IN SITU ROCK STRESS AND ITS EFFECT IN TUNNELS AND DEEP EXCAVATIONS IN SYDNEY

L. B. McQueen
Golder Associates, Sydney

1 INTRODUCTION

The behaviour of rock surrounding an underground opening depends primarily on the strength of the rock mass and the level of stress existing in proximity to the excavation. As the excavation is made, a redistribution of the virgin stress field occurs, leading generally to a concentration of stress parallel to the wall, and a relief of stress normal to the wall. If the concentrated stress magnitude is high enough relative to rock strength the rock mass can dilate or fracture. Likewise, the relief of stress normal to the wall can lead to loosening and unravelling of blocks of rock bounded by discontinuities in the wall.

The role of the virgin *in situ* stress field has become well recognised in the Sydney area over recent years. Most geotechnical investigations for mining and civil engineering projects in the Sydney Basin will include a consideration of this factor, often involving direct stress measurements. This paper gives a brief summary of some of the factors controlling *in situ* stress, the historic evidence for the role of stress in the Sydney area, a discussion of the results of a large number of stress measurement campaigns, and a description of a number of projects undertaken in recent years that illustrate the importance of stress in the Sydney Basin.

2 FACTORS INFLUENCING NEAR SURFACE *IN SITU* STRESS

The *in situ* stress field is influenced by a number of factors. The origin and nature of near surface stress in Australia has been discussed by a number of authors in recent years, for example Brown and Windsor (1990) and Enever and Lee (2000). The total stress at a point in the Earth's crust is generally considered to be made up of the following components:

- Gravity due to the weight of overburden. Gravity also contributes to the horizontal stress due to the Poisson's effect.
- Tectonic component, which could be an active or a remnant tectonic stress from movement of the earth's plates, and generally impacts the horizontal stress field.
- Thermal and physio-chemical effects.

The tectonic contribution to the regional stress field is illustrated within continental Australia in Hillis et al. (1999), Reynolds (2001) and in the Australasian Stress Map, 2004 (Figure 1). Continental Australia lies within the Indo-Australian tectonic plate which undergoes an absolute movement of approximately 7 cm per year to the N-NNE.

The measured stress field has not been found everywhere to reflect this simple model. In south-eastern Australia there appears to be an approximate E-W compressive component that may be related to movements along the Tonga-Kermadec Trench. The measured data suggests that the horizontal stress field in some areas can be relatively high and anisotropic. In general, the degree of anisotropy appears to be somewhat less in the Sydney Basin than elsewhere, leading to the relatively greater impact of local factors on stress field orientation in the Sydney area. Despite this, it is possible to discern some general trends in the measured data for the Sydney region, reflecting a predominant N-NNE orientation closer to the coast and a more E-W orientation elsewhere (Hillis et al., 1999).

Superimposed on those stress components indicated above is the local stress induced effects of topography and discontinuities in the rock mass such as faults.

Topographic effects can have a significant influence on the *in situ* stress field within the depth range of civil engineering works. Features such as valleys, coastal cliffs and palaeochannels (bedrock channels infilled by recent alluvial or marine deposits) can realign the stress field and concentrate or reduce its magnitude. This occurs due to the removal of geological restraint by erosion and has been referred to as stress relief, stress rebound and valley bulging. The results of this include features such as compression faults in valley floors and tension fractures in valley walls, and have long been recognised (Ferguson, 1967; Nichols, 1980 and Nichols & Collins, 1991). In the Sydney region, low angle thrust faulting has occurred due to the elevated stresses below valleys (Mangrove Creek Dam, McNally, 1981;

Blue Mountains, McElroy, 1969; McQueen, 2000) and below palaeochannels (Middle Harbour, Parker et al., 1999). This appears to be confined to the anisotropic geological conditions of interbedded sandstones and siltstones.

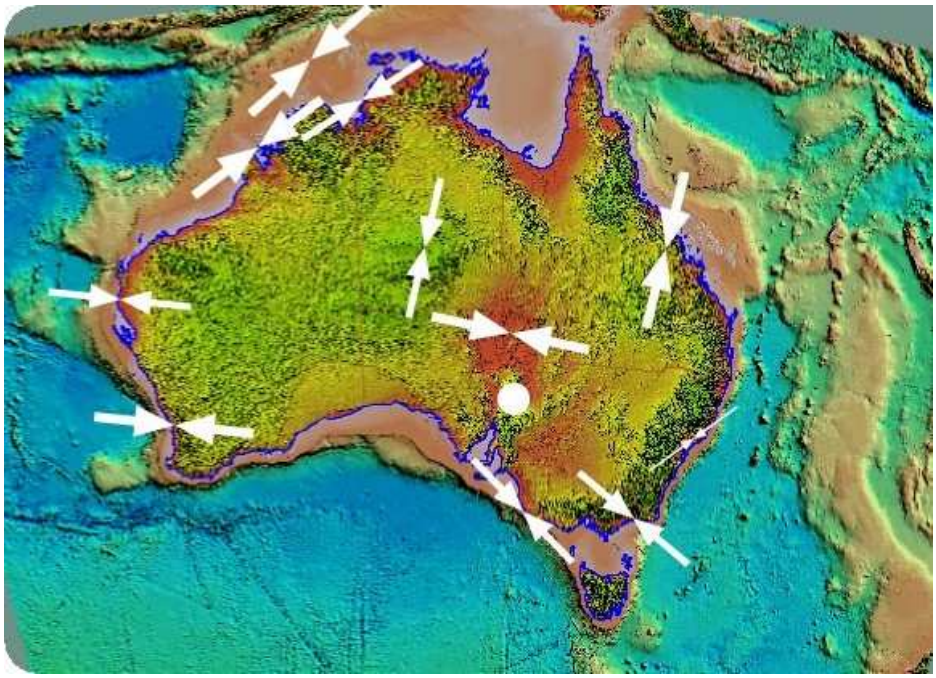

Figure 1: Orientation of Major Horizontal Stress in Continental Australia (from Australasian Stress Map, 2004).

3 HISTORICAL INFLUENCE OF IN SITU STRESS IN CIVIL ENGINEERING

The existence of horizontal in situ stresses at shallow depths, exceeding the vertical stress due to gravity, is a well established phenomena in the Sydney Basin (Chesnut, 1983 and Enever et al., 1984). The adverse effects of these stresses, combined with horizontal bedding, have been demonstrated in various civil engineering excavations.

Prior to about 1970 it was generally believed that high horizontal or locked-in tectonic stress could not be a geotechnical problem in the Sydney Basin, since these rocks were relatively undeformed, weak sedimentary strata. In addition, they had been subjected to long-duration weathering, which should have released any strain energy remaining from distant tectonic events such as the basin's uplift and the formation of structures such as the Lapstone Monocline, Kurrajong Fault and Kulnura Anticline.

The problem of rock mass dilation and spontaneous cracking in Sydney excavations seems first to have been recognised as a stress release phenomenon in about 1970, during construction of deep railway stations in Hawkesbury Sandstone at Martin Place and Town Hall (Branagan, 1985). Rock face damage due to stress relief may have been present in the 1920's tunnelling for the city loop railway, but was not identified as such because it was obscured by more extensive blast damage. Stress induced features are now much more obvious in machine-cut excavations.

The effects of concentration of stresses below the base of valleys was evident during the construction of Warragamba Dam. Cracking and movement along bedding planes (total of 230 mm in a calyx hole) occurred below the valley floor in the Warragamba Dam cutoff (stress measurements for spillway upgrade indicated up to 8 MPa maximum horizontal principal stress below the valley). At Woronora Dam closure of about 25 mm was recorded in the sides of the 50 m deep spillway channel, which crushed bridge footings spanning the excavation (Gray, 1982). Both dam sites are in Hawkesbury Sandstone.

Stress induced problems were certainly acknowledged during site investigation for the F3 Freeway cutting at Kangy Angy in the mid-1970's (Nyland and Williams, 1974). When this 60 m deep cutting was excavated in 1977, it was made as steep and narrow as possible to minimise the visual impact of the white sandstone gash against the surrounding grey-green bush. Although not the deepest freeway cutting at that time, it was excavated in the very strong (100 MPa) and thickly-bedded Wyong Sandstone Member of the Terrigal Formation, a much stiffer and less weathered unit than the Hawkesbury Sandstone. Some months after excavation, large quantities of rock spall were noted on the road shoulders below and eventually the freeway had to be narrowed from six lanes to four to create a rock trap. It remained

this way for 20 years, until stress relief was considered complete. The cutting was found in 1977 to have closed up by 400 mm in places and had horizontal movements along shale partings of 100 mm (Chappell et al., 1984).

Further evidence of stress relief along the F3 included inward movements of abutments on the Pacific Highway overbridge above the Mooney Mooney cutting. This bridge, built in a shallow trench prior to excavation of the cutting to about 60 m depth in Hawkesbury Sandstone, closed up so that the deck expansion joints ceased to provide any gap. Eventually the end of the deck had to be trimmed to allow for thermal expansion (J.R. Williams, RTA, pers comm.). Closure of bridge abutments due to rock excavation were also reported for the Warringah Expressway (Pells, 1990).

Early examples of tunnels where stress fracturing was recognised as a result of stress concentration effects from the high horizontal stress, include the Boomerang Creek Tunnel, a 11 km TBM-bored tunnel, up to 300 m deep in the Narrabeen Group west of Gosford (Enever et al., 1990); the Pipehead to Potts Hill tunnel (Bringelly Shale) in the early 1970s; and the Malabar Ocean Outfall Tunnel (Hawkesbury Sandstone and upper Narrabeen Group) in 1985/6. The Malabar and Potts Hill tunnels are included in the case studies later in this paper.

The high horizontal *in situ* stresses in Sydney have also influenced the construction of deep excavations for building basements in the Sydney CBD. Deep excavations have experienced stress effects resulting in displacement along bedding features causing lateral inward movements of the sides (Pells, 1990; Braybrooke, 1990). The construction of deep excavations adjacent to City rail tunnels and other deep basements has resulted in varying stress concentration and stress relief features as discussed in the case studies.

4 MEASUREMENT OF IN SITU STRESS

4.1 DIFFICULTIES AND LIMITATIONS

Stress is not a simple quantity to visualise or measure like a scalar (defined by a single value) or vector (magnitude plus direction). Stress at a point is a tensor quantity and needs six independent components for definition, viz. three normal and three shear components. It has magnitude, direction and orientation with respect to reference axes. If the orientation of the axes changes then the values of the six components change. In situ rock stress is generally described in terms of principal stresses. These are the three normal stresses remaining after the principal stress axes are orientated until the shear stresses on the three principal planes perpendicular to the axes are zero.

Hudson (1993) highlights the complexities of measuring rock properties, particularly, in situ stress, "*if we are attempting to measure a particular property and it depends on a variety of other properties, then how can we know that we have actually measured the rock property itself in isolation?*" He illustrates this by pointing out the following factors, which can severely affect in situ stress measurement:-

- The theory used in most methods to calculate the stress components from the strain/deformation measurements, assumes homogeneous, isotropic, linear elastic materials. Some solutions can take anisotropy into account.
- Rock mass structure such as faults and joints may cause a change in magnitude and local rotation in the stress field.
- Inhomogeneous rock will have different localised stresses.
- Topography, including infilled valleys, may affect the magnitude and orientation of the local stress field.
- Rock anisotropy, e.g. interbedded rocks of varying stiffness, will influence local stress magnitude.
- Measurement error, e.g. due to temperature differences between the rock and drilling water. Strain should be measured at the ambient temperature of the rock. The limitations of various measurements have been well documented, e.g. Brown and Windsor (1990).

4.2 TECHNIQUES

The primary techniques of stress measurement in Sydney have been hydraulic fracturing and overcoring. Other techniques used include the borehole 'slotter' (no results were available for inclusion in this paper). Brief descriptions of these techniques follow:

1. **Hydraulic Fracturing** (Brown & Windsor 1990, Enever et. al., (1984), is generally used at the investigation stage since it can be carried out in boreholes over the range of depths of civil construction. The method assumes that the borehole axis is parallel to a principal stress direction. In a vertical borehole the minor and major horizontal

secondary principal stresses (σ_h, σ_H) are calculated and the vertical stress (σ_V) is estimated from the weight of the overburden.

The method involves isolating a section of the borehole by packers and increasing the fluid pressure until a fracture is generated in the borehole wall. Massive rock, free of discontinuities, is required to create a fracture. Various borehole diameters ('B', 'N' and 'H' sizes) can be tested. The 36 mm diameter CSIRO minifrac system, which uses the same techniques and calculation methods as the larger version, has also been used in Sydney.

A successful test fracture is normal to the borehole axis, and for a vertical borehole, normal to σ_h. The shut-in pressure (pressure in the fracture when the system is sealed) represents the magnitude of σ_h. The fracture initiation pressure (breakdown pressure) and the re-opening pressure along with σ_h are used to calculate σ_H. An impression packer is used to determine the orientation of the fracture and the principal stress axes are deduced. Laboratory mini-hydraulic fracture tests on core samples from the test section are required to obtain parameters for the calculation. It is desirable to assess any problems (breakout) in the borehole wall at the test section by borehole caliper log (tool used in downhole geophysics) or borehole imaging prior to testing.

2. **Overcoring Methods** – Overcoring measurements are usually carried out from initial underground drives, and are restricted to borehole depths of up to 20 - 30 m. Strain due to stress relief from overcoring is measured by a stress cell, which allows the determination of the complete stress tensor. The cell used is typically the CSIRO Triaxial Cell and also, more recently, the ANZI (Australia New Zealand Inflatable) stress cell:

    - CSIRO Triaxial Stress Measurement Cell (Worotnicki 1993) – This cylindrical cell with 12 strain gauges was developed in the early 1970's. It is epoxy cemented into a predrilled hole (EX 38 mm diameter) and the stresses in the rock around the cell are relieved by overcoring with a larger diameter drill bit (150 mm, or more recently 92 mm diameter). The resulting borehole wall deformation is measured by the strain gauges to allow calculation of the complete state of stress. The section of overcored rock is removed from the hole for biaxial testing to measure Young's Modulus and Poisson's ratio. Temperature effects on strain measurements can be offset by measuring the ambient rock temperature and then controlling the temperature of the drilling water used for overcoring (Max Lee, AMC, pers comm.).

    - ANZI (Australia New Zealand Inflatable) stress cell – (Mills, 1997) describes the design and operation of the ANZI cell. Developed in the early 1980's, it was designed for three dimensional *in situ* measurement in coal. It is a pressuremeter design of 29, 56 and twin 56 mm diameter, with 18 strain gauges which are pressure bonded directly to the borehole wall. Prior to overcoring, a pressure test is carried out to confirm operation of the gauges and measure *in situ* modulus. As with the CSIRO cell, elastic parameters of the overcored rock are measured by biaxial testing. The application of this cell has included the Cataract Tunnel and Northside Storage Tunnel.

3. **Borehole 'Slotter'**. This is a 2D stress relief method, developed at James Cook University and first used in Queensland in the mid 1980's. A slot is cut in a borehole wall and the tangential strain resulting from the release of the tangential stress in the borehole wall, is measured with a recoverable strain gauge. A circular saw cuts at least three half-moon shaped slots (about 0.8 mm wide and 32 mm deep) at 120° apart in a HQ borehole wall (Bock 1993). An additional three slots are normally cut about 100 mm from the first. The induced tangential stress, and hence the *in situ* stress, is calculated by the Kirsch solution (for homogeneous, isotropic, elastic material). Young's modulus and Poisson's ratio are required from laboratory testing of the core from the test location.

In situ stress may also be estimated by back analysis of tunnel deformations. Convergence and/or extensometer measurements in the roof and walls have been used to estimate the horizontal to vertical stress ratio, with the assumption that the tunnel alignment is normal to one of the horizontal principal stresses.

5 IN SITU STRESS FIELD

There is a significant database of in situ stress measurements in the Sydney Sedimentary Basin from the coal mining and civil engineering industries. Compilations of this data have been made, and the stress fields discussed, in Brown and Windsor (1990), Enever et al., (1990) and Enever (1999). Measurements from civil engineering projects in Sydney within the 0 to 200 m depth range are summarised in this section. Figure 2 presents the orientation of the measured major horizontal principal stress along with major structural geological features of Bembrick et al., (1973). Figures 3 and 4 show the magnitude of the major and minor horizontal principal stress with depth, and the ratio of major to minor horizontal stress is shown in Figure 5.

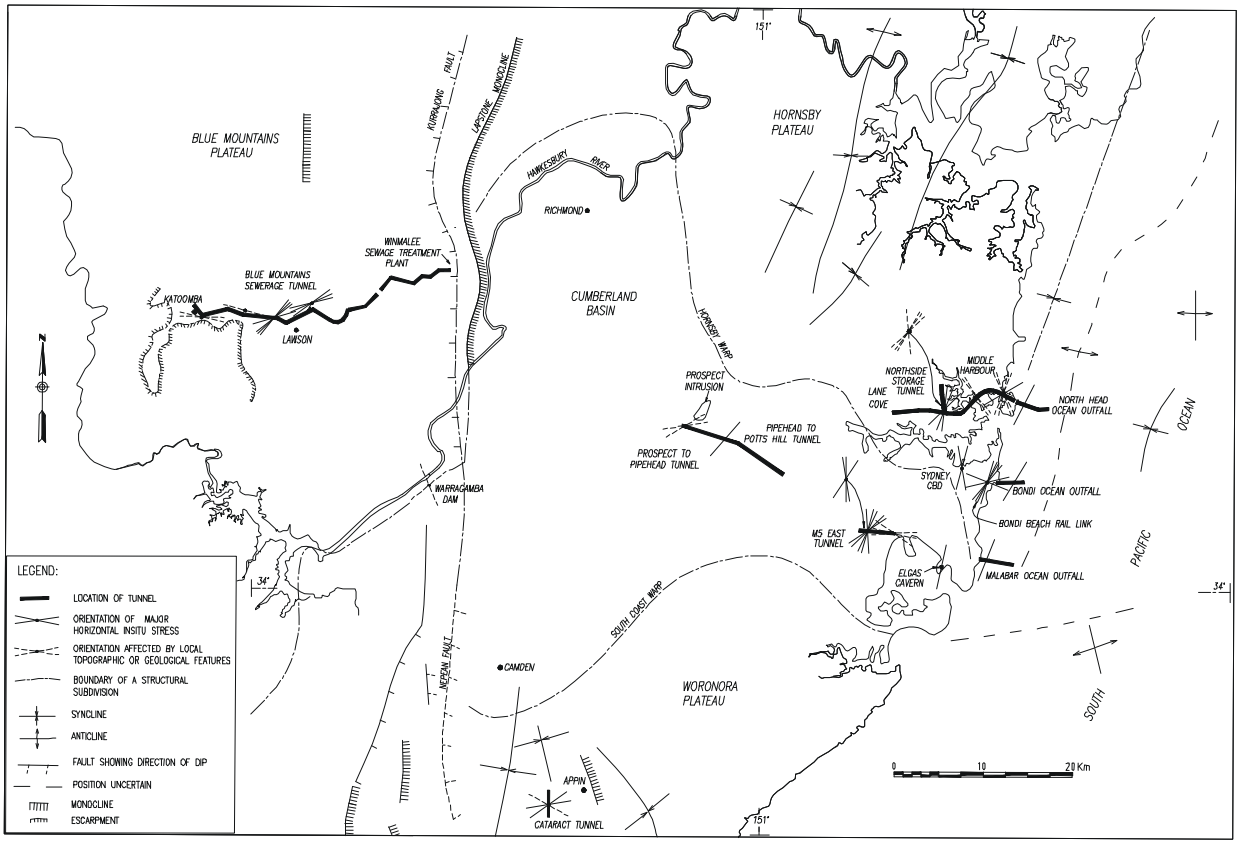

Figure 2: Major Horizontal Stress Orientation. Structural geological features from Bembrick et al. (1973).

Most of the stress measurements in these figures have been obtained by the hydraulic fracturing technique or by overcoring using the CSIRO triaxial cell. The data are mainly from the Hawkesbury Sandstone, with some from the upper part of the Narrabeen Group (some data from the ocean outfall tunnels, Blue Mountains tunnel and the 3 deepest results from the Northside Storage Tunnel) and the Bringelly Shale (Prospect to Pipehead Tunnel). The upper Narrabeen Group includes the Newport Formation, Bald Hill Claystone and Bulgo Sandstone and their western Sydney Basin stratigraphic equivalents, the Burralow Formation, Wentworth Falls Claystone and Banks Wall Sandstone, respectively.

A current assessment of the horizontal stress field in the Sydney metropolitan area is given by Enever (1999). His findings for the area between Botany Bay and Port Jackson and extending 7 km inland, in the depth range 0 to 200 m, are:

- The horizontal stress consists of a gravity component, equivalent to the weight of cover, plus a tectonic component. The magnitude of the tectonic component for 20 to 200 m depth for the major horizontal principal stress, ranges from zero to 6.5 MPa above overburden stress (Figure 3). The suggested upper bound line for the minor horizontal principal stress is the overburden stress plus 4.5 MPa (Figure 4). Also shown is the upper bound of the tectonic stress component for 0 to 20 m depth, viz. 2.5 MPa above overburden stress for the major horizontal principal stress and 2 MPa above overburden for the minor horizontal principal stress.
- The major horizontal stress is oriented mainly northeast between 000 and 050, and does not change systematically with depth.
- The ratio of the major horizontal stress to the minor horizontal stress is 1 to 2.
- There is no clear relationship between horizontal stress orientation and magnitude.
- Topography can have a major effect locally.

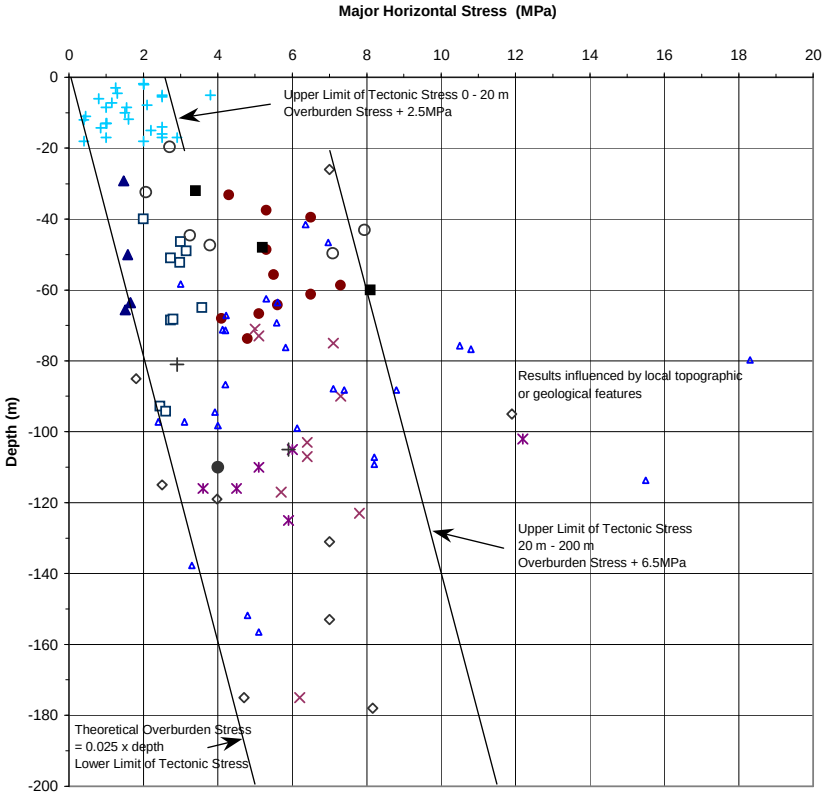

Figure 3: Major Horizontal Stress vs Depth.

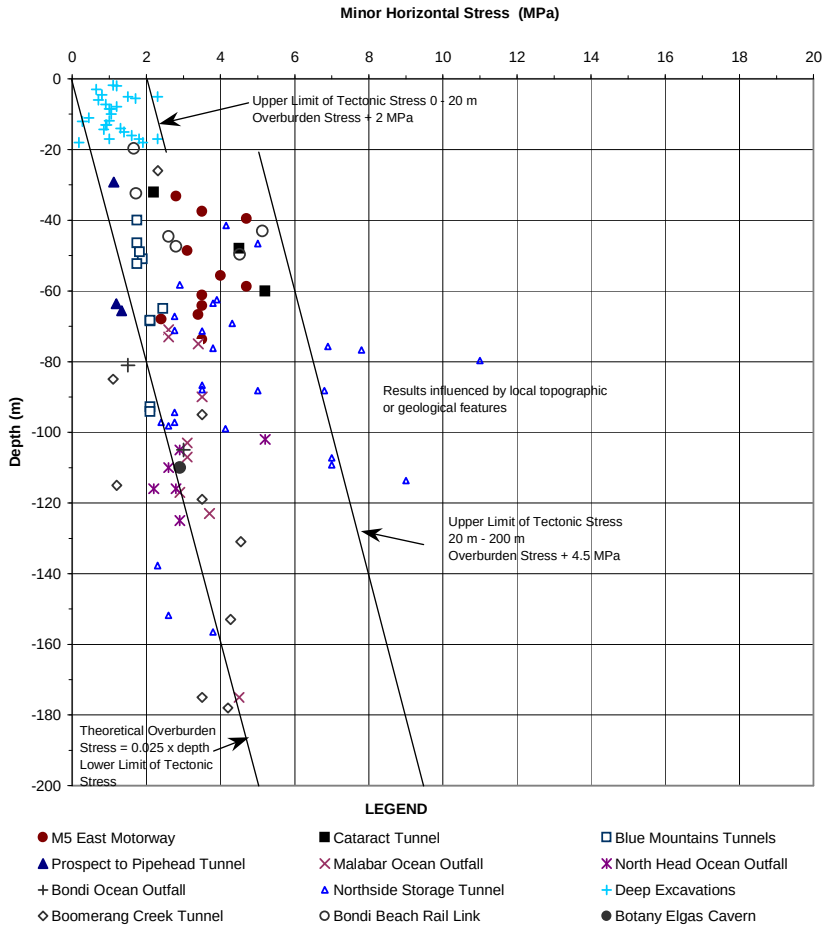

Figure 4: Minor Horizontal Stress vs Depth.

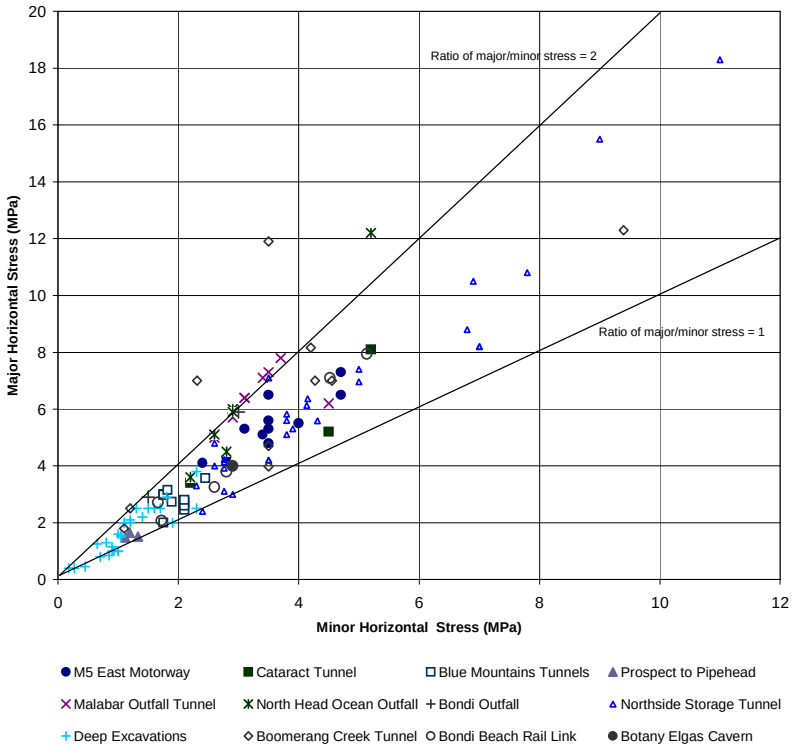

Figure 5: Major vs Minor Horizontal Stress.

Further conclusions can be deduced from the data subset in Figures 2 to 5, which relate to local trends at project sites, as follows:

- The magnitude of the major horizontal stress (σ_H) does not show a discernible trend with depth, i.e. individual σ_H project results tend to be relatively constant over this depth range (data below 20 m). Thus, higher ratios of σ_H/σ_V will occur at shallower depths as the vertical stress (σ_V) reduces with lower overburden depths, and the horizontal stress remaining relatively constant. Thus the horizontal stress remains "locked in" as vertical stress is reduced by erosion.
- The minor principal stress is generally σ_V, with the major and intermediate principal stress being horizontal.
- The "locked in" horizontal stress is demonstrated within the three formations, Hawkesbury Sandstone, Narrabeen Group and Bringelly Shale.
- Topographical influences can result in stress concentration and/or local reorientation of the horizontal stress field (Figure 2). Reorientation can occur for a relatively small feature, such as the 20 m deep infilled palaeodrainage channel which occurs at the eastern end of the M5 East tunnel (see Case Study). Concentration effects are illustrated by the elevated σ_H of 12.2 MPa, obtained by hydraulic fracturing near the intersection of the 70 m high vertical headland and the seabed at the North Head Outfall (see Case Study).
- The reasons for higher stress magnitudes, and/or a maximum horizontal stress orientation that is not consistent with the regional direction, are not always apparent and explainable by reference to topographic influences. For example, the reason for the high magnitudes of σ_H for the Northside Storage Tunnel of 10.5 to 18.3 MPa at 75 to 90 m depth were not immediately obvious (Strata-Tek, 1998). The horizontal stress field ratio was within typical limits (Figure 5), thus the cause of the higher stress may not have been directionally controlled.
- In general, the maximum horizontal stress direction reflects the regional structural geology (Figure 2).

Data sources for the data in Figures 2 to 5 include Brown and Windsor (1990), John Braybrooke (Douglas Partners, pers comm), Enever et al., (1984), Enever (1999), Strata-Tek (1999), Strata Control Technology (1998), Thomas et al. (1991), Van Putten & McQueen (1993), Strata-Tek (1998).

6 CASE STUDIES

Case studies demonstrate the effect of the *in situ* stress field on civil engineering excavations. These are intended to assist the reader in interpreting the *in situ* stress measurements. McQueen (2000) provides more details of some of these projects.

6.1 CATARACT TUNNEL

Cataract Tunnel was the subject of recent studies in relation to subsidence effects due to proposed coal mining below the tunnel (Gale et al., 1998 and Golder Associates, 1997). The 3 km long, 3.5 m span, water supply tunnel near Appin (Figure 2), was constructed by drill and blast in the 1880's, entirely through Hawkesbury Sandstone at up to 70 m overburden and no long term rock support or lining.

This case study demonstrates the behaviour of an unsupported tunnel in Hawkesbury Sandstone under conditions of relatively low overburden and high horizontal *in situ* stress. Stress induced effects were the dominant cause of rock falls, which probably occurred during construction; but also as progressive, time dependent failure. This type of failure is indicated by an inverted 'V' shaped fracture in the sandstone roof due to induced tangential compressive stresses. Figure 6 illustrates a more dramatic example of this failure. The time-dependency of stress failures is also shown from rockfall debris and fracture surfaces, indicating that failure occurred in a number of locations much later than construction.

Stress fractures concentrated in the area of highest overburden, however small stress fractures were also noted in areas of low overburden (20 m). The location of the failures indicates that anisotropic geological conditions, in combination with a high horizontal stress field, are required for stress failure. The stress fractures indicate the maximum horizontal stress to be near normal to the tunnel (ENE - WNW), which is consistent with coal mine orientation data 500 m below the tunnel. On this basis one of the overcoring measurements, which gave a maximum orientation of NW-SE, was assumed to be influenced by local geological features (Figure 2).

A model for the stress failure mechanism, using the example of this tunnel, is given in the next section.

Figure 6: Cataract Tunnel. Inverted V-shaped roof failure surface in massive sandstone below siltstone. Rockbolts were installed for tunnel support prior to longwall coal extraction below the tunnel.

6.2 M5 EAST MOTORWAY TUNNEL

The M5 East tunnel in Sydney is Australia's longest road tunnel at 4 km in length and driven through Hawkesbury Sandstone (Figure 2). Stress measurements by hydraulic fracturing were obtained from three boreholes (Strata-Tek 1999). The results illustrate the influence of buried topography (20 m deep infilled palaeochannel) on the *in situ* stress field. Measurements below this feature indicated a reorientation of σ_H to normal to the palaeochannel.

Measurements at the areas of highest overburden (about 65 metres) were generally consistent with the regional major horizontal principal stress orientation and magnitude (Figures 2 and 3). Stress induced problems did not have a significant influence during construction (Gibbs et al., 2002), however the effects of stress fracturing in thin sandstone beams were observed in the tunnel roof .

6.3 NORTHSIDE STORAGE TUNNEL

The main TBM-bored tunnel for this project is 16 km long and 6 m diameter, between North Head and Lane Cove. Constructed between 1998 and 2000, it passes below deeply incised palaeochannels at Manly, Middle Harbour (Clontarf), Cammeray and the Lane Cove River (Parker et al., 1999). These palaeochannels extend down to about RL -66 m. A hydraulic fracturing stress measurement programme included tests below these features (Strata-Tek, 1998). Below the Manly and Cammeray palaeochannels the σ_H/σ_V ratio was 6 to 9 (Parker, 2000). Below the Middle Harbour palaeochannel, Parker et al. (1999) describe 37 m of pre-existing low angle fractured zones and subvertical joints associated with a lower stiffness 19 m thick laminite layer within the sandstone. This deformation was probably the result of release of the concentrated stresses below the palaeochannel, resulting in a highly fractured and permeable rock mass. The release of stress due to deformation was reflected in lower stress measurements at Middle Harbour, with σ_H up to 1.5 times σ_V.

The occurrence of high magnitudes of σ_H were indicated in Section 5. Stress-induced fracturing was encountered in the tunnel.

6.4 BLUE MOUNTAINS SEWERAGE TUNNELS

About 38 km of tunnels were constructed within Hawkesbury Sandstone, Burralow Formation, Wentworth Falls Claystone and Banks Wall Sandstone between Winmalee and North Katoomba (Figure 2, Van Putten and McQueen, 1993). This included about 25 km of tunnels constructed by 3.4 m diameter TBMs between Faulconbridge and North Katoomba. *In situ* stress measurements by hydraulic fracturing were obtained between Lawson and North Katoomba (Strata-Tek, 1991) where the major influence on the stress field was the 200 m high escarpment south of Katoomba (Figure 2). The sandstone proved to be a difficult rock to fracture hydraulically, that is to induce a vertical fracture in a borehole test section. The unsuccessful tests were due either to initiation of horizontal fractures or to no fracture initiation from a lack of pressure build up in the porous rocks.

Stress measurements indicated the topographic effect from the escarpment with the major horizontal principal stress realigned parallel to the east-west escarpment. Near the central part of the tunnels, the major horizontal stress and topography were related with shallower measurements tending to align with major drainage directions. However, the deeper measurements reflected a regional trend of approximately 050 to 070 (Strata-Tek, 1991).

The major influence of stress on tunnelling was due to low stresses. In narrow ridges, stress relief due to erosion of adjacent valleys resulted in open rock mass conditions to depths of 90 m, leading to a permeable rock mass and deep penetration of weathering effects. The effect on tunnelling included a 180 m section where increased overbreak, increased support needs, poor gripper support and floor conditions for the TBM, were experienced. Also high groundwater inflow occurred where the tunnel passed below a small valley floor. The inflow was from a single, sub-horizontal bedding plane, which was open due to stress relief from valley formation. No problems related to high induced stresses were reported (Van Putten and McQueen, 1993; Bleuler and Bennet, 1996).

6.5 MALABAR OCEAN OUTFALL SEWERAGE TUNNEL

Three ocean outfall sewerage tunnels were constructed at Malabar, Bondi and North Head between 1985 and 1990. Malabar and North Head outfalls comprised a 5 m span roadheader and drill and blast decline tunnel and 4 m diameter offshore TBM drive about 4 km long. The Malabar decline tunnel traversed the Hawkesbury Sandstone and Newport Formation (Lowe and McQueen, 1990a) and demonstrated stress fracturing and deformation due to the stress concentration effects of a high horizontal stress field within anisotropic geological conditions.

Time-dependent stress failure occurred in the Hawkesbury Sandstone in the decline tunnel at about 100 m depth, illustrated in Figure 7. This occurred more than one year after excavation, prior to completion of the offshore TBM drive. The fallout occurred in a sandstone bed isolated below a siltstone parting less than 0.3 m above the roof.

In situ stress and stress-induced effects were examined by overcoring in the interbedded sandstone and siltstone of the Newport Formation. This was carried out in the bottom station (base of the decline tunnel) to determine stress impacts for the TBM offshore drive. Higher stresses were measured in the sandstones compared with the more jointed interbedded siltstone layers (Enever et al., 1988, Enever et al., 1990). This was attributed to the preferential distribution of stress from the low modulus siltstones to the stiffer sandstones. The data indicated that, following stress relief due to excavation, shear slip of beds and compressive failure could occur where thin incompetent layers were intersected in the tunnel roof and invert (Enever et al., 1990; Brown and Windsor, 1990). Figure 8 illustrates stress fracturing in the bottom station in a thin sandstone bed within interbedded siltstone.

Figure 7: Malabar Outfall, decline tunnel roof. Time-dependent stress failure in sandstone below siltstone bedding (lower part of the photo). Additional rockbolts were installed after the fallout, about one year after excavation.

The roadheader-excavated surface of the decline and bottom station did not reveal the effects of preferential stress concentration to the same extent as the smooth walled TBM tunnel. Stress movement along bedding was easily observed in the TBM tunnel due to the full face, one-pass cutting action of the TBM, compared with the partial-face and trimming cutting action of the roadheader. This effect does not appear to be peculiar to the Malabar situation and was alluded to by Gray (1982).

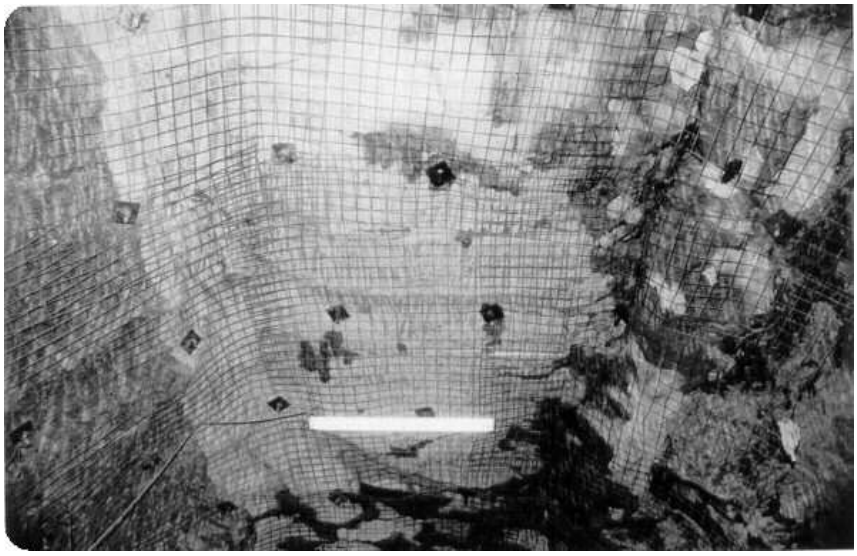

Figure 8: Malabar Outfall, bottom station roof in Newport Formation. Stress induced "guttering" in a thin sandstone layer within interbedded sandstone and siltstone (right hand side of the photo).

The stress effects in the TBM tunnel included movement along bedding in the roof and invert as well as stress fracturing through massive sandstone beds and stress-assisted fallout of fractured siltstone from the roof. As a result of these failures, additional support was required, especially in the shoulder area of the circular tunnel. Inwards movement of beds also caused difficulties for the installation of pre-cast concrete liners due to a decrease in clearance between the pre-cast lining and tunnel walls. Stress cracking in the roof, and movement along bedding partings, also occurred in the Bald Hill Claystone and Bulgo Sandstone formations which were intersected further offshore where the overburden was about 60 m.

6.6 NORTH HEAD OCEAN OUTFALL SEWERAGE TUNNEL

Hydraulic fracture measurements demonstrate the local stress concentration effects of topography on *in situ* stress for the North Head decline tunnel, which passes within 40 m of the base of the North Head headland. The decline is within the Hawkesbury Sandstone and Newport Formation at this location (Lowe and McQueen, 1990b).

Elevated horizontal stresses were encountered below the abrupt topographical change from vertical cliff to sub-horizontal seabed. The concentration effects of the deflection of horizontal stress below this area resulted in a σ_H of 12.2 MPa (see Section 5). σ_h was not as affected, giving a σ_H/σ_h ratio greater than the typical upper limit of 2. The stress concentration effects were confined to less than 40 m from the seabed intersection as no stress related problems were encountered in the decline tunnel, which was 40 m from the cliff/seabed intersection. Convergence measurements in the Hawkesbury Sandstone appeared to indicate some time dependency, taking more than 6 months to become constant, although total wall displacement was only about 2 mm (Lowe and McQueen, 1990b).

Low stress conditions also affected tunnelling. The 70 m high headland has been isolated by erosion to the north, east and south, allowing relief of horizontal stress from loss of lateral confinement. The result was open jointing, allowing greater depth penetration of weathering, increased fallout of blocks from tunnel walls and higher tunnel groundwater inflows.

Fracturing due to high induced stress was relatively rare in the offshore TBM drive at North Head compared with Malabar (reflected in stress measurements in Figure 3).

6.7 PIPEHEAD TUNNELS

The Pipehead to Potts Hill Tunnel ('Potts Hill tunnel') and Prospect to Pipehead Tunnel ('Pipehead tunnel') are 4 m diameter TBM-bored water tunnels 7.6 km and 7 km long, respectively (Figure 2), driven in Bringelly Shale with a maximum cover depth of about 55 m. The Potts Hill tunnel was constructed between 1971 and 1974 and the Prospect tunnel between 1992 and 1993 (Lowe, 1993).

Cracking occurred in the roof and shoulders of the Potts Hill tunnel (Gray, 1982). An inverted V-shaped fracture, reported in the roof some months after excavation (Braybrooke, 1985), is consistent with high excavation-induced stresses, and it also appears to be time-dependent. The Bringelly Shale is mainly laminite with interbedded sandstone, i.e. interbedded rocks of various stiffness. These anisotropic conditions were probably a factor in contributing to elevated stresses. Steel sets were the main temporary support system.

No stress-related problems were reported in the Pipehead tunnel (Lowe, 1993). Rock bolts and channels were the main temporary support system. Stress measurements by hydraulic fracturing indicated that the major horizontal stress was relatively low and consistent with depth (Figure 3). This suggests a low probability of stress-induced failure by fracturing through the rock substance, consequently adversely orientated rock defects would be required to assist stress related failure.

The reason for the absence of stress failures in the Pipehead tunnel compared with Potts Hill tunnel, given similar geology, excavation method and tunnel size, is not clear. Different support may have contributed to this; rock bolts provided reinforcement to the rock mass in the Pipehead tunnel, compared with steel set support in the Potts Hill tunnel which allows greater rock deformation before taking load.

6.8 ELGAS CAVERN, PORT BOTANY

This LPG storage facility was constructed between 1996 and 2000 and comprises 4 parallel caverns, 230 m long, 14 m wide and 11 m wide, at 124 m depth. Openings also include connection galleries and a water curtain gallery. The long axis of the caverns is orientated at 335°.

Two stress-induced failures from the roof of the cavern have been described by De Ambrosis and Kotze (2004). The failures occurred in 1997, the first being 13 m long x 7 m wide and the second 7 m long x ~5 m wide and up to 1.2 m thick. The initiation of both failures was attributed to compression induced movement along bedding planes due to high lateral stresses. The available *in situ* stress measurements were two overcored results from the caverns (John Braybrooke, Douglas Partners, pers. comm.), giving a σ_H of 4 MPa and σ_h of 2.9 MPa at 110 m depth. The orientation of σ_H was 014°, which is consistent with the regional trend.

6.9 CASE STUDIES - DEEP EXCAVATIONS

The horizontal *in situ* stress field in Sydney has influenced the construction of deep excavations. Removal of lateral restraint has resulted in lateral movement of the sides of excavations in the range of 0.5 mm to 2.5 mm per metre depth

with 5 mm or more occurring at distances greater than twice the excavated depth from the excavation (Pells, 1990). Movement occurs along a discontinuity in the rock mass such as bedding lined with low strength shale or weathered sandstone. Detailed studies of deep excavations in the Sydney CBD have been undertaken, particularly where excavations are adjacent to the city rail tunnels (Baxter and Nye, 1990; Braybrooke, 1990; Pells, 1990; Enever et al., 1990; Hewitt et al., 1999). The effects of the change in the horizontal stress field due to excavation, include:

- Cracking in the linings of railway tunnels behind the face of deep excavations, for example, the tunnel through the ANA site adjacent to the D2 excavation (Enever et al., 1990; Baxter and Nye, 1990) and the tunnels behind the World Square excavation (Braybrooke, 1990).
- Cracks in basements of adjacent buildings. Concrete cracking was reported in a deep basement due to the redistribution of stress after excavation of an adjacent deep excavation (Pells, 1990) and also in a basement adjacent to the Regent Place excavation (Hewitt et al., 1999).
- Lateral movement of the excavation face, which occurs in a stepped form along low strength continuous bedding planes and is accompanied by dilation of the overlying jointed rock mass. Up to 45 mm of lateral movement was recorded at the surface of the 26 m deep World Square excavation and 5 mm in the rail tunnel at about 11 m depth, and 27 m from the excavation face.
- Movements just below the base of the excavation. A borehole inclinometer recorded movements below the base of the partially completed World Square excavation (Braybrooke, 1990). This was attributed to compression of stressed sandstone beds beneath the floor (Pells, 1990).
- Vertical fracturing in thin sandstone beds in the excavation floor overlying a shale layer, attributed to the concentration of high horizontal stresses in the sandstone (Pells, 1990).
- Development of “drummy” conditions in excavations for pad footings, particularly in thinly bedded rock (Braybrooke, 1990).

The recent KENS Project in the Sydney CBD, experienced the effects of stress concentration in the floor of the deep excavation and the subsequent stress release. This was displayed in the lateral movement of sandstone blocks, where shale was less than 500 mm deep below the floor of the excavation, and in stress relief along joints in shale exposed in the floor (G. K. Scholey, Golder Associates, pers. comm.).

The 30 m deep excavation at the Regent Place in the CBD (formerly the Genting Centre) demonstrated the effects of the release of horizontal stress (Hewitt et al., 1999). This excavation in Ashfield Shale, Mittagong Formation and Hawkesbury Sandstone, is 3 m to 5 m from CityRail tunnels along George Street face. Inclinometers behind this face indicated 6 mm lateral movement into the excavation along bedding partings lined with low strength siltstone. Up to 8 mm displacement was recorded in the closest rail tunnel, however no cracking was observed in the liner. Surface movements were up to 15 mm at the top, which is about 0.5 mm per metre depth of excavation.

7 STRESS FAILURE MECHANISM

As demonstrated in the case studies, observed stress fracturing in tunnels typically occurs where anisotropic geological conditions exist in the roof. Induced stresses, higher than would be predicted from an isotropic case, can occur in these types of conditions given that other contributory factors such as the shape of the excavation are the same. The anisotropic conditions include sandstone interbedded with lower stiffness siltstone. In this situation, Enever et al., 1990 showed through stress measurements at the Malabar outfall tunnel that there was a tendency for higher stresses to be localised in stiffer beds. Anisotropic conditions also include the presence of a single low shear strength bedding feature just above a tunnel roof in relatively massive sandstone. The induced stress concentration effects of this situation were demonstrated by Pells and Best, 1991 and Pells, 2002. On the other side of the stress/strength ratio, a lower in situ rock mass strength may also be a significant contributor to stress failures, particularly in rock that may be prone to tensile fracturing, rather than recourse to elevated stress levels alone.

The propagation of fractures in brittle rock under low confinement, and within a tangential compressive stress field condition in tunnel walls, can be explained by a bi-linear failure envelope (Kaiser et al., 2000). In conditions of low confining pressure, tensile failure is likely to be dominant rather than shear failure, as demonstrated by Kaiser et al. (2000). This model is briefly examined for application to Sydney stress failures, which appear to have brittle behaviour.

The lower limit of the bi-linear failure envelope at which damage to the rock occurs, is represented by the ‘damage threshold’. This is the line represented by the Hoek and Brown equation with $m = 0$ and $s = 0.11$ (lower limit at which acoustic emissions occur). It was observed that visible damage or observed yield is encountered in massive, brittle rocks, when s reaches about 0.25. These limits correspond approximately to 0.3 to 0.5 times the unconfined compressive strength of the intact rock. When this threshold is exceeded shear stress failure can occur if confinement is

sufficiently high. However, at low confinement tensile spalling occurs from fractures propagating parallel to the maximum principal stress, which at a tunnel boundary is parallel to that boundary. Above the damage threshold at low confinement, tension zones were shown by Kaiser et al. (2000) to be generated in heterogeneous rocks (at the micro-scale) in a compressive field, where cohesion breaks down before frictional strength can be mobilised. Failure occurs predominantly by tensile failure (spalling) before the stress level reaches the rock mass strength envelope.

Failure may not occur instantaneously when the damage threshold is exceeded. Microcracks may continue to propagate due to such effects as moisture infiltration and/or other physiochemical effects. The resulting failure may not occur until much later after excavation. As indicated later in the case studies, some stress failures in Sydney have been observed to be time dependent.

The above failure model is demonstrated by a Sydney example from Cataract Tunnel in the Hawkesbury Sandstone (see case study). Figure 9 illustrates this case with the damage threshold and laboratory failure envelope for 15 triaxial tests for a range of sandstone rock types from the tunnel, and for the site of the *in situ* stress measurement at 48 m depth.

The induced stress in the roof and floor, calculated from the Kirsch solution, or from the maximum tangential stress (σ_θ) equation $\sigma_\theta = 3\sigma_H - \sigma_V$ for a circular opening, is 14.5 MPa, which is at about the level of the damage threshold. If an empirical stress concentration factor of 4 was used in this equation to represent anisotropic conditions (instead of the isotropic factor of 3), then the induced stress would be about 20 MPa (Pells, 2002 uses a factor of 6 for a quick assessment of potential stress failure in a circular tunnel). A stress path at the tunnel boundary may proceed from the *in situ* stress point to the region of low confinement following excavation, to the induced tangential stress of 20 MPa (Figure 9). This point is above the damage threshold and within the region where spalling can occur. The spalling region is between the damage threshold and the failure envelope and to a $\sigma_1/\ \sigma_3$ ratio of 20 (intact rock) or 10 (rock mass).

To make an estimate of the rock mass strength failure envelope, the Hoek/Brown failure criteria is applied for an intact unconfined compressive strength of 35 MPa (average for medium to coarse sandstone), $m_i = 13$ (from triaxial tests), and assuming a GSI = 90 (for the more massive conditions on the scale of stress failures, see Figure 6). The resulting failure envelope gives an unconfined rock mass strength of about 20 MPa. Above that strength, stress failure can be expected in the low confinement conditions of a tunnel boundary. Based on the approach of reduction of laboratory strength due to size effects, Pells (2002) suggests a reduction factor of about 0.6, which gives a similar rock mass strength. This model predicts spalling at the tunnel boundary with stress conditions above the damage threshold, and with a rock mass strength at or close to the calculated induced stress for anisotropic conditions.

For the time dependent failures observed, the stress level at the tunnel boundary may be above the damage threshold, where microcracking begins to propagate, but less than the rock mass failure strength envelope, where immediate failure would be expected.

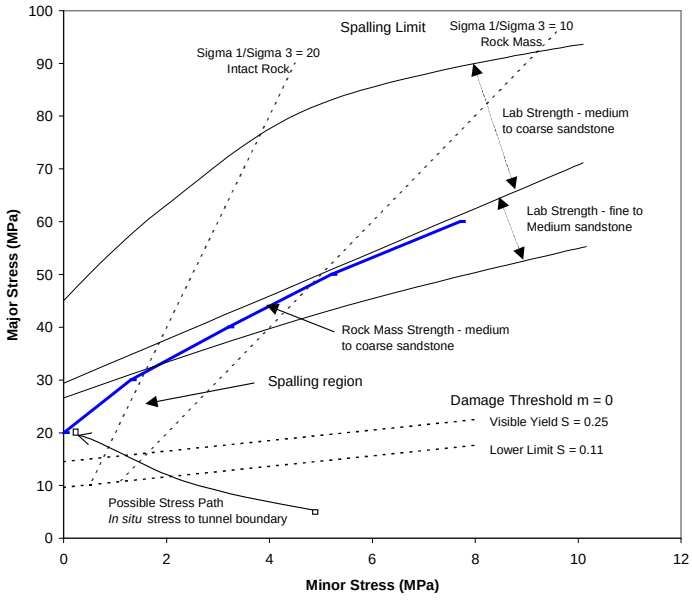

Figure 9: Bilinear Failure Model for Cataract Tunnel (after Kaiser et al., 2000).

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