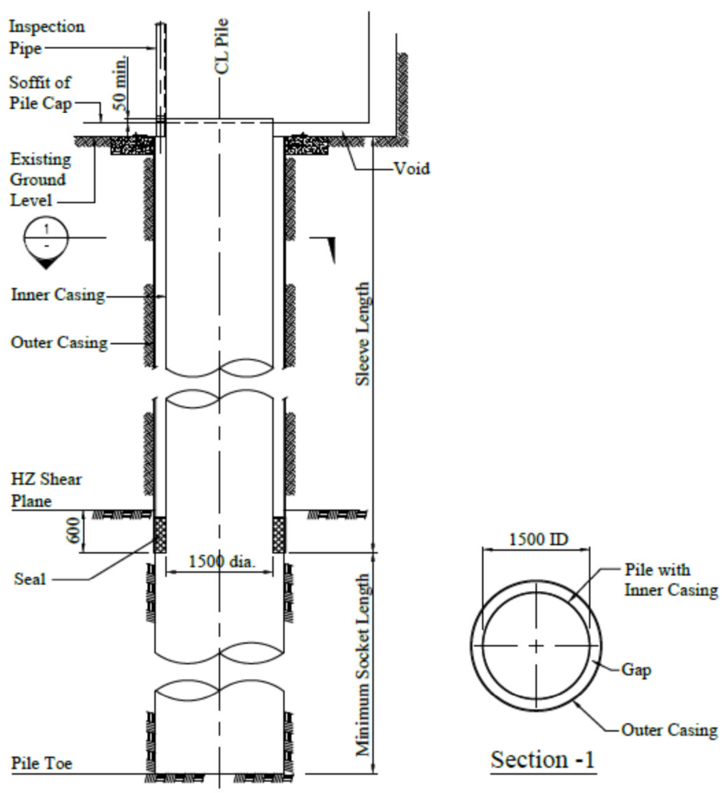


Figure 10. Pile sleeve details

SUMMARY AND CONCLUSION

The effect of mine subsidence is a major risk for the long term performance of bridges of any kind and configuration. It requires special details in the design and construction. There are governmental bodies regulating the consequence of mining subsidence on the infrastructure built in such areas. Predicting the magnitude of ground movements is a separate field of specialization and is a required input for major bridge projects. These movements however, should not prevent building major bridges in these areas. As demonstrated in this project, it was possible to design long span bridges having an internal span length of 75m on the basis of lowest direct cost, by developing a specific articulation and joint system. The project needs a suitable commercial and innovative work environment and to be staffed by appropriately qualified engineers to enable the interplay of data, ideas and solutions from the different interfacing disciplines.

ACKNOWLEDGEMENT

The Hunter Expressway Alliance comprises of Thiess, Hyder, Parson Brinckerhoff as non-owners partners and the Roads and Maritime Services of the NSW government as owner partner. Other companies involved in the project are SCT Operations, BG&E, VSL, Avopiling. Bearings and Expansion joints are fabricated and supplied by Mageba.

The authors feel honoured to be taking part in the opportunity presented by the Alliance in the design of these viaducts from concept stage through to the detailed design and construction phase services. These are now under construction and facing several challenges, with mine subsidence being the major one. A friendly team, innovative environment and interaction with the construction team during design development are greatly appreciated.

Particular thanks to Mark Bennett from Roads and Maritime Services of NSW for providing references of past bridges built in areas of mine subsidence and suggestions during the whole design development of this project.

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