Fourth Australia - New Zealand
Conference on Geomechanics,
Perth, 14-18 May 1984

Scheduled Removal of Railway Tunnel

E.G. TRUSCOTT
Director, GHD-Wood Geotechnical Pty. Ltd., Brisbane
and
I.G.F. DAVIDSON
Senior Engineer, Gutteridge Haskins & Davey Pty. Ltd., Brisbane

SUMMARY

In the duplication of the railway line from Blackwater to Rockhampton, Queensland, economic studies showed that a hundred year old, sandstone masonry lined tunnel should be replaced by an open cut. As the line carries the coal from mines in the Bowen Basin delays to rail traffic were critical.

Detailed studies consisting of field investigations, finite element analyses, economic analyses and construction planning were undertaken to determine the most economic alternative that would lead to an acceptable delay to train traffic. The paper describes the design and construction aspects of the work.

1. INTRODUCTION

The current development and expansion of coal mines in the Bowen Basin area in Central Queensland, has resulted in the need to duplicate sections of Queensland Railways Central Line to carry the increased coal traffic. Duplication works have comprised the widening of existing embankments and cuttings, extension of culverts and duplication of bridges to carry the new line at 4 m offset from the existing track. However, near Edungalba there was an 85 m long tunnel at a location named appropriately, Tunnel. This tunnel was built in 1875, according to the date inscribed on both portal keystones. The lining was constructed from hewn standstone masonry blocks.

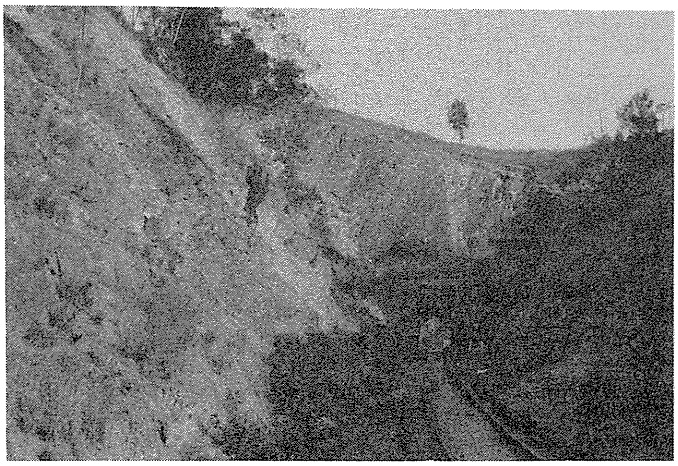


Figure 1: Tunnel Before Demolition

Although the overall cost of the part of the project involving the tunnel was relatively small (about $1 million) the economic consequences of significant delays to the transport of coal from the mines to the port were large. Hence the duplication of the line in the tunnel section had to be carefully planned to achieve an economic project layout that could be constructed with a minimum delay to rail traffic.

2. GEOLOGY

The tunnel is located in the indurated siltstone and mudstone of the middle Bowen formation. In the immediate vicinity of the tunnel the rock is closely jointed and varies from highly to moderately weathered. The strike is approximately parallel to the tunnel alignment and the dip varies from about 70° on the southern side of the tunnel to 40° on the northern side. There is only limited soil cover. A significant slope failure occurred on the south slope adjacent to the western portal during the construction of the original railway line. The debris from this failure was cleaned back to the failure plane and rock beneath the plane is only slightly weathered with widely spaced joints. It dips under the southern corner of the tunnel.

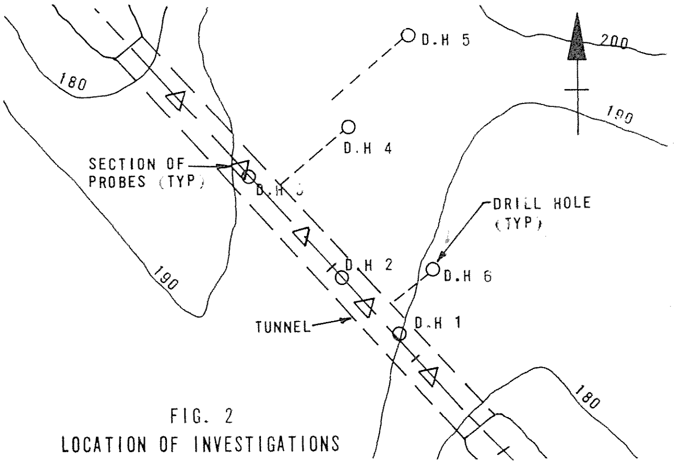


FIG. 2
LOCATION OF INVESTIGATIONS

3. FIELD INVESTIGATIONS

(a) Tunnel Lining

The tunnel lining consisted of sandstone blocks up to 500 mm square and of varying lengths mortared together to form an arch. Investigations were carried out to check the quality of the lining and the amount of support it provided to the surrounding rock. This was done by drilling a series of percussion holes through the lining from a rail mounted platform. Typical details are shown on Figure 3. A total of 26 holes were drilled at 5 stations spaced approximately equally along the tunnel.

The condition of the sandstone lining varied. In several locations crown blocks moved when touched with the drill. Voids up to 500 mm deep were frequently encountered immediately behind the lining. On many occasions dust and drill cuttings issued from holes in the tunnel lining several metres away from the hole being drilled, showing that voids were continuous. Hard rock was encountered 1 to 1.5 metres from the inside of the lining. Drilling in the rock between the void and hard rock was easy, indicating either highly fractured bedrock or loose rubble. Rotted timber was encountered in some holes suggesting that at least part of the tunnel had been supported by timber prior to installation of the sandstone lining.

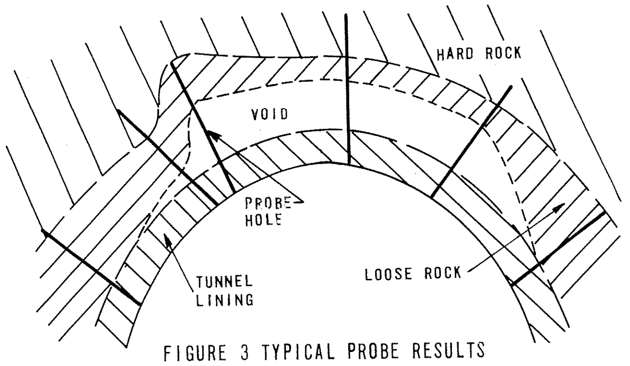


FIGURE 3 TYPICAL PROBE RESULTS

These investigations revealed that the sandstone lining was providing limited to zero support to the rock arch and could not be relied upon to withstand any sudden load thrown upon it. Thus only minimal disturbance could be accepted to the tunnel during the excavation stage.

(b) Surrounding Rock

Core drilling with N size diamond bits was carried out to study the quality and nature of the rock above and adjacent to the tunnel. The location of drill holes are shown on Figure 2. Tests were carried out with an OYO Elastmeter Model 100 pressuremeter to give insitu rock moduli. Results are summarised in Table 1.

Hole No	Depth (m)	Modulus (MPa)	RQD
DH2	6.5	275	0
DH2	10.5	250	0
DH2	14.5	240	0
DH4	15.0	240	0 to 12
DH4	19.8	422	42
DH4	25.0	45	0

Table 1. Results of Pressuremeter Tests

All drill holes encountered mudstone for their full depth with minor variations in colour, weathering and degree of fracturing. The maximum rock quality designation (RQD) was 81 percent and about half the runs had an RQD of zero, indicating that overall rock quality was poor.

Four of the moduli measured by the pressuremeter fell into the narrow range 240 to 275 MPa. The low value in DH4 was considered unrepresentative because the test was conducted quite close to the tunnel. The high value in DH4 corresponded to an area of higher RQD (see Table 1). As the RQD was generally low, especially near the tunnel, a homogeneous modulus of 250 MPa was adopted for the rock around the tunnel.

4. FINITE ELEMENT ANALYSES

(a) General

A two dimensional finite element method of anlysis was used to investigate the effects on the overall stability of the tunnel of excavation of rock both above and adjacent to the tunnel. The effect of plant working above the tunnel was also briefly investigated.

The finite element analysis method assumes an isotropic, elastic material. This is obviously an oversimplification of the actual rock mass. Thus the results serve primarily to indicate trends in behaviour and identify likely unstable regions. These data can then be used as a basis for judgement decisions.

The finite element analysis identifies regions of tensile stress above or adjacent to the tunnel for given rock parameters. Kaltani (1975) reported that a study of rock slope case histories has shown that tensile stress regions in a rock mass appear to identify areas that are unstable and that are likely to fail. Unloading around a tunnel tends to reduce theoretical tensile stresses induced by gravity loading. Unacceptable conditions have been assessed as those which result in an increase in tensile stress, an increase in the size of the region affected by tensile stress or the development of tensile stresses in a previously compressive region.

(b) Description of Analyses

The computer program CEASAP was used for all analyses. Details of the program may be found in the CEASAP Users Manual.

The mesh was drawn up to analyse a section at the midpoint of the tunnel and included the geometry of the various excavations. Small element sizes were adopted in the region of the tunnel to give greater accuracy in the area of interest. The boundaries of the mesh were placed sufficiently far from the tunnel to ensure that boundary effects were negligible.

As the lining was not providing significant support to the rock arch the analysis assumed the tunnel opening extended to the hard rock line. In most analyses the rock was assumed homogeneous with a modulus of 250 MPa. (see Table 1) A few analyses were performed with a higher modulus zone (500 MPa) on the southern side to model the fresher rock in that area. A poissons ratio of 0.25 was adopted.

The first set of analyses modelled excavation levels down to RL180. The second set considered side cuts at increasing distances from the tunnel. The analyses run are summarised in Table 2.

The analyses showed that the only area of initial tensile stress (homogeneous rock properties) was

1. Homogeneous Rock		Level of Initial Cut Ground Surface	185	182.5	180
No side cut		*	*	*	*
Distance of side cut from tunnel centreline	11.0m			*	*
	13.5m				*
	16.0m	*		*	*
D8 Dozer					*
2. Hard Southside No side cut		*	*	*	*

Table 2. Finite Element Analyses Performed

at the crown of the tunnel. Both the size of the tensile area and magnitude of the maximum stresses decreased as excavation was carried out. However, at an excavation level of 180 additional tensile stress zones developed at about the tunnel spring line. For the case with the stiffer rock mass to the south of the tunnel the tensile stress areas and magnitude were much larger. The tensile stress magnitudes and affected areas reduced with reduced excavation level to RL182.5 but at RL180 interconnection occurred between tensile zones and general instability resulted. Thus both analyses indicated open cut excavation could not proceed below RL182.5 without endangering the tunnel.

Side cuts were checked at locations of 11, 13.5 and 16 m from the tunnel centreline. Large increases occurred in the magnitude of tensile stresses at the tunnel crown with the cut at 11 m from the centreline but these reduced reasonably rapidly so that at 16 m distance only a minor increase resulted.

Considerable excavation could have been saved by steepening the bottom of the side slope adjacent to the tunnel. Analyses showed that local steepening of this nature lead to adverse stress conditions and probable tunnel failure.

One analysis was run to check the effect of carrying out the side cut first, i.e. excavating from existing ground surface. This resulted in large increases in both magnitude and area of the tensile stress.

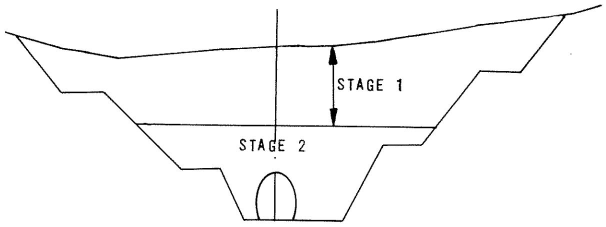
The static loading due to a bulldozer above the tunnel was modelled by applying a strip load of weight about double that of a reasonable sized bulldozer both at the tunnel centreline and offset 3 m from the tunnel centreline. In both locations the changes to stresses at tunnel level were negligible.

5. DESIGN OF LAYOUTS

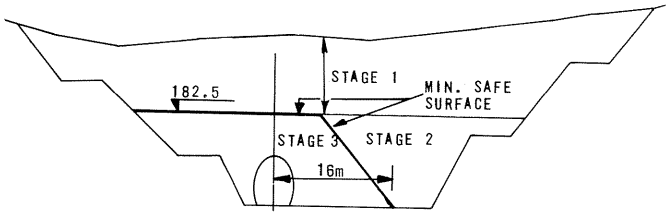
The design of the layout involved determining the most economic method of duplicating the track in the tunnel area which would not cause undue delay to the coal trains. A minimum delay period of about 24 hours would be required to rearrange track connections when new lines were introduced. This could be tolerated by appropriate re-scheduling of rail operations.

Preliminary layouts were prepared for a double track open cut in the tunnel location, twin tunnels, a double track completely bypassing the tunnel location, and a combination of open cut remote from the tunnel and open cut at the tunnel. In alternatives involving the existing tunnel considerable expense was required to stabilise the structure. Economic studies indicated that the open cut in the tunnel area was considerably cheaper than the other alternatives.

Detailed studies were undertaken to select the optimum open cut layout. Two possibilities were investigated. In both cases the ground surface was lowered to RL182.5. In the first alternative (base width 13 m) the rail track was closed down for the period required to excavate below RL182.5 and re-establish the rail traffic. In the second alternative (base width 25 m) a side cut was excavated 16 m from the tunnel centreline, the rail line rerouted around the tunnel and the tunnel area excavated as a separate operation. Details are shown on Figure 4. Studies showed that first alternative would cost about $300 000 less than the second and delays to rail traffic could be limited to 48 hours with proper planning. The Railway Department decided that a scheduled 48 hour delay could be tolerated by carefully stockpiling coal at the port prior to the closure. Hence the first alternative was selected.



ALTERNATIVE 1 - 13m BASE WIDTH - 48HR. DELAY



ALTERNATIVE 2 - 25m BASE WIDTH - MIN. DELAY
FIGURE 4 ALTERNATIVE CUT LAYOUTS

As noted previously parts of the existing tunnel lining had deteriorated badly and could not be relied upon to provide support if rockfalls occurred during excavation. In case a minor lining failure occurred enough preformed steel sets together with cribbing and blocking to support a 20 m length of tunnel were kept on site. In the event of a major collapse all Contractor's and Railway equipment in the area would have been mobilised and a 24 hour a day operation mounted to clear the track as soon as possible.

In view of the importance of safeguarding the tunnel, movements of the tunnel walls and crown were monitored, and geophones were installed in the rock above the crown of the tunnel to measure the intensity of vibrations induced by construction equipment.

6. EXCAVATION OF BENCH AT RL182.5

(a) General Description

The process of removal involved firstly the excavation of up to 10 metres depth of overburden to RL182.5 m. The widening of the north side approach cuttings to formation level was also carried out at this time.

The highly fractured and jointed overburden material was easily ripped by Komatsu 355A bulldozers using two tynes. The ripped material was removed by Komatsu WS 231 scrapers, hauling a nominal 12 cubic metre load, push loaded by the dozers. The excavation was maintained approximately level at all times.

A portal protection structure was constructed at each end of the tunnel to prevent debris falling onto passing trains during excavation and to accommodate material in the remote event of a major slip at the portals. The structure comprised an elevated platform with parapet 8 m above the track extending 5 m out from the portal face at each end of the tunnel and extending 3 m each side of track centreline. In practice no instability occurred at either portal face and only minor spalling fell onto the platform.

Prior to excavation a continuous sleeper mat was fastened down to the existing track through the tunnel and in both approach cuttings. Clean ballast was placed in the cess areas to rail level thus providing a level platform. The principal function of the mat was to provide protection to trackwork components from the frequent passage of heavy plant and equipment. It also braced the existing track structure, provided a level platform for clearing spalled debris from the rail track and formed a level walkway for all personnel going through the tunnel.

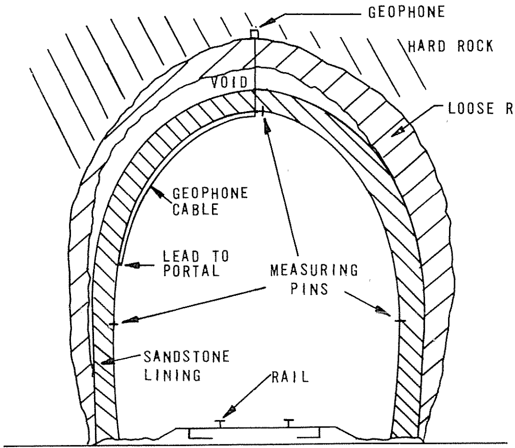


FIGURE 5 INSTRUMENT LOCATIONS

(b) Construction Vibrations

At the design stage the peak particle velocity due to construction vibrations was estimated from the curves provided by Wiss (1981) for a large bulldozer. A safe peak particle velocity in the rock in the crown of the tunnel of 3 mm/sec was assumed. This gave a minimum distance from the rock at the tunnel arch to the closest construction equipment of 6 m i.e. an excavation level of about RL182 which coincided approximately with RL182.5 obtained from the finite element analyses.

Construction vibrations were monitored by installing geophones at three locations in the rock in the crown of the tunnel. Details are shown on Figure 5. Vibrations were measured using the peak vibration monitor PVM4 manufactured by the Zero Corporation USA. Measurements taken when trains passed through the tunnel gave a peak particle velocity of 0.55 mm/sec. Measurements during operation of construction equipment gave peak particle velocities varying from 0.2 mm/sec at RL186.5 to 0.9 mm/sec at RL182.5. Peak daily velocities are plotted against distance in Figure 6.

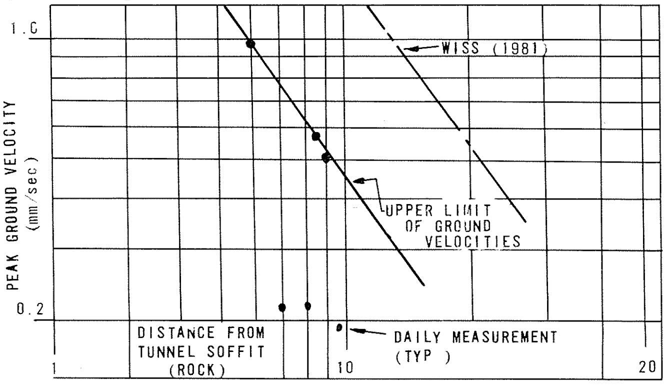


FIGURE 6 MAXIMUM GROUND VELOCITIES

When the excavation level reached RL182.5 measurements were taken of vibrations caused by the largest bulldozer on the job, a Komatsu 355A. The effect of speed, distance from the geophone and location relative to geology were investigated. There was no apparent effect of direction as velocities measured from equipment operating north and south of the tunnel centreline were similar under similar conditions. Measured data are plotted in Figure 7. As can be seen the effect of bulldozer speed was very significant with peak ground velocities at higher speeds being up to three times those at low speeds. The tests were carried out on relatively flat areas. Peak ground velocities up to twice those measured during the tests were observed when the bulldozer travelled over rough ground .

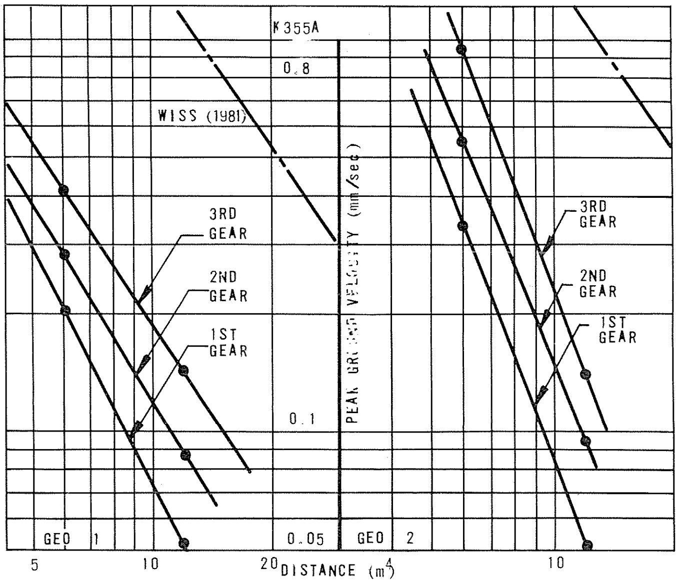


FIG. 7 VIBRATION MEASUREMENTS - EFFECT OF VELOCITY

Both the daily peak velocities and those measured during the tests with the 355A varied with distance in the same way as the Wiss (1981) data. However, the values were significantly less than those shown by Wiss. As noted previously the minimum rock thickness obtained using Wiss's curves was 6 m. The distance based on the measured data would have been 4 m. It is considered that the factor of safety provided by the extra 50 percent thickness is not excessively conservative. Thus Wiss's curves provided a reasonable basis for design.

(c) Movement Measurements

Movement measuring points were installed by ramsetting nails into the sandstone blocks. Ten stations were established at approximately equal spaces along the tunnel. Each station consisted of a measuring point in the roof and a measuring point on each side of the tunnel about 2 metres above rail level. Details are shown in Figure 5.

The elevation and transverse location of each point were measured at regular intervals as excavation proceeded above the tunnel. Elevation measurements showed that no significant vertical movements occurred. Transverse measurements showed that movement of the north wall was negligible and that slight but significant movements of the south wall occurred. These movements ranged from 1 to 6 mm and averaged 3 mm. Movements ceased after the excavation reached RL182.5.

The inward movement of the south wall is consistent with the stress changes and geology. The general dip of the beds is towards the north. Removal of the overburden lowered stresses on the side and top of the tunnel thus reducing frictional resistance between beds. This allowed pieces of rock to slide up against the tunnel wall causing the small movements.

7. DEMOLITION OF TUNNEL

The demolition of the tunnel and removal of enough excavated material to re-establish rail traffic on the existing track was scheduled within a 48 hour line closure during the weekend of 19 to 21st March 1983. The amount of material to be excavated during tunnel closure was 22 000 m^3.

As much excavation as possible was carried out prior to track closure. This included widening of the approach cuttings and excavation to the berm below RL182.5 on the north side of the cutting. (see Figure 4) Blast holes were drilled prior to tunnel closure to save time.

About 40% additional construction plant was mustered for the period. All machines underwent a comprehensive maintenance service during the preceding week. The only breakdown during the period was a flat scraper tyre.

Detailed precautions were taken to protect rails and services from blast damage. The braced rails were covered with a ballast blanket. Services close to the blast were disconnected and removed. Larger services were protected by special covers. In addition crews of specialist railway repairmen were on hand in case any major repairs were needed. Actual damage was limited to minor local movements of the rails and minor damage to the heads of a few rails.

Detailed consideration was given to design of the blasting sequence and pattern and a high powder factor was adopted to ensure that complete demolition of the tunnel occurred at the first detonation. The holes in the tunnel wall were detonated prior to overburden holes to drop the tunnel lining. Overburden holes were raked at 12° to the vertical, drilled at 2-3 m grid centres and detonated in a chevron pattern with 25 milli- second delays successively from each portal inwards. Molonite was used as primer and nitro prill as the main blasting agent. This blasting pattern resulted in a profiled hump of material with ramps for immediate dozer access.

Actual work started at 0430 hrs Saturday 19th March 1983. Protective ballast was in place over the rails and blast holes wired by 0700 hrs with detonation taking place at 0702 hrs. Bulk excavation commenced at 1100 hrs after access ramps had been shaped by the dozers. An excavation rate of 1100 m^3 per hour was achieved (compared with the required rate of 700 m^3 per hour) with the bulk of the excavation being completed by midmorning Sunday. The work was completed by 2100 hrs Sunday, 8 hours ahead of schedule.

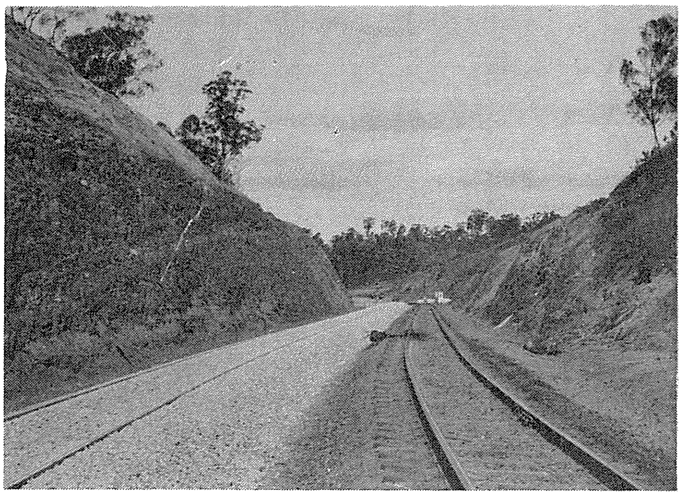


Figure 8: End of Construction

8. CONCLUSIONS

1. The duplication of the railway line at Edungalba was successfully achieved by replacing the existing tunnel with an open cut. Extensive planning, including significant geotechnical investigations and analyses, was required to obtain an economic solution that did not result in serious delays to rail traffic.

2. Relatively simple finite element analyses provided a better understanding of the behaviour of the rock mass during a given construction sequence, and therefore aided significantly in making judgement based decisions.

3. Measurements of vibrations due to construction equipment showed that peak ground velocities are dependent on equipment speed and surface roughness as well as equipment weight and distance from the measuring point. Velocities measured were less than those presented by Wiss (1981) but not excessively so.

Acknowledgements

The project management, including design work and construction supervision was carried out by Gutteridge Haskins and Davey Pty Ltd and Queensland Railways are the owners of the project. The authors would like to thank both organisations for permission to publish this paper.

References

Kalkani E C, "Two Dimensional Finite Element Analysis for the Design of Rock Slopes", Proceedings of the Sixteenth Symposium on Rock Mechanics, University of Minnesota, USA, 1975.

CEASAP Users Manual, Computer Engineering Applications Pty Ltd.

Wiss, 1981 "Construction Vibrations - State of the Art", ASCE, Geot Eng Vol 107, No GT2.