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VICTORIAN SYMPOSIUM  
**Geotechnics and  
transport infrastructure**

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AUSTRALIAN GEOMECHANICS SOCIETY  
**VICTORIA CHAPTER**



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# PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2018 Australian Geomechanics Society Victorian Symposium on 'Geotechnics and transport infrastructure' held on 24 October 2018.

In recent years Victoria has seen significant investment in transport infrastructure as part of a plan to manage the demands of a growing population and expanding urban fringe. The construction of Melbourne Metro, a second crossing of the Yarra River, rail and freeway upgrades as well as numerous level crossing removal projects are just some of the major transport projects currently underway in Melbourne and regional Victoria. Many of these projects carry numerous complex geotechnical challenges.

The 2018 Australian Geomechanics Society Victorian Symposium covers a variety of geotechnical challenges associated with transport geotechnics and present overviews of current infrastructure challenges, state of-the-art practices, innovation, new research results and case studies demonstrating applications of advanced techniques and cost effective solutions in the construction and design of local transport infrastructure. The Symposium brought together professional engineers, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of transport infrastructure and associated geotechnical challenges and applications.

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## Keynote Address

### Design of large span tunnels and caverns: back to basics

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#### ABSTRACT

Increased demand to future-proof tunnel projects with respect to traffic has led to the proposal of some very large spans in recent road tunnel projects in Australia. For example, four lane tunnels are currently under construction in Sydney with mined spans of approximately 20 m and Y-junction caverns of unprecedented spans for road tunnels in Australia, all with a requirement for 100-year design life. As these spans are unprecedented in Australian civil tunnels, a direct comparison with local past experience is not possible and simple extrapolation of precedent designs, although potentially solving the problem, often result in uneconomical solutions that do not necessarily target the actual failure mechanisms involved in the excavation of such large spans. International experience could certainly be used but adequate design justification would still have to be provided. Although there is certainly room for cutting edge innovation, robust solutions can also be achieved by simply going back to basics. As a result, this paper intends to present and discuss how designs that focus on first principles and the basic objectives of rock reinforcement may allow for a better understanding of the design requirements and how to satisfy codes and standards but also provide savings with respect to ground support. The key to the design involves understanding the failure mechanism that needs to be addressed, its relationship with the different actions of rock bolting, i.e. suspension/anchorage and/or rock reinforcement and what could be acceptable.

*Keywords:* large span, rock support design, rock bolt reinforcement, suspension

#### 1 INTRODUCTION

With rapid development of cities, it is crucial that the use of the underground space is made efficiently with projects that can cater for the needs of the population for several decades. This has led to an increased demand to future-proof tunnel projects with respect to traffic which resulted in the proposal of some very large spans in recent road tunnel projects currently in construction in Australia all with the requirement for 100-year design life.

For example, several kilometres of four lane tunnels are currently under construction in Sydney with mined spans of approximately 20 m. Such spans had only been experienced in localized excavations in widened sections such as breakdown bays and Y-junction caverns but not for long lengths of tunnelling. In addition, the Y-junction caverns now required for these tunnels are also unprecedented for road tunnels in Australia with spans reaching 31 m and exceeding experience in Australia which include the Eastern Distributor in Sydney

(24 m) and both Kedron (26 m) and Lutwyche (27 m) caverns of the Airport Link tunnels in Brisbane.

These large span road tunnels are currently in construction for the New M5 and M4-M5 Link tunnels as part of the infrastructure project known as WestConnex. The WestConnex project is a 33-kilometre underground motorway currently being constructed in Sydney's Inner West (Figure 1).

As these excavation spans are unprecedented in Australian civil tunnelling, a direct comparison with local experience is not possible particularly considering the semi-flat roof tunnels typically excavated in Sydney.

Although simple extrapolation of precedent designs could potentially provide a solution, two risks arise: (1) the extrapolation based on different excavation shapes that do not necessarily target the actual failure mechanisms involved in the excavation of such larger



Figure 1. WestConnex Motorway

spans; and (2) provide uneconomical solutions. International experience could certainly be used but adequate design justifications and analysis would still have to be provided to verify its application locally.

The search for solutions to new problems often target innovation. However, considering that the new challenge described above in fact involves an “old” problem but at a larger scale, it is considered appropriate to review the basic design assumptions to find robust solutions in more fundamental design principles. As a result, this paper intends to present and discuss how a design that focus on the basic objectives of rock reinforcement may allow for a better understanding of the design requirements and still provide savings with respect to ground support. The key to the design involves understanding the failure mechanism that needs to be addressed, its relationship with the different actions of rock bolting, i.e. suspension/anchorage and/or rock reinforcement and what could be acceptable.

## 2 PRECEDENT DESIGN

The first step in any design is typically a comparison with precedent experience of what has worked and what has not. Therefore, Figure 2 presents a comparison between span and bolt length for several successful projects in Australia prior to 2014.

Significantly experience is observed for tunnels under 18 m span. For these span tunnels, the bolt length roughly follows a linear relationship with bolt length equal

to approximately one third of the span. On the other hand, only a small number of projects with spans greater than 20 m exist in Australia and if they are included, the global best fit indicates a power curve that deviates from the linear trend. Based on such a best fit, a bolt length of approximately 6 m would be considered a precedent design for tunnels of approximately 20 m span.

For tunnels where instability is generally controlled by geological structures, the rule of thumb for bolt length approximately equal to one-third of the span is somewhat related to conservative assessments of the largest possible wedge assuming ubiquitous rock defects and ignoring stress effects. These assessments generally result in wedge widths that span approximately the entire tunnel as illustrated in Figure 3, and wedge apex heights approximately equal to one third of the span, particularly for tunnels smaller tunnels of, say, 15 m span or less.

For the flat-roofed tunnels typically excavated in Sydney, the design approach generally was that proposed by Bertuzzi and Pells (2002) as illustrated in Figure 4. The design is no longer governed by structurally controlled wedges but involves selecting a roof beam thickness which would in turn dictate the rock bolt length based on deflections that are deemed acceptable. This deflection typically varies between 10 mm and 20 mm. It is important to note that Bertuzzi and Pells (2002) suggested to add 1 m extra in bolt length to the beam thickness for embedment purposes.

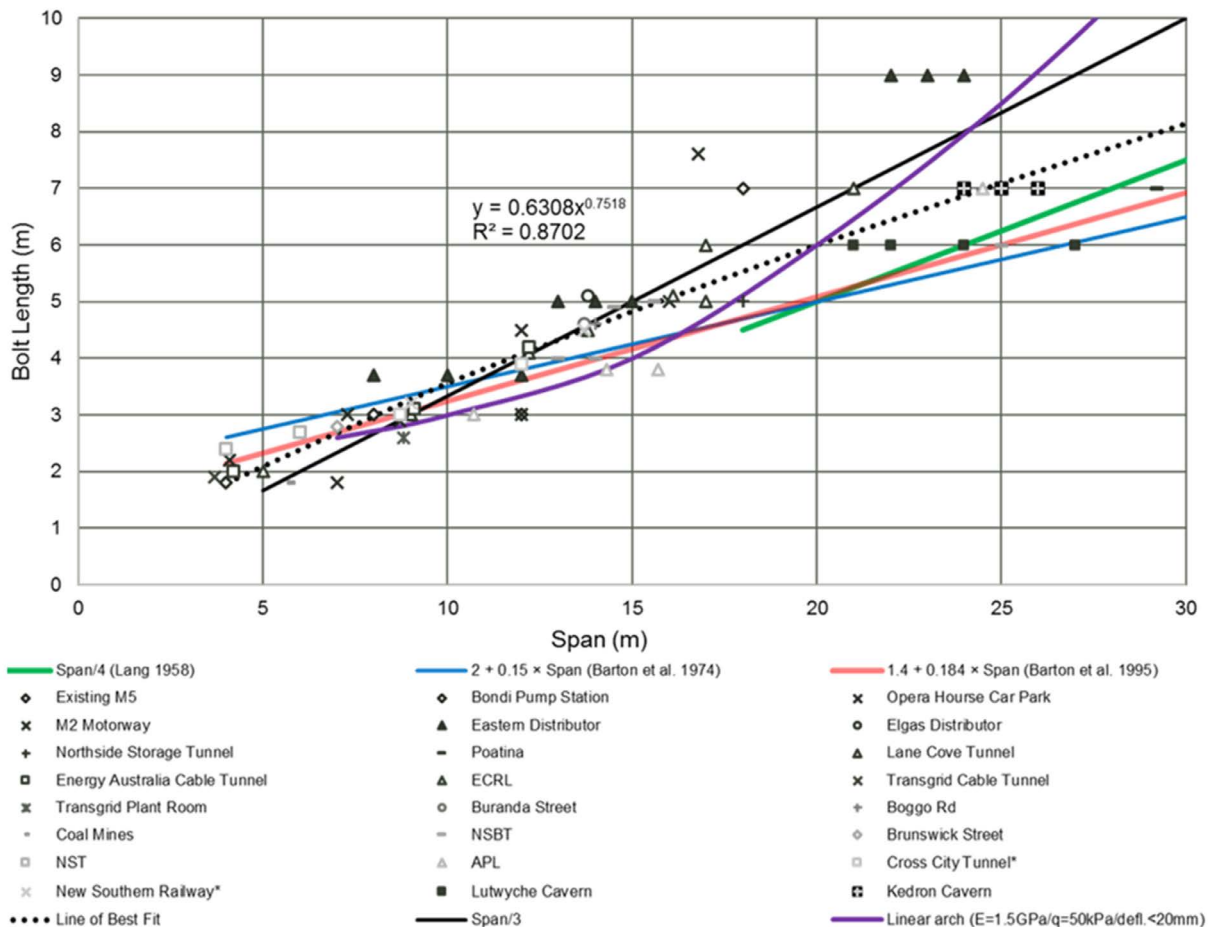


Figure 2. Span versus bolt length for several projects in Australia

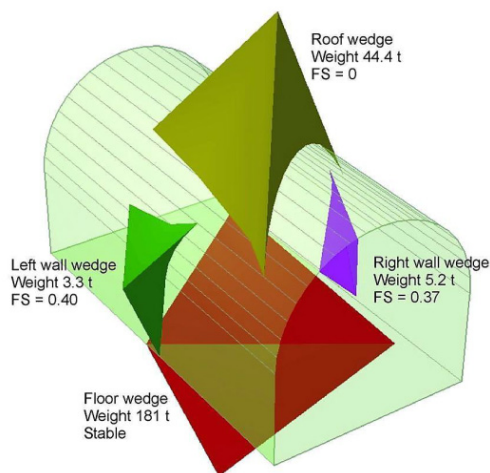


Figure 3. Example of unstable wedge formed in roof with width approximately equal to tunnel span (after Hoek, 2007).

For a 20 m span tunnel, the required bolt length would be approximately 6 m bolt length to achieve a mid-span deflection of approximately 15 mm in accordance with Bertuzzi and Pells (2002) and assuming a fair quality pseudo-equivalent rock beam with  $E = 1500$  MPa. This is somewhat consistent with the best-fit of all projects shown in Figure 2.

The limited data beyond 18 m span indicate a distinct difference from the smaller tunnels. Most of the larger span bolt lengths deviate from both the linear-arch theory curve and the span/3 linear trend and approximately approach the empirical rule originally devised during the Snowy Mountains project in Australia by Lang (1958) for bolt lengths of span/4 (for spans greater than 18 m) and those of Barton et al (1974) and Barton et al (1995). The exceptions are the Eastern Distributor cavern with 9 m long bolts and span up to 24 m and the Lane Cove Tunnel with 7 m bolts for a 22 m span excavation which are in Sydney. These two projects still follow approximately the span/3 or linear arch theory curve as proposed by Bertuzzi and Pells (2002). On the other hand, the other projects are outside Sydney, namely the Poatina Power Station cavern in Tasmania with 7 m bolts for an approximately 29 m span excavation, the Kedron cavern in Brisbane with 7 m bolts for a 26 m span and the Lutwyche cavern with 6 m bolts for an approximately 26 m excavation. The main difference between the larger excavations outside Sydney and the large span tunnels excavated in Sydney is that they are generally all fully arched structures where the tunnels in Sydney are semi-flat roof tunnels, i.e. very low arch tunnels. Stress arching and combined use of passive shotcrete support in arched excavations may allow for shorter bolts than what would have been used in semi-flat-roofed tunnels in Sydney using the Bertuzzi and Pells (2002) approach as depicted in Figure 2. As a result, such difference must be considered in design.

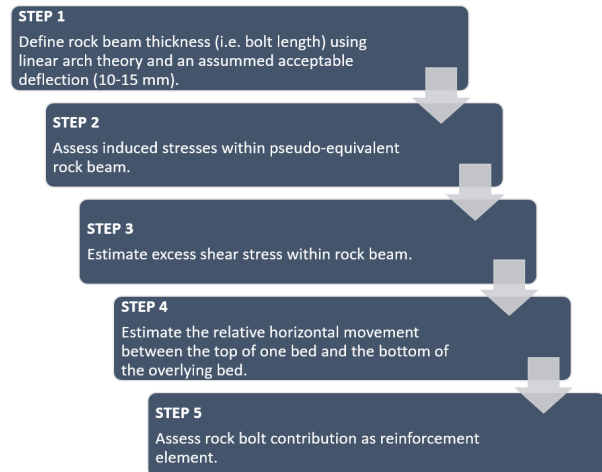


Figure 4. Rock beam design steps (after Bertuzzi and Pells 2002).

### 3 WHAT CHANGES IN LARGE SPAN TUNNELS?

The precedent design above already provided some initial indication that in large span tunnels, bolt length may not follow similar trends to those of the smaller span tunnels. Two main factors are considered to influence this change: stress effects and a change in design focus to reinforcement effects rather the suspension of unstable wedges.

#### 3.1 Stress effects

As previously discussed, for tunnels where instability is generally controlled by geological structures, the rule of thumb for bolt length is somewhat related to conservative assessments of the largest possible wedge assuming ubiquitous rock defects and ignoring stress effects. These assessments generally result in wedge widths that span approximately the entire tunnel and wedge apex heights approximately equal to one third of the span. Making similar assumptions in large tunnels and caverns, very large wedges would therefore be found in the roof which would require excessively long cable bolts.

Figure 5 illustrates an unstable wedge, i.e. factor of safety  $FoS = 0$ , with an apex height of approximately 12 m within the roof of a 40 m span cavern. Such a wedge would require cable bolts lengths of approximately 12-13 m. Firstly, the designer must question if such a large-scale wedge is indeed reasonable based on the typical persistence of the controlling discontinuities. Secondly, an important assumption typically ignored may be reconsidered which is the positive clamping effect of horizontal stresses. If a stress state equal to the in-situ stress is assumed, despite horizontal stress likely increasing due to the excavation, the wedge would show a stable behaviour with a  $FoS = 1.7$ .

Hoek (2007) states that this large difference in safety suggests a tendency for sudden failure when the in-situ stresses are diminished for any reason and is a warning sign that care must be taken in terms of the excavation and support installation sequence. For this reason, many tunnel designers consider that it is prudent to design the tunnel support on the basis that there are no in situ

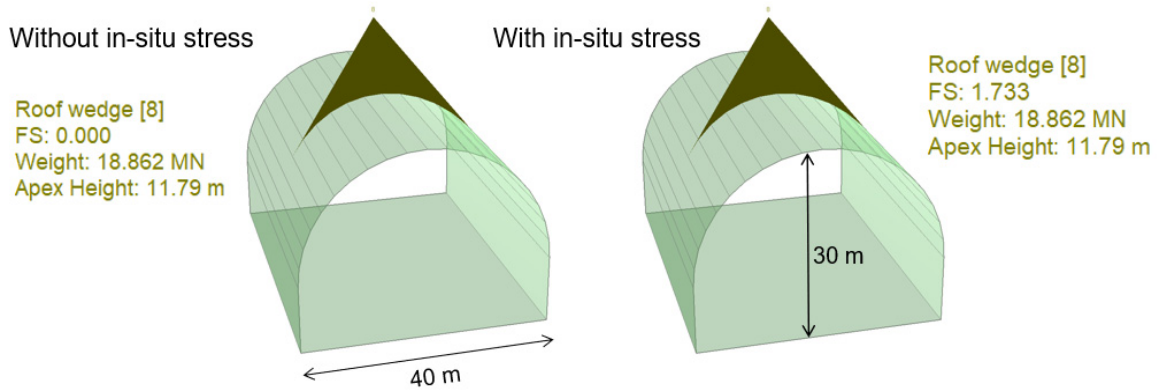


Figure 5. Example of the effect of in-situ stresses (40 m deep and  $k_0 = 1$ ) on a large unstable wedge in a large span and unsupported cavern.

stresses ensuring that, for almost all cases, the support design will be conservative.

Although some level of conservatism is generally desirable, one main difference should be considered in the support of large span tunnels and caverns. In almost all cases, these large span excavations will be sequentially excavated such that large deformation that could induce wedge dislodgment would typically be controlled and some anchorage and shear reinforcement would already be in place prior to full exposure of the largest possible wedge, making the use of the positive clamping effect of stresses more amenable. Figure 6 depicts this scenario.

An example of such large span with considerably shorter bolts in comparison to the excavation span is the Gjøvik Olympic Hall in Norway with a record-breaking span in civil tunnels of approximately 61 m which only used a maximum bolt length of 12 m (Broch et al, 1996).

### 3.2 Reinforcement effect

Another important factor is the change in focus from an anchorage or suspension effect of rock bolts to the reinforcement effect within the tunnel roof.

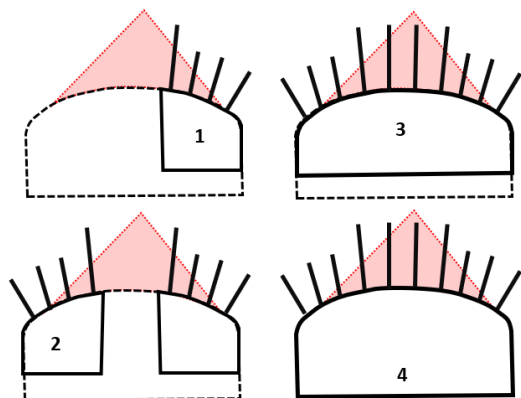


Figure 6. Example of the effect of sequential excavation on large wedge support combined with the clamping effect of stresses.

### 3.2.1 The design myth

There is significant misunderstanding and design myths on how rock bolts provide such reinforcement effects. To this end, Pells (2008) stated that rock bolts are sometimes ascribed abilities that verge on magic. For example, they are said to prevent stress induced failure, or said to interlock a rock mass like aggregate. The latter is based on Tom Lang's famous 1960's upside-down bucket experiment during construction of the Snowy Mountain project which initiated a design myth that is often misused. Tom Lang's experiment intended to demonstrate to the Snowy workers how rock bolts work.

Evert Hoek visited the Snowy project in the 1960's and he was impressed with such a demonstration (Hoek, 2007). He then started using his own version for teaching purposes at the University of Toronto (Hoek, 2007). In Hoek's version, he indicated that a zone of compression is induced in the region shown in red in Figure 8 and this provides effective reinforcement to the rock mass when the rock bolt spacing,  $s$ , is less than 3 times the average rock piece diameter,  $a$ , and the length,  $L$ , approximately  $2s$ .

This concept of a compression zone promoted by rock bolts was then extended to tunnels as a rock bolt compression ring concept around the opening by Pender et al (1963) as depicted in Figure 9.

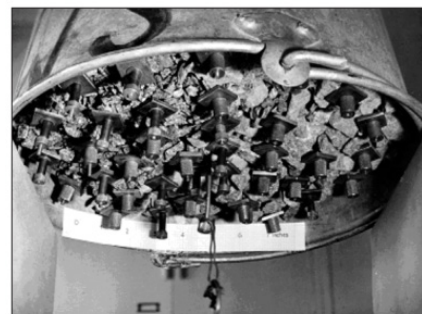


Figure 7. Lang's Bucket (after Pells, 2008).

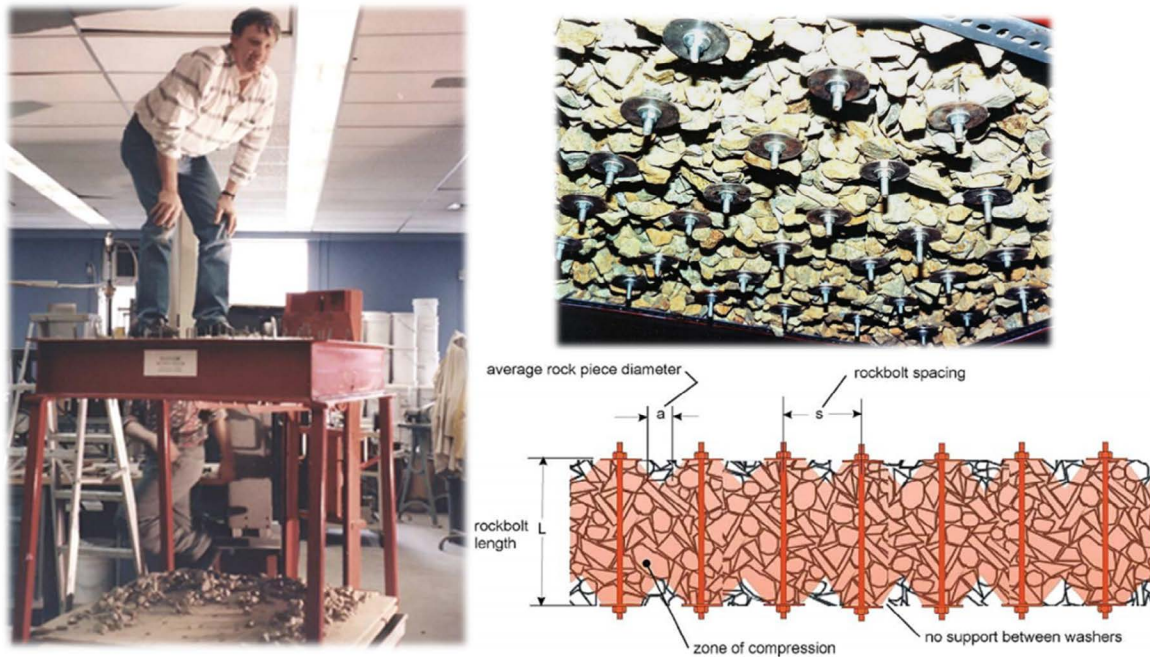


Figure 8. Hoek's educational version of the upside-down bucket experiment (after Hoek, 2007).

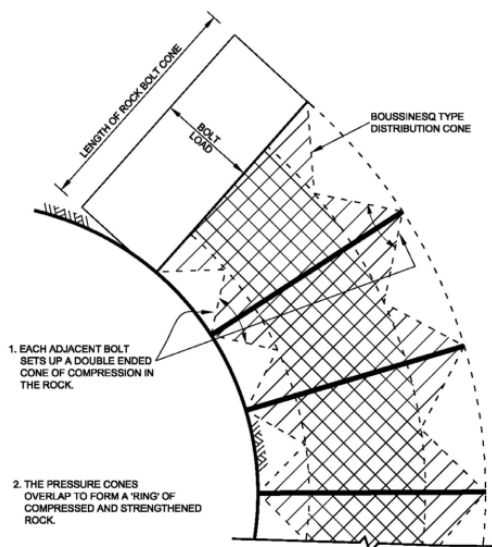


Figure 9. Overlap of rock bolts compression zone forming a compression ring around the tunnels (after Pender, 1963).

The above concept could be said relatively intuitive and simple to explain how rock bolts work. However, Pells (2008) demonstrated that Tom Lang's bucket experiment and consequently Hoek's gravel table have little relationship to the action of rock bolts around a tunnel because the stress scale is all wrong. In these experiments the confinement or compression provided by the bolts is of the order of a few kilopascals which is too small, by several orders of magnitude, to have any effect on the rock mass strength by "confinement" in a tunnel scale. In addition, although Figure 9 has been reproduced in many text books, Pells (2008) also demonstrated via analysis that similar scale issue is observed with this concept as shown in Figure 10.

### 3.2.2 How it really works

Despite the stress scale issue, Hoek's experiment can still be used to demonstrate how rock bolts work. Not necessarily due to a confinement effect which is relevant at the scale of the Hoek's experiment but in fact due to its reinforcement effect allowing stresses to arch linearly to the abutments as shown in Figure 11. In fact, such concept is not different to the concept of the linear arch theory used in laminated rock beam design as depicted in Figure 12.

#### Flat or semi-flat-roofed tunnels

For flat or semi-flat roofed tunnels, Oliveira and Paramaguru (2016) presented an approach for the design of rock reinforcement in laminated rock beams where the focus is on satisfying the development of the compressive arch like the one presented in Figure 12 within a pseudo-equivalent thicker voussoir beam. The rock bolt reinforcement is designed to provide the necessary capacity to overcome the excess shear stresses in the bedding partings, thus stitching thinner beams together into an equivalent thicker rock beam that can control deflections (Figure 13).

It should be noted that the iterative design approach proposed Oliveira and Paramaguru (2016) does not define the rock beam thickness and consequently rock bolt length based on deflections defined as acceptable prior to the analysis. In their approach, deflections are an output, not an input, with the primary objective of the rock bolting design approach proposed by the authors being the development of the compressive arch within the "stitched" pseudo-equivalent rock beam. This is achieved by assessing the excess shear forces developed within the equivalent beam and comparing against the mobilised rock bolt forces estimated through methods such as those presented by Pells (2002). This approach has in general allowed for a reduction of bolt lengths in recent projects when comparing to precedent designs as the effect of bolt spacing is taken into

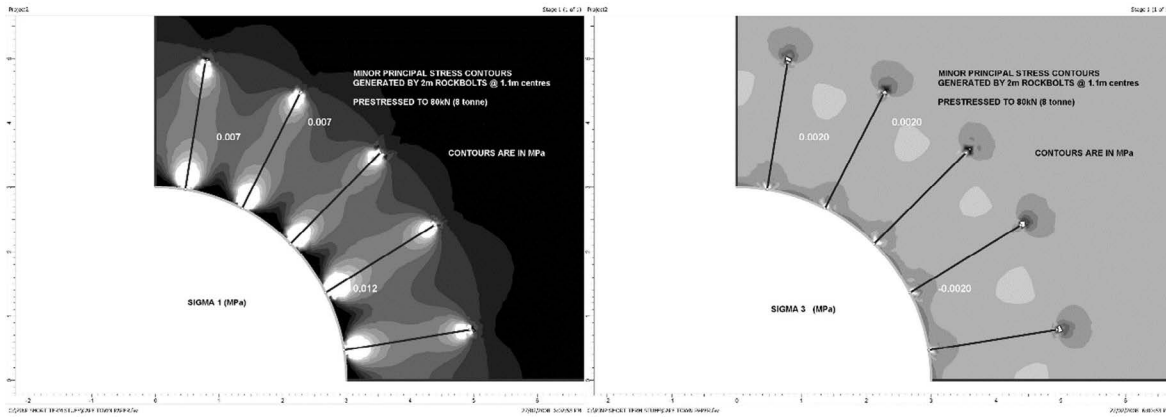
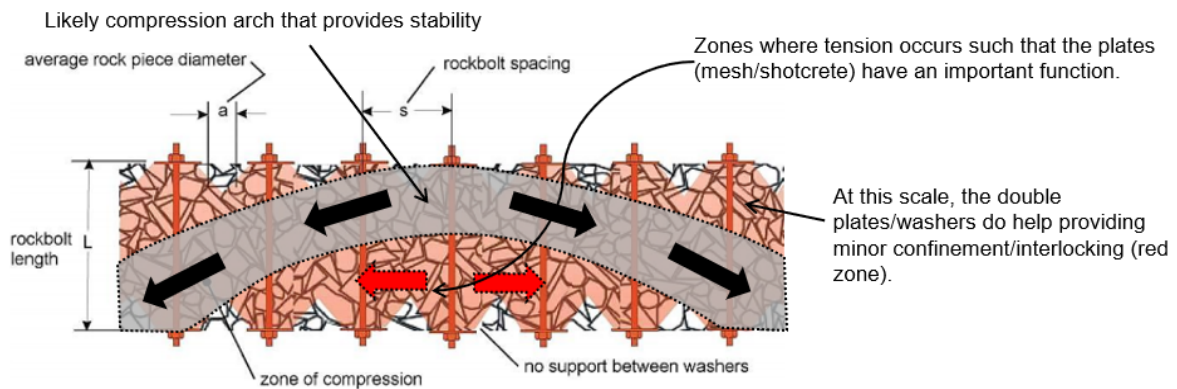


Figure 10 Contours of major and minor principal stress induced by 2m rock bolts at 1.1m centres pretensioned to 80 kN (after Pells, 2008).



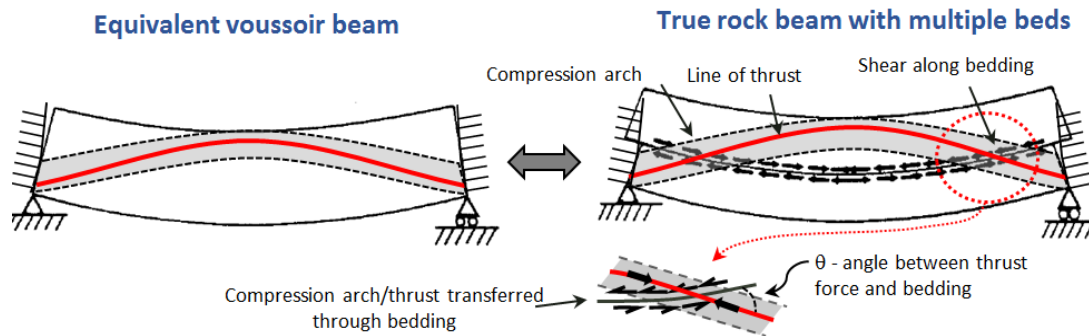


Figure 13. Reinforcement effect for equivalent rock bolt stitched rock beam (after Oliveira and Paramaguru, 2016).

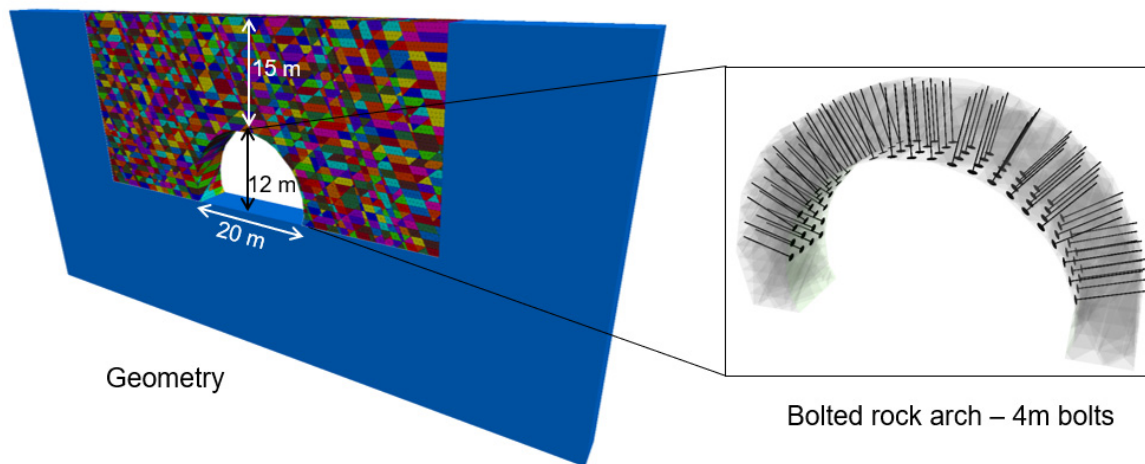


Figure 14. Model of a 20 m span arched shallow tunnel in fair to poor quality siltstone rock.

face with no further significant deflections observed in the tunnel crown.

It should be noted that the large deflections observed in the rock bolted case of Figure 15 would be further reduced if additional passive support such as a thick layer of shotcrete (e.g.  $t > 300$  mm) is applied which is generally the case in shallow tunnels. Shotcrete would particularly control the fall-out of wedges between bolts and reduce overall rock loosening.

Based on the discussion above, the positive effect of the rock reinforcement in promoting a more stable stress redistribution is evident through the comparison between the ground reaction curves and equivalent deflections at several points behind the excavation face.

The improvement in stress arching promoted by the bolts becomes even more evident when comparing the stress tensor within the bolted-arch for the two cases. This is illustrated in Figure 16 with the stress tensor coloured with respect to the magnitude of the major principal stress (Minimum Principal in 3DEC as compression is negative). The major principal stress indicates the ability of the rock to redistribute or arch the stresses around the tunnel. For the unsupported case, a large zone in the immediate roof of the tunnel is observed to be in tension or at zero stresses at a boundary traction equivalent to 10% of the original in-situ stresses which confirms the loss of support and associated collapse. On the other hand, the rock bolted case indicates that the major principal stress in the immediate roof of the tunnel is approximately 600 kPa at a similar 10% boundary traction stage, locally increasing

to approximately 1 MPa. In fact, the zones with 1 MPa stresses indicate the likely line of thrust associated with stress arching.

An analogy between conventional structural tunnel support and the overall effect of rock reinforcement could then be made by treating the reinforced rock beam or reinforced arch as structural elements where loads are applied with appropriate factors to test compliance to design standards.

For example, using the previous example, a load equivalent to approximately 15% of the original in-situ stress, i.e. equivalent to the collapse initiation load, could be assumed and multiplied by a load factor of 1.5 in accordance with AS 5100. Such loads could then be applied to a bolted rock arch modelled isolated from the rest of rock mass to verify its ability to promote the stress arching and form the line of thrust, thus, providing support to the overlying rock mass as illustrated in Figure 17. In this case, the structural capacity of the rock bolts would also have a reduction factor applied to satisfy code compliance.

It is important to note, that similarly to the approach adopted for rock beam design (Oliveira and Paramaguru, 2016), the bolted-arch should first be allowed to deflect elastically while maintaining a non-zero tensile strength within the joints. This initial elastic deflection is assumed equivalent to the gradual excavation mechanism so that some deformation occur before the roof is fully excavated and formed.

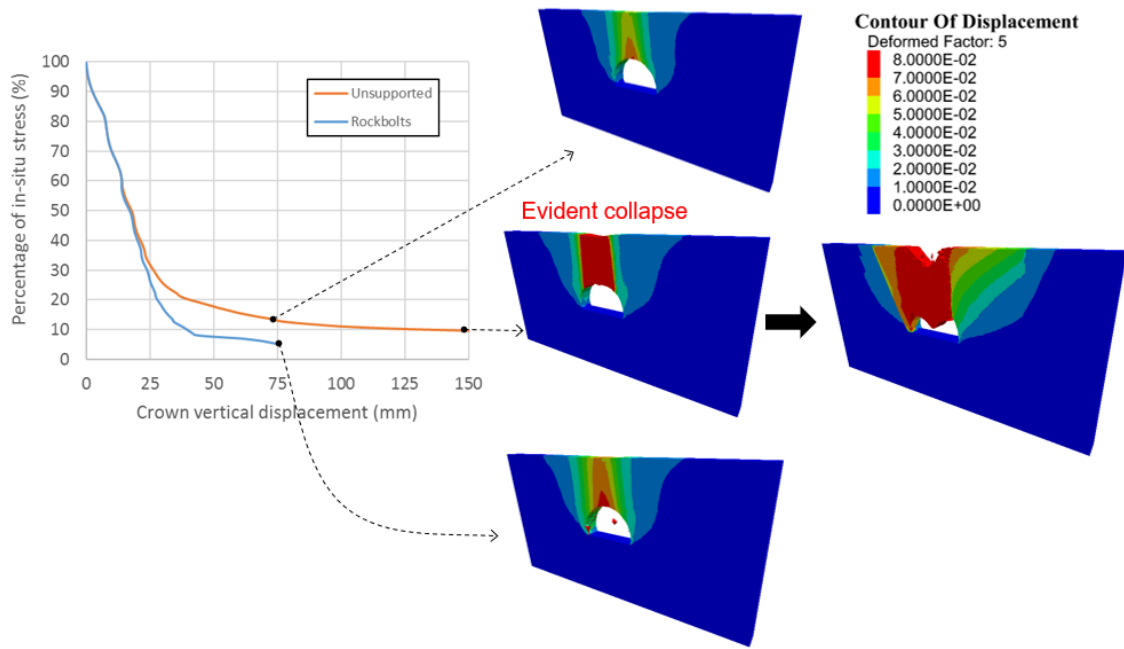


Figure 15. Ground reaction curve and representative behaviour with respect to roof displacements.

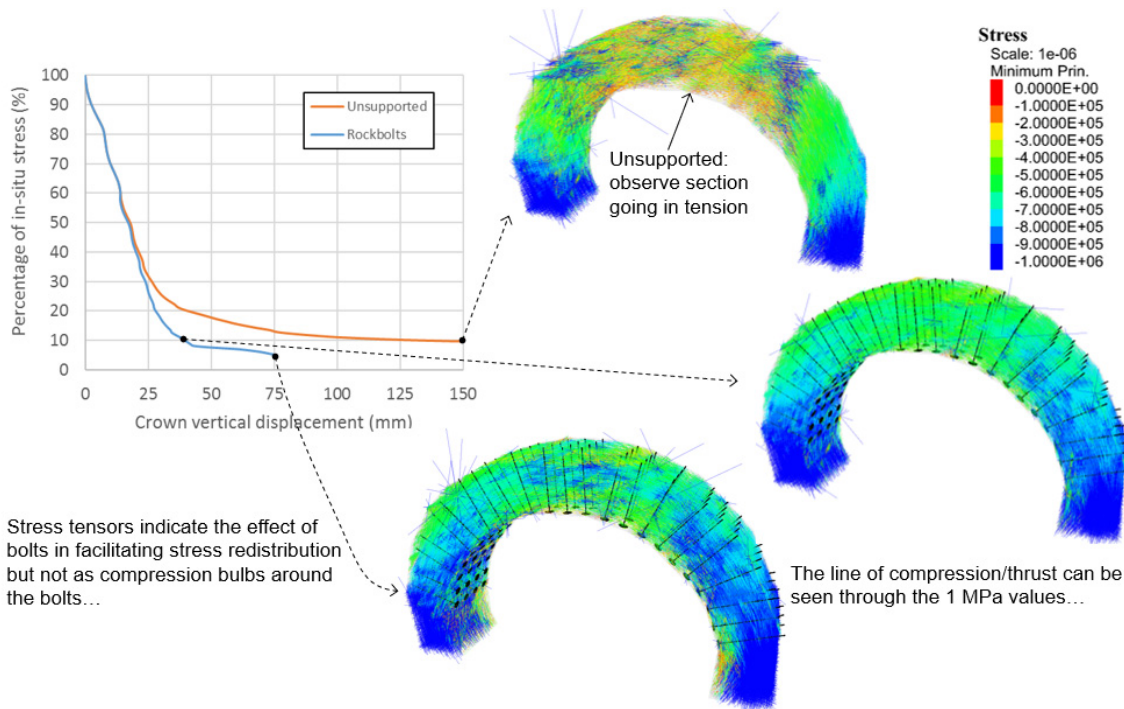


Figure 16. Ground reaction curve and representative behaviour with respect to stress tensor within "bolted-arch".

After the initial elastic equilibrium is achieved, the joint tensile strength should be set to zero and the arch allowed to continue deforming until either equilibrium or failure occurs.

**4 UPDATED EMPIRICAL CHART**

Figure 18 presents an updated empirical design chart including recent projects where some of the concepts discussed above have been applied. There is an evident increase in points below the previous precedent design trends and approaching the low bound curves equal to

span/4 for spans greater than 18 m as proposed by Lang (1958) and the curves proposed by Barton et al (1974) and Barton et al (1995).

For example, long tunnels with spans of up to 20 m have been recently designed with bolt lengths of 5 m in contrast to the precedent design of 6 m long bolts. Although this may seem a small reduction at first, in a long tunnel, this results in savings of several kilometres of rock bolts which include drilling and grouting and the associated time required to install a large number of them.

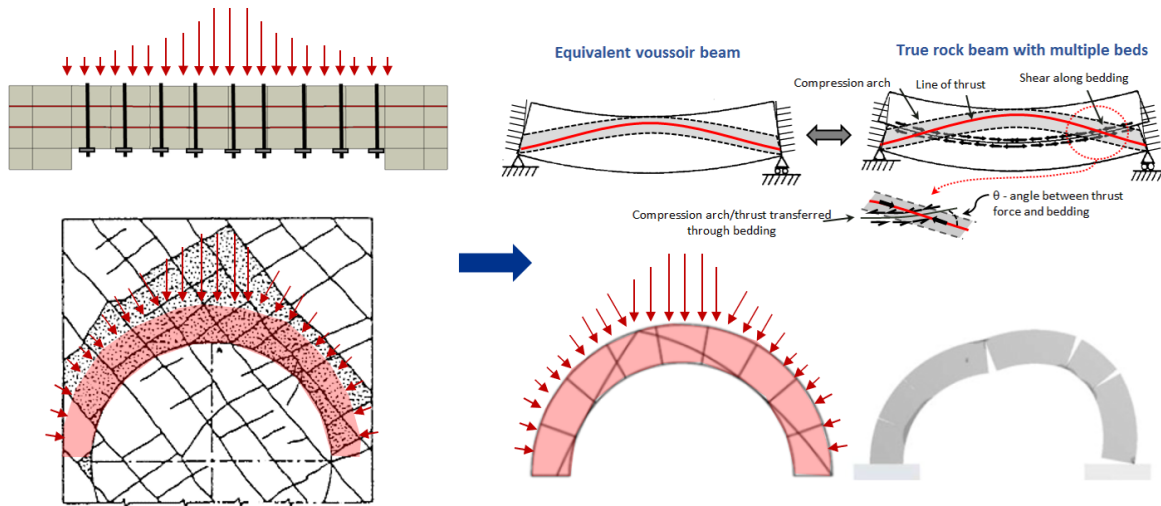


Figure 17. Bolted-arch zone analogue as main structural tunnel support.

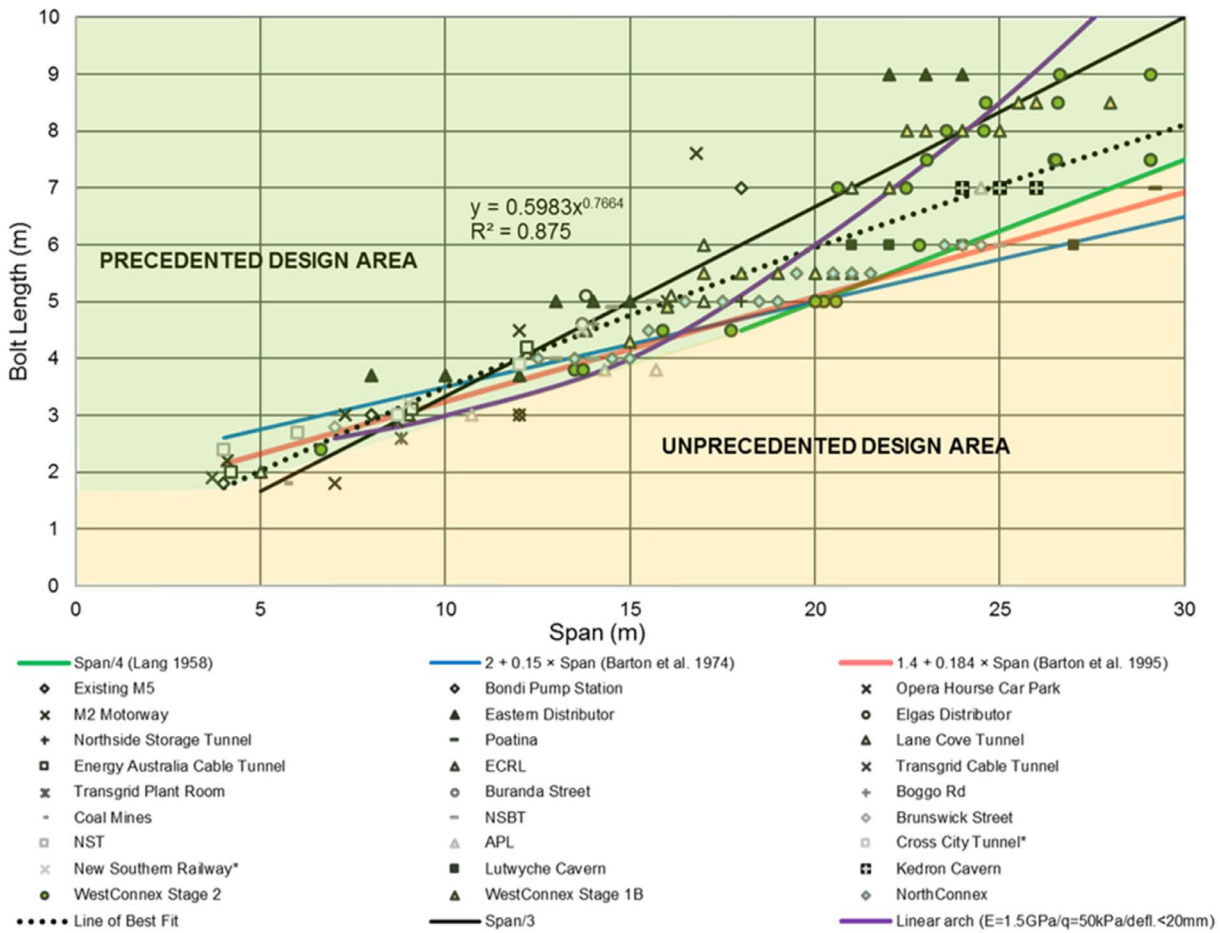


Figure 18. Current precedence including recent projects in Australia.

### 5 CONCLUSIONS

This paper presented a discussion on the design of large span tunnels and caverns with particular reference to the Australian experience. Some precedent design was presented and demonstrated that, although very valuable, past projects are not the only single source of design.

Design efforts that focus on first principles and the basic objectives of rock reinforcement can overcome the

challenges of these designs, satisfy codes and standards requirements but at the same time provide savings with respect to ground support.

The key to the design involves understanding the key failure mechanisms that needs to be addressed, its relationship with the different actions of rock bolting, i.e. suspension/anchorage and/or rock reinforcement and what could be considered acceptable for design.

The compression promoted by rock bolts is often seen in text-books as an oversimplification that does not represent a realistic case. However, rock bolts do facilitate the development of a compression arch resulting from stress redistribution within the rock arch or rock beam. When properly designed, the rock bolts allow the line of thrust to be transferred across discontinuities such that it maintains compression but such compression is still from the “rock formation”

Code compliance can then be verified with simplified models based on the reinforced rock arch or beam

## 6 ACKNOWLEDGEMENTS

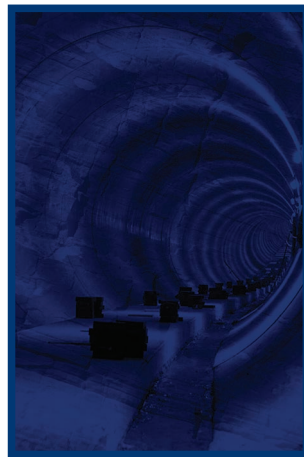
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