Management of Twin Bridges over Nepean River at Douglas Park During Mining-related Ground Movements

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SYNOPSIS

Twin bridges over Nepean River at Douglas Park are 285 and 236 m long structures supported by piers up to 55 m high.

The owner of the coal mine in the vicinity of the bridges, BHP, extracted coal by a longwall method which resulted in horizontal and vertical ground movements that extended to the bridges. The concerns of the owner of the bridges, RTA NSW, and of the Department of Mineral Resources on the effects on the bridges that are a part of the heavily trafficked arterial road link between Sydney and Melbourne led to the establishment of a comprehensive management system.

The paper explores the interaction between all parties involved and focuses on the effects of ground movements on the twin structures. Comprehensive computer models of the bridges supplemented by spreadsheets allowed regular evaluation of all critical components and detailed weekly reporting. The paper highlights the performance of critical components of the structures.

The mining had ceased mid-Apil, 2000, and the paper also covers three months of post-mining ground movements.

INTRODUCTION

The longwall mining operations of the BHP Coal Division Tower Colliery extend to the vicinity of the twin bridges that carry the South Western Freeway over the approximately 60 m deep gorge of the Nepean River at Douglas Park, some 70 km southwest of Sydney.

The minimum horizontal distance between the bridges and the closest point of the mining was 560 m. The extracted coal seam was approximately 500 m below the surface, its thickness was approximately 3 m, the width of the longwall panel was 200 m and the extraction proceeded at a rate of up to 50 m per week.

It has been recognized that extraction of coal by the longwall method causes disturbances of the surface topography above and adjacent to the zone of extraction. The presence of the gorge introduces horizontal movements beyond the vertical projection of the excavation in addition to the vertical subsidence directly associated with extraction. Prediction methods for vertical subsidence are relatively well developed and the vertical subsidence is readily quantifiable. However, information on the prediction of mining induced horizontal ground movements is limited.

The body of knowledge relating to horizontal ground movements, caused by longwall mining operations, has been greatly enhanced by the monitoring work that has been carried out as longwalls 16 and 17 have been mined and analyses of results. It is intended to include in this paper only these facts regarding the ground movements as surveyed that are necessary to present an overview of the structural performance of the bridge and analytical work that has been carried out to monitor regularly the serviceability of the bridge. Findings and conclusions from this project are likely to be presented in due course at another specialist forum.

Since the geometry of the longwall 16 was identical to that of previously mined longwall 15, it was considered likely that the movements at the freeway bridges would be of the same order of magnitude as gorge movements measured closer to the mine. This likelihood was further supported by the measurements that had earlier been made in the cliffs and base of the Cataract River Gorge (Refer to Figure 1) over longwalls 6 to 14. It was predicted that the closure of the gorge at Douglas Park Bridges would be less than 100 mm and that the differential lateral movements between the piers in the base of the gorge and the bridge abutments would be less than 50 mm. The actual movements to date have not exceeded the predicted movements.

The relatively low reliability of the prediction methods for horizontal ground movements was recognized by the owners of the mine, BHP, in the application for approval to mine longwalls 16 and 17. In addition to BHP's own Assessment Committee, an interdepartmental Douglas Park Bridges Subsidence Technical Committee (TC) was established to provide a review mechanism of the potential subsidence of longwalls 16 and 17 on the bridges. The TC was made up of the principal stakeholders: BHP - mine owner, Roads and Traffic Authority (RTA) - bridge owner, and Department of Mineral Resources, (DMR,) and experts in the fields of mine subsidence, geology, rock mechanics and bridge engineering. John Zavesky, Associate, Cardno MBK was the bridge expert member of the TC, engaged by the mine owner. The stated objective of the TC, chaired by an RTA representative, was to ensure that the Bridges over Nepean River (and the nearby bridge over the Freeway at Moreton Park Road) remain serviceable at all times.

The TC held meetings approximately every three to five weeks or as required and it advised the RTA and DMR on necessary actions to maintain safety and serviceability of the bridges. The DMR Chief Inspector Mines approved continuation of the mining in stages, based on recommendations by the TC. The time frame of this project was as follows:

Initial period. Trial analyses, predictions of ground movements, calculation of structural limits:........................September, 1998 - January, 1999

Meeting of all parties. Agreement on establishment of monitoring regime. Douglas Park Bridges Subsidence Technical Committee established:..5 February, 1999

Longwall No. 16 mining completed:......................................18 August, 1999
Longwall No. 17 mining commenced:.................................. 21 September, 1999
Longwall No. 17 mining completed:......................................Mid-April, 2000
End of monitoring and reporting:..Anticipated October, 2000

This paper describes the measures taken during the mining of longwalls 16 and 17 to ensure the integrity of the twin bridges subjected to ground movements from the commencement of mining to the time of preparation of this paper in mid-July, 2000. The period of monitoring of ground movements after mid-July will be covered in the presentation at the Conference.

For the relative locations of the twin bridges and the longwalls 16 and 17 refer to Figure 1.

THE BRIDGES

The Douglas Park bridges are spaced 30.5 between centrelines and comprise 6 spans each. The overall width of the superstructure of both bridges is 13.94 m. The overall length of the northbound bridge is 285.94 m and the span lengths between the centrelines of bearings are 42.0 m, 4 spans @ 50.0 m and 42.0 m. The overall length of the southbound bridge is 236.44 m and the span lengths between the centrelines of bearings (from north to south) are 25.5m, 42.0 m, 2 spans @ 50.0 m, 42.0 m and 25.0m. The superstructures are prestressed concrete three cell box girders 2.24 m deep with vertical webs.

Each bridge is supported by 5 piers comprising twin variable cross-section rectangular columns. There are two tall piers No. 2 and 3, that are approximately 55.0 m high, one shorter pier, No.4, that is approximately 30.0 to 39.0 m in height and two short piers, No. 1 and 5, approximately 12.0 to 16.0 m high. The tall pier columns are each connected by two intermediate cross-beams, Pier 4 columns by one cross-bean and the short pier columns have no cross-beams. Piers are founded either on spread foundations in sandstone or on short cast-in-situ piles. The abutments are reinforced concrete beams on cast-in-situ piles.

Figure 2 contains the Plan and Elevation of the twin bridges.

The articulation of the twin bridges is as follows:

- The tall, relatively flexible piers, No 2, 3 and 4, are connected to the superstructure by fixed pot type bearings. At the top of the short piers, No 1 and 5, are guided sliding pot bearings allowing movement in the direction of traffic.

- The decks are each "fixed" at both abutments by spherical pot bearings that allow rotation in the vertical planes.

- A special feature of these spherical bearings, shown on Figure 3, is the elastomeric ring between the top and bottom parts of the pot bearing. Compression of this elastomeric ring allows limited longitudinal and transverse movements between the top and bottom parts of the bearings. They therefore effectively behave as horizontal spring supports to the deck under small longitudinal and transverse loadings or movements. Under large forces or movements the elastomeric ring would become fully compressed and the bearings effectively become "fixed".

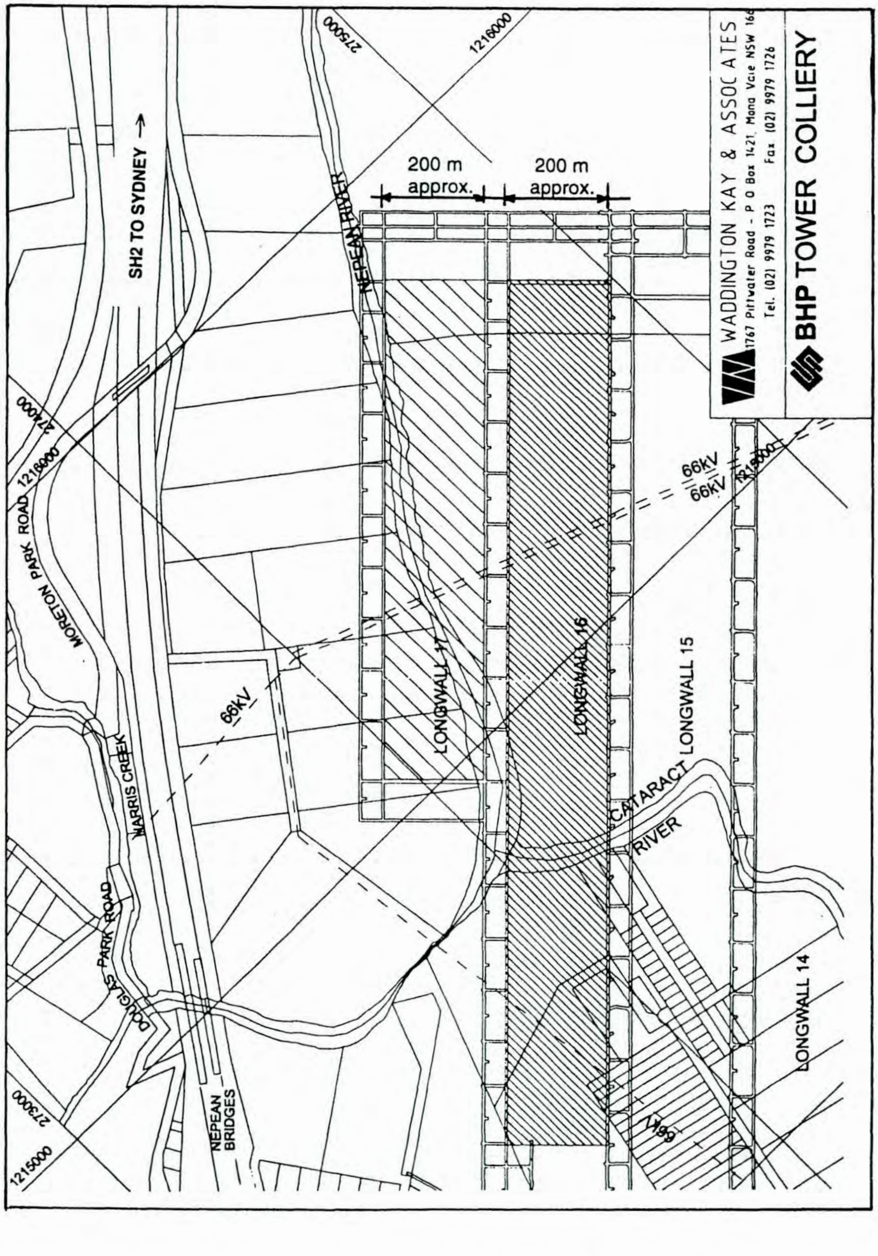

Figure1: Layout of Tower Colliery and Location of Twin Bridges over Nepean River at Douglas Park

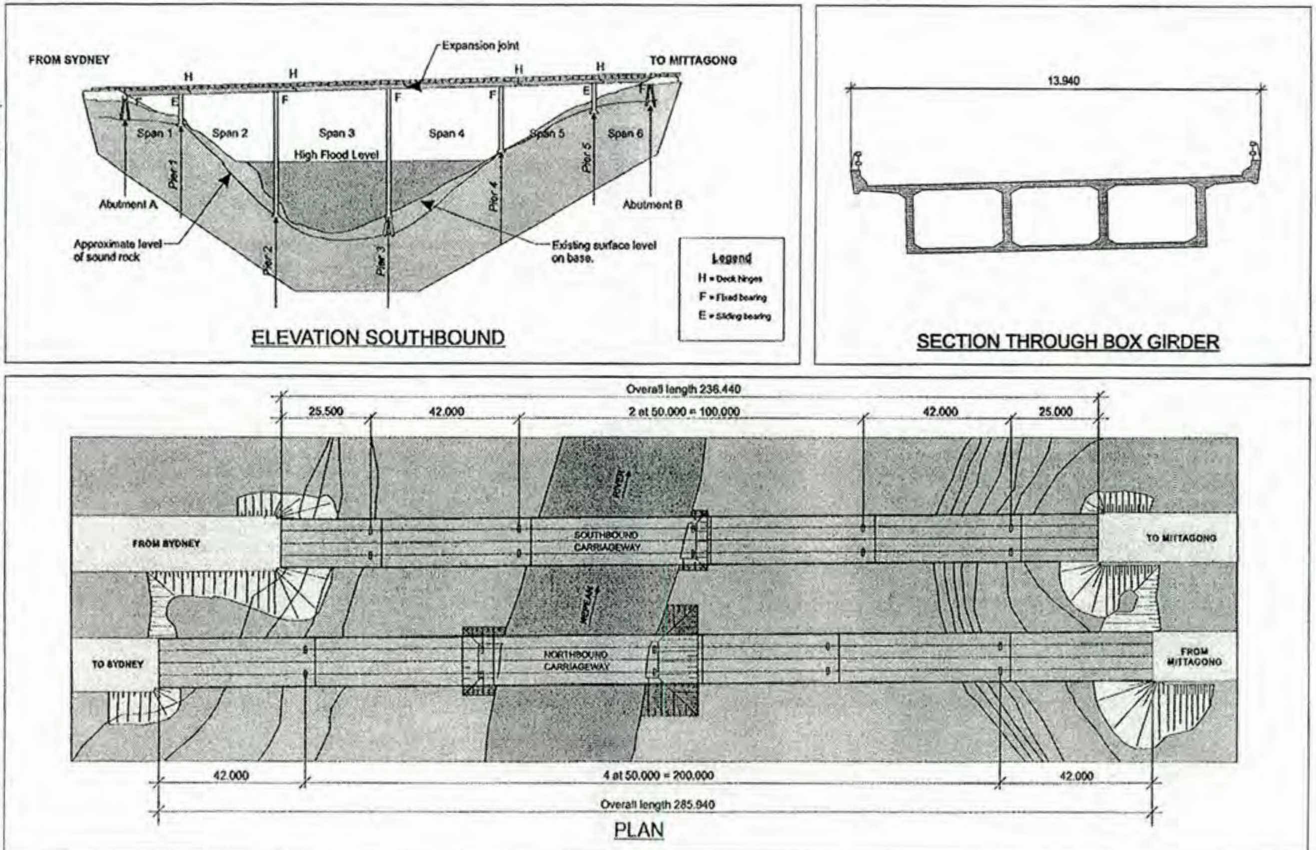

Figure 2: Plan and Elevation of Twin Bridges over Nepean River at Douglas Park

- Each bridge has one finger type deck joint in Span 4, near Pier No. 3, allowing longitudinal expansion/contraction movements.

  At this joint there is a half joint within the box girder with four stainless steel roller bearings, which allow relative rotation between the girder ends in both the vertical and horizontal planes and longitudinal translation.

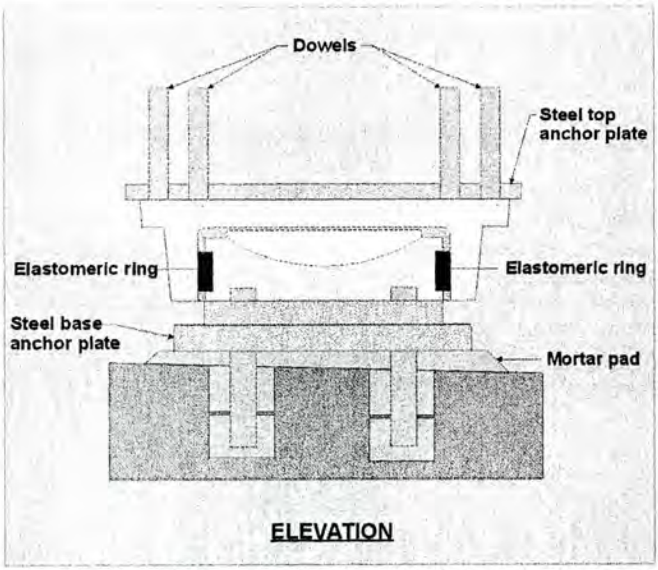

Figure 3: Fixed Abutment Pot Bearing with Elastomeric Ring Inserts

- In spans 2,3,5 and 6 of each bridge there are half joints within the box girder, with four stainless steel hinge type bearings.

- The deck girders are simply supported and are connected in all spans except span No. 1 by deck hinges located in half joints near the piers. The deck hinges in spans 2,3, 5 and 6 are stainless steel hinge bearings with dowels. The hinges allow rotation in vertical plane but no longitudinal or transverse translations or rotation in a horizontal plane.

- At all locations there are four bearings in a row.

As a result of this articulation, the bridge decks are effectively a series of simply supported spans.

Figure 4 provides schematic details of the articulation of the twin bridges.

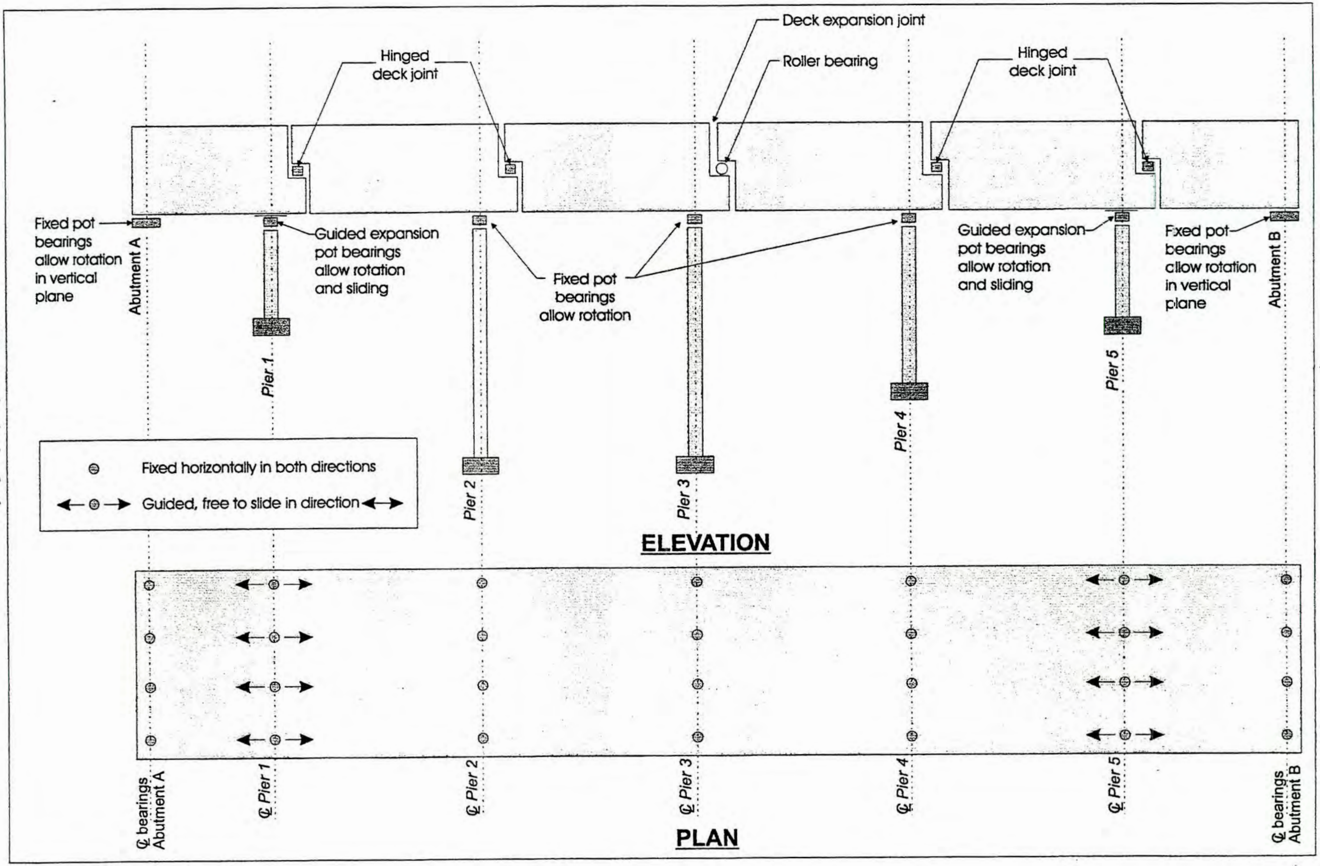

Figure 4: Articulation

The bridges were designed in 1975 by the Department of Main Roads, today the RTA. At the time of design it was not planned to extract coal beneath the bridges using longwall techniques. The Department of Mines indicated that only partial extraction of the seam would be permitted in order to limit subsidence impacts on the bridges to acceptable levels. The allowances in the design for the effects of mining were therefore restricted to provisions for the differential settlement of support and mining induced ground strains along the length of the bridge. Initially the bridges were not designed to accommodate subsidence, but in the later stages of design hinges were introduced in the deck to accommodate differential settlement between piers. It was stated that the design could not accommodate differential tilting between adjacent piers in the lateral direction.

The bridges are therefore capable of withstanding relatively large vertical differential settlements, however, the spherical bearings at the abutments and the deck hinges allow only very limited relative rotation of the bridge decks in the horizontal plane. The large distance between the outer bearings in the row of four, approximately 8.5 m, results in development of large forces in the outer deck hinge joints should horizontal curvature of the deck develop.

While the Douglas Park bridges were designed in accordance with the NAASRA 1970 Highway Bridge Design Specification, the owner, RTA, stipulated that for the current determination of the overall effects of service loadings and ground movements and calculation of the structural capacity of bridge members and components, provisions of the current Australian Bridge Design Code (ABDC) were to be used. This requirement was applied as follows:

- No structural check of the prestressed concrete superstructure was carried out. Since the spans are effectively simply supported, vertical bending effects are unlikely to be a problem, while for transverse loading effects, the strength of the deck in bending in the horizontal plane far exceeds the strength of the bearings at the deck joints.

- The review of the capacity of the bridge to carry ABDC service loads was carried out for members that may be affected by the additional loads arising form ground movements: ie the piers including cross-beams, abutment and pier bearings, deck hinge joint bearings and expansion joint bearings.

- All members and elements included in the capacity check were found to be adequate for the ABDC service loadings. The strength reserves for additional loadings arising from ground movements ranged from being substantial at the piers to relatively small at the bearings and deck joints.

MODELING OF THE STRUCTURES AND INITIAL ACTIONS

Conventional 3-D MICROSTRAN models of the structures were used for all linear elastic global analyses. Because of the differences between the two bridges, a separate model was prepared for each bridge.

The deck was modeled as a grillage with four longitudinal beam elements to represent the webs and transverse elements to represent the deck slabs. The model also simulated the hinged deck half joints located close to the piers. At the half joints the deck was free to rotate in the vertical plane, with the span No. 4 expansion joint also free to rotate in the horizontal plane and translate in the longitudinal direction. The deck was modeled as initially fixed at both abutments and Piers 2, 3, & 4, and free to slide longitudinally at Piers 1 & 5.

The piers were modeled as vertical beam elements with varying section properties to represent the tapered columns. Pier cross beams were also included in the model. The piled and spread foundations were modeled as fixed supports.

Design service loads were analyzed separately and results for potentially critical members were exported to a spreadsheet. The effects of the ground movement were analyzed as a separate load case comprising 3D translations and rotations of the supports. The results were then exported to the spreadsheet for combination with the design loads, and the combined results compared with the member section capacities. Service loads included wind loads, temperature and shrinkage effects. Pier columns and cross-beams were analysed at ULS and bearings, hinge bearings, deck joints, etc, were analysed at SLS.

Initial analyses of predicted ground movements indicated that the resulting forces in the bearings and deck hinges were very sensitive to the support conditions at the abutments. Using an assumption that the bearings were fixed in the horizontal plane resulted in very large horizontal reactions caused by even very modest relative transverse translation of the pier bases. Whilst the special abutment pot bearings with elastomeric ring insert were described in the design drawings as fixed for translation in the horizontal plane the compressibility of the elastomeric ring provided some, albeit very limited horizontal translation capacity. Springs were therefore introduced instead of the full fixity of the bearings in the horizontal plane for forces or ground movements resulting in spring displacements less than 10 mm. The bearings were modelled as fixed for effects producing greater displacements.

A sensitivity study of the effects of different spring stiffnesses indicated that determination of the spring stiffness was required with reasonable accuracy. This proved to be more difficult than originally thought because neither the RTA nor the bearing supplier, Advanx, had any detailed drawings of these bearings. In the end, the most likely thickness, width and hardness of the elastomeric ring were determined from several pieces of information available and incorporated in the Microstran model. Confirmation by physical inspection was not possible because practically the whole bearing is covered by the cylindrical top part.

Sensors were therefore installed at these bearings measuring horizontal movements in two directions with an accuracy of $\pm$ 0.05 mm. These measurements indicated that there is no metal to-metal contact between the top and bottom parts of the bearing and that the assumption of spring supports was appropriate.

The bearings at the pier tops, four at each pier, are fixed at the tall piers and guided sliding at the short piers. Four guided bearings raises the question of lateral load sharing. A typical gap between the bottom part of the bearing and the guides is in the order of 3 mm and, due to installation tolerances and post-construction changes of dimensions, equal load sharing is seldom achieved. In recognition of the difficulty in achieving load sharing between several bearings, modern designs usually rely on one bearing only in a line to resist lateral loads, or else shear keys used in combination with free sliding bearings. For the purpose of this analysis, full load sharing between the four bearings at each location has been assumed and assessment of the load sharing was carried out for critical cases in each of the regular reports. The bearings were inspected at the beginning of the monitoring and also at intervals during the mining and the gaps measured and recorded. Instrumentation of the bearings recorded consequent changes of the available gaps.

At the inaugural meeting of the TC, eight possible relative ground movement types were identified. These were: closure and opening of the gorge, vertical differential displacement of the supports, upsidence of the gorge (the bottom of the gorge raises relative to sides), absolute tilt of the piers (all piers uniform tilt), relative tilts of piers, shear of one side of gorge against the other and shear of the top of the gorge against the piers in the gorge.

For each of these ground movement types approximate limits of movements were determined from the MICROSTRAN model analysis, tabulated and the assumptions made for the analyses included, ie the shear limits were determined on the basis of the initial prediction that all supports will translate transversely proportionally to their distances from the abutments.

The purpose for the determination of limits on a range of effects of ground movements was to allow the development trigger points with the intention to either cease mining or initiate some form of corrective action in case any of the trigger points were reached.

The concept of eight identified types of relative ground movements enabled calculation of separate limits for each of the effects. However, a few trial analyses with combinations of these movement types indicated that the capacity of members may be exhausted by movements with magnitudes significantly less than any of the single movement type limits. Because of the number of possible combinations, it was not even practical to determine the limits for relative tilts of piers, let alone in combination with other effects. The only "stand alone" limits include the opening/closure of the expansion joints (given by physical limits of the joint) and vertical displacements.

The concept of setting trigger points to institute corrective actions was therefore abandoned and the TC instituted a regime of regular surveys of ground movements at pre-determined points instead. Analyses of these survey results were used to determine the capacity reserves of the structure.

Creep and shrinkage of the two structures resulted in the opening of the expansion joints from their initial settings and therefore it was determined that closure of the gorge (and consequent shortening of the distance between the abutments) of 75 mm at the northbound bridge and 45 mm at the southbound bridge would not require any remedial actions. For the event of gorge closure in excess of these values, procedures were prepared for temporary support of the bridge decks at the abutments, release of the bottom anchorages of the spherical pot bearings, jacking of the deck halves apart to provide additional movements in the deck joints and resetting of the bearing bases in new locations. However, the actual closure of the gorge was well under the calculated limits and these remedial measures were not required.

SURVEY AND INSTRUMENTATION

At the commencement of actions connected with the management of the Douglas Park Bridges for the effects of mining, the owner of the mine, BHP Tower Colliery, had already in place a comprehensive survey network established to record regularly any changes of topography related to the mining activities. Following the decision of the Technical Committee, the survey information was extended to cover the bridge structures. In addition, selected parts of the structure were instrumented to provide additional information.

To ensure that the survey provided all information required for full analysis of the structures, a list of locations to be surveyed and instrumented was prepared. These included:

Abutment and pier bases	• Transverse and longitudinal tilts
	• Transverse and longitudinal translations
	• Relative levels
Tops of pier shafts	• Transverse and longitudinal translations
	• Relative levels
Fixed bearings at abutments	• Relative longitudinal and transverse movements between top and bottom parts of the bearing (Compression of the elastomeric ring)
Guided sliding bearings at piers	• Changes of transverse clearance
	• Integrity of the attachment to concrete (relative movements)
	• Available sliding capacity
Fixed pot bearings at piers	• Integrity of the attachment in longitudinal and transverse directions
Inside the boxes	• Temperature
Expansion joints	• Available gaps
Bridge deck	• Relative levels at abutment and pier locations
	• Transverse and longitudinal translations at abutment and pier locations

Surveyed locations included intentionally a number of redundant locations where the surveyed values could be compared with the corresponding values from the computer models. This check was routinely carried out for tops of piers. The differences between the measured and calculated values at these locations were generally 5 mm or less, with somewhat higher differences on two or three occasions. As the stated accuracy of the survey was ± 3 mm, the Microstran models were considered to represent the structures well.

Typical accuracy of the survey and instrumentation was: translations at piers and abutments ±3mm, transverse tilts 0.00005 Radians, longitudinal tilts 0.00050 Radians, translations at bearings measured by instrumentation 0.05 mm.

Because of the uncertainty of the load sharing between several bearings in a row it was originally intended to instrument the bearings for strains. However, instrumentation specialists indicated that such installation was not possible in the given circumstances. Analysis indicated that the calculated strength of the connection of the bearing to the concrete member was less than the calculated strength of the bearing itself. To obtain some indication of possible overload of the bearings (which was expected to be demonstrated by yielding of the attachments), sensors were installed to record any relative movement of the bearing against the concrete member.

On the two bridges the total number of locations surveyed or instrumented was 60. Up to five different items of information were surveyed at each location and the total number of items of information provided by each survey was 216. To facilitate the exchange of information from the survey and instrumentation and to minimize the occurrence of misinterpretation of results, a spreadsheet matrix was developed jointly by Cardno MBK and the BHP Technical and Engineering Services in charge of the survey and instrumentation. The matrix contained the date of survey, location of the longwall face, and for each location the definition of the location, descriptions of the measurements, minimum accuracy, method of measurement and the sign convention. Figure 5 contains a sample page of the survey and instrumentation information matrix. For the periodical analyses, the principal inputs into the MICROSTRAN model were the five effects of the ground movement at each abutment and pier base: longitudinal and transverse translations, longitudinal and transverse tilts and vertical translations.

The survey results were provided in the form of translations relative to the southwestern abutments (abutment B of the northbound bridge). Therefore the relative longitudinal and transverse translations and the relative levels of the N/B Abutment B were always zero.

REPORTING

During the early stages of the mining of longwall No. 16, surveys and analyses were carried out approximately fortnightly, but from the time the coal face was approximately 750 m from the bridge, the DMR requested that a weekly cycle be adopted. Because the ground movements continued in the period between completion of mining of longwall 16 and commencement of longwall 17, and because of high level of stresses in some bridge members, a weekly cycle of analysis and reporting will continue until at least early August, 2000. Further monitoring may be carried out with reduced frequency, as directed by the TC.

A typical weekly cycle was as follows: survey on Monday and Tuesday, analyzing and processing of survey data and instrumentation data by BHP Technical services on Wednesday and Thursday morning, transfer of the survey information matrix from BHP to Cardno MBK on Thursday mid-day, structural analyses on Thursday afternoon and Friday, drafting of the report and preparation of attachments on Monday and the reports were dispatched on Tuesday morning.

Preparation of each report by Cardno MBK included:

- Receipt of the survey information matrix (Excel spreadsheet).
- Entering the survey information into the Microstran models and structural analyses.
- Comparison of calculated member forces and member capacities. This was carried out in the form of Excel spreadsheets.
- Preparation of graphs of principal translation affecting the structures.
- Analysis of results and drafting of the report. Each report highlighted the prominent effects of the ground movements related to the current survey results.
- Recommendations regarding the required remedial actions.

Copies of the reports were distributed to the client, BHP Coal, and to the owner of the bridge, RTA. Each report included analysis of both bridges for service loads prescribed by the ABDS plus the effects of ground movements. Forces in all members were compared with their calculated capacities and all instances where the calculated forces were found to be close to or in excess of the calculated capacity were dealt with in detail in the report and appropriate recommendations made for the consideration of the TC.

DOUGLAS PARK TWIN BRIDGES - SURVEY INFORMATION FOR ANALYSIS

Update: 23/03/00

SOUTHBOUND BRIDGE

No.	Location	Measurements provided	Meas. by:	Accuracy	How measured	Survey Notation:	Comment:	21/03/00 143.0 DPS49	14/03/00 187.25 DFS48	07/03/00 238.8 DPS47
ABUTMENT AND PIER BASES										
31a	Abutment A beam	Transverse tilt	AWT	0.00005 ra	Three pins in the beam +	NEAE, NEAW	From horizontal	-0.000028	0.000038	-0.000047
31b		Longitudinal tilt	AWT	0.0005 rad	precise levelling	NEAA, NEAE	From horizontal	0.000071	0.000140	0.000142
31c		Transverse translation	AWT	3 mm	EDM (GPS Control)	NEAP	From initial location	-32.5	-31.4	-27.6
31d		Longitudinal translation	AWT	3 mm	EDM (GPS Control)	NEAP	From initial location	1.7	2.3	0.4
31e		Relative level	AWT	0.010 m	EDM/levelling of pins	NEAP		-0.0024	-0.0048	-0.0043
32a	Pier 1 base	Transverse tilt	AWT	0.00005 ra	Three pins in the beam +	E1E, E1W	From horizontal	-0.000055	-0.000055	-0.000083
32b		Longitudinal tilt	AWT	0.0005 rad	precise levelling	E1W, E1A	From horizontal	0.000000	0.000044	0.000000
32c		Transverse translation	AWT	3 mm	EDM (GPS Control)	EL1P	From initial location	-32.0	-32.1	-30.2
32d		Longitudinal translation	AWT	3 mm	EDM (GPS Control)	EL1P	From initial location	3.0	3.6	1.10
32e		Relative level	AWT	0.010 m	EDM/levelling of pins	EL1P, EL1N		0.0010	-0.0005	-0.0008
33a	Pier 2 base	Transverse tilt	AWT	0.00005 ra	Three pins in the beam +	E2E, E2W	From horizontal	0.000032	0.000000	0.000032
33b		Longitudinal tilt	AWT	0.0005 rad	precise levelling	E2W, E2A	From horizontal	0.000000	-0.000093	-0.000093
33c		Transverse translation	AWT	3 mm	EDM (GPS Control)	EL2P	From initial location	-26.5	-32.9	-23.40
33d		Longitudinal translation	AWT	3 mm	EDM (GPS Control)	EL2P	From initial location	7.9	8.3	7.70
33e		Relative level	AWT	0.010 m	EDM/levelling of pins	EL2P, EL2N		0.0039	0.0052	0.0042
34a	Pier 3 base	Transverse tilt	AWT	0.00005 ra	Three pins in the beam +	E3E, E3W	From horizontal	0.000102	0.000091	0.000102
34b		Longitudinal tilt	AWT	0.0005 rad	precise levelling	E3W, E3A	From horizontal	-0.000158	-0.000105	-0.000053
34c		Transverse translation	AWT	3 mm	EDM (GPS Control)	EL3P	From initial location	-10.5	-6.1	-4.30
34d		Longitudinal translation	AWT	3 mm	EDM (GPS Control)	EL3P	From initial location	-4.5	-2.7	-3.80
34e		Relative level	AWT	0.010 m	EDM/levelling of pins	EL3P, EL3N		0.0127	0.0136	0.0146

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Figure 5: Typical Page of the Survey and Instrumentation Information Matrix

PERFORMANCE OF THE BRIDGES SUBJECTED TO GROUND MOVEMENTS

The maximum absolute ground translations at the bridge site to mid-July, 2000, were in excess of 50 mm while ground translations of the order of 150 mm were recorded above the longwalls 16 and 17. The maximum RELATIVE movements of the major bridge members by mid-July, 2000, were recorded as follows:

Relative translations or tilts:	Maximum values recorded:
Closure of the gorge (shortening of the distance between abutments)	10 mm
Lateral translations of abutments and piers (piers and abutments at the northern end moved east)	48.6 mm (Pier 1, Southbound)
Vertical subsidence of any pier or abutment	18mm
Transverse tilt of piers	0.000150 Radians (July, 1999)
Longitudinal tilt of piers	0.000900 Radians (April, 1999)

The lateral translations actually recorded were relatively small. However, the non-proportional distribution of these translations along the length of the bridge resulted in bending of the deck in the horizontal plane. The resulting stresses in the deck were of no concern because of the very large capacity of the deck section, however, the bearings at the deck hinges within the spans were highly stressed due to their rigidity in the horizontal plane.

The effect of the gorge on lateral translations demonstrated itself relatively early by differentiation of magnitudes of lateral translations of supports north and south of the river: translations of the abutments A and Piers 1 and 2 were significantly larger than translations of the southern supports. Lateral translations on 13 July, 1999, when the mining of the longwall 16 was some 220 m from the end, did not exceed 12 mm. However, the calculated forces in the deck hinge bearings in the northbound Span 2, located close to Pier 1, reached 95 % of their calculated strength; refer to Figure 6. Due to the differences in span lengths and heights of piers, the forces developed in the deck hinge bearings of the northbound bridge were higher than those developed in the southbound bridge. All calculated forces included the effects of ground movements and the effects of other service loads specified by the ABDC. The largest effect was the transverse wind. High calculated forces in the northbound bridge deck hinge joint bearings were recorded only a few weeks after the commencement of mining of the longwall 17, on 19 October, 1999 (130 % of calculated capacity). At that time the mining was some 920 m from the end, and the ground movements were therefore due to mining of the longwall 16, which ceased 2 months before the maximum forces were reached. To date the maximum calculated forces were recorded in the northbound deck hinge joints near Pier 1 on 6 June, 2000, at 135% of calculated strength.

The high forces in the deck hinge bearings were of particular concern, because the deck hinge bearings are located in half-joints within the deck and the access for repairs in the case of damage would be very difficult. The side view of the half joint on Figure 7 illustrates the difficult access to the joints and Figure 8 shows the details of the deck hinge joint bearings and anchorages.

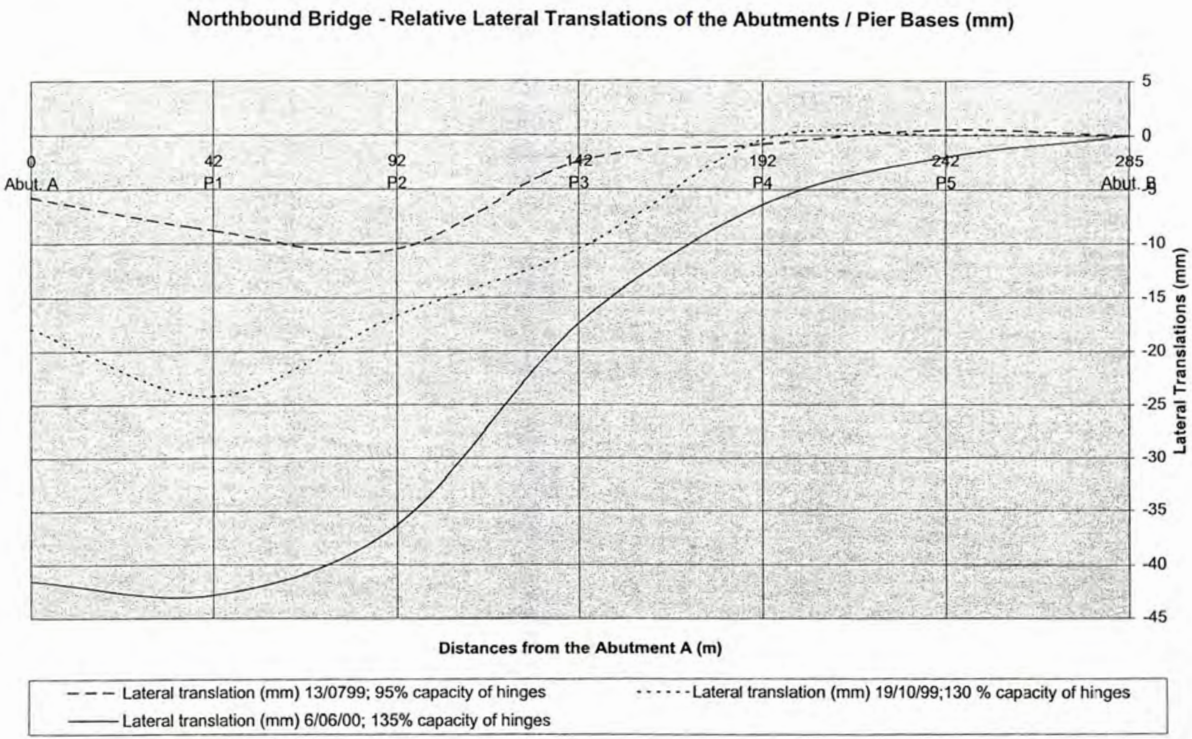

Figure 6: Plots of Relative Transverse Translations of Abutments and Pier Bases and Acting Forces on Deck Hinge Joints as Percentages of Capacity

The effects of transverse translations may be magnified or reduced by lateral tilts. The actual lateral tilts were relatively small and their effect on the overall calculated forces were not significant. The cases where the capacity of a member was exceeded resulted mostly from lateral translations.

Although the actual forces in the deck hinge bearings, which include service loading effects such as transverse wind loadings, have not actually been reached during monitoring of the structures, the RTA will not accept permanent overload of any of the bridge components and the TC requested that a study be conducted into means of alleviating the overstress. A number of analyses were carried out with a variety of support conditions and it was found that removal of lateral guides from guided sliding bearings at Pier 1 resulted in re-distribution of lateral loads and significant reduction of the forces acting on the deck hinge bearings in the critical location near Pier 1. The report prepared by Cardno MBK concluded that whilst the total lateral forces can be resisted by other supports, it would be prudent to re-instate the lateral guides at he N/B Pier 1 bearings after modification to provide additional clearance between the guides and the bearing housing. The report also recommended that unless the forces consistently and significantly exceeded the calculated capacity, the modification should be postponed until the ground movements had largely stabilised.

The instrumentation also indicated that the longitudinal movements of the guided bearings at Piers 1 and 5 were only a fraction of the calculated movements due to temperature change, longitudinal translations and tilts of the pier bases. These bearings were repeatedly inspected and the concensus was that the likely reason for the limited movements is that the guides of some of the bearings are not completely parallel with the bearing housing and at zero lateral clearances some bearings ceased to further translate. The structures were analysed for the upper boundary condition in which the bearings were assumed to be completely fixed. This did not result in an increase in the forces in the piers above their capacity, however, under this condition the lateral capacity of the bearing attachments was slightly exceeded.

The calculated total effects in other members never approached their calculated capacity. The piers in particular had capacity reserves that alleviated any concerns about the stability of the structures.

Relative longitudinal and transverse movements between the top and bottom parts of the "fixed" abutment bearings were regularly analysed. A lack of movement would indicate that the compressibility of the elastomeric ring had been exhausted and that there was a metal-to-metal contact. This would lead to a sudden increase in the horizontal reactions at these bearings and possible damage to the bearings and/or their attachments. An investigation into possible remedial action aimed at release of such large forces, if the need arose was carried out. A procedure for cutting the concrete around the bottom anchor bolts, temporary support of deck, re-positioning of the bearing bases and restoration of the supports has been developed and documented.

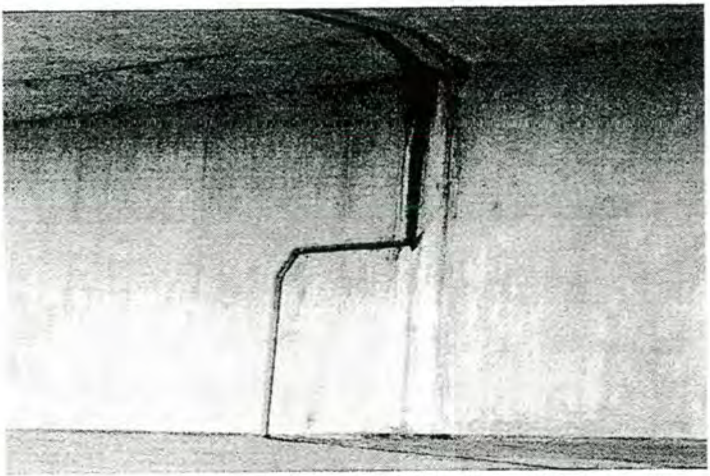

Figure 7: Elevation of the Deck Half Joint with Deck Hinges Inside

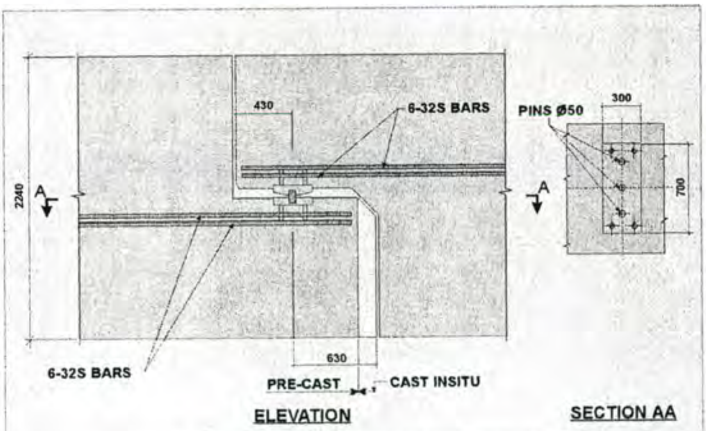

Figure 8: Deck Hinge Joint Bearing

CONCLUSIONS

The bridge members and components which were identified as being most likely to be affected by ground movements were assessed in the initial design check. These components, designed to the 1970 NAASRA Highway Bridge Design Specification, were found to be adequate for the service loads specified the current Australian Bridge Design Code.

Methods for predicting mine subsidence induced movements, and assessing the effects of mine subsidence on structures, were well advanced in 1975, when the freeway bridges were designed. It was known that vertical subsidence would be accompanied by tilts, curvatures and strains. It was understood that the strains resulted from differential horizontal movements that are generally directed towards the mined area and that the maximum horizontal displacement occurred at the point of maximum tilt. It was, however, believed that all of these ground movements were confined within the limit of subsidence defined by an angle of draw of 26.5^{o}, i.e. within a distance of half the depth of cover from the edge of the longwall.

Little was known at that time regarding the effects of mining on gorges, river valleys and cliff lines, though the Department of Main Roads were concerned that the gorge could cause the mining induced strains to be concentrated at the bridge location. It is only recently that horizontal movements have been recorded within the strata and in some cases these horizontal movements have been recorded well outside the normal limit of subsidence even though the vertical ground movements have been confined within the subsidence trough. The mechanisms causing this are not fully understood and are the subject of ongoing research, but the movements appear to be principally caused by the redistribution of in-situ horizontal stresses in the strata as mining occurs. This stress redistribution is further complicated at Tower colliery by the presence of the Cataract and Nepean Gorges and the fact that the in-situ stresses in the strata are very high.

Clearly, any ground movements are of concern wherever major bridges are built over deep gorges and the experience at Tower Colliery is unique in that this is the first time that the implications have been fully recognised and the situation comprehensively managed.

The most significant actual effects on the bridges resulted from the relative transverse translations of the pier bases and abutments. Although the magnitudes of these relative movements were below the initially determined single movement type limits, combinations of lateral translations with other effects of ground movements, ie tilts, and particularly the fact that the transverse movements were not proportional to the distances of the piers from abutments (refer to Figure 6) , caused large stresses in the deck hinge and joint bearings from the time the magnitudes of transverse translations were approaching 20 mm. Due to the combination of outer/inner span ratios, heights of piers and ground movements, the location of maximum stresses in the deck hinge bearings has been consistently at the outer two hinge bearings in span 2 (near Pier 1) of the northbound bridge.
There are lessons to be learned from this experience for improvement in design approaches and detailing.

The design appeared to assume an equal distribution of the lateral forces between the four guided bearings at each pier. Because of the nominal gap between the bearing and the guide in pot bearings, construction tolerances and the long term effects of shrinkage, it is unlikely that such an uniform distribution can be actually achieved. Good design practice, which is believed to be now generally followed, is to design one bearing (or other device) to carry all transverse loads at any one pier or abutment. Such detailing provides for a clear load path and removes any uncertainty. During the mining operations, the calculated lateral forces required the combined capacity of two to three bearings out of four to share the lateral loads. Considering that the actual capacity of bearings to carry lateral loads is normally in excess of the stated capacity, and that the calculated strength of the attachments exceeded the stated capacity of bearings, this was considered to be acceptable.

The calculated capacity of the deck hinge bearings was somewhat higher than the minimum calculated capacity of their anchorages. If the anchorage lengths of the reinforcement parallel with the steel plates of the hinge bearings had been longer, the ratio of forces acting to anchorage capacity would have been more favourable and up to the date of writing this paper, the forces acting on the bearings would have rarely exceed the capacity. However, it is acknowledged that for the design service loads, excluding the effects of ground movements, the design and detailing was adequate.

The experience with the Douglas Park bridges strongly supports the ABDC requirement for bearing replaceability, which was not mandatory at the time of design. It is not the fault of the original design that under the effects of ground movements the deck hinge bearings were found to be the weakest component of the bridge. However, complete lack of access for any maintenance or replacement made the bridge vulnerable to relatively small transverse movements. Such movements may occur (ie due to settlement of supports) on other bridges not affected by mining.

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