M5 EAST TUNNELS: A FLAT ROOFED, BOLT AND SHOTCRETE-LINED HIGHWAY

Daniel N. Adams
Jacobs Associates

Manfred K. Lechner
Baulderstone Hornibrook/Bilfinger + Berger JV

Isabelle Lamb
Hyder Consulting

ABSTRACT

Sydney Australia's M5 East Motorway connects southwestern suburbs to the airport. Central to the project are twin 3.8 km roadway tunnels which are mostly driven through rock. In order to come up with the winning tender, the project team developed several unique approaches for tunnel excavation and support. The project's flat roof with thin shotcrete lining, 20 m wide underground intersections, thixotropic-grout sheathed rockbolts and cablebolts, and new shotcrete testing methods all differ significantly from traditional concepts. This paper presents the design basis and construction methods used to achieve the 100-year design life required by the project's owner.

INTRODUCTION

The M5 East will be a four-lane divided carriageway connecting to the existing M5 motorway in Sydney's southwest to General Holmes Drive adjacent to Sydney International Airport in the East. In August of 1998, the Roads and Traffic Authority of New South Wales (RTA) awarded the design, construction, and 10 year operation and maintenance contract to the joint venture of Bauderstone Hornibrook Bilfinger + Berger (BHBB JV). BHBB JV subsequently awarded a subcontract to provide design of the project to Hyder Consulting (Hyder). Hyder retained Jacobs Associates as sub consultants to provide tunnel excavation and support design. Principal elements of the project include 4 km of open dual lane carriageway, 3.8 km twin dual lane tunnels, a 700 m long cut and cover river crossing and two viaduct structures. The focus of this paper is the design and construction of the twin rock tunnels.

The Main tunnel consists of twin drives of between 49 m^2 and 56 m^2 area. Pedestrian cross passages were required at 120 m intervals, and one vehicular cross passage located at near mid-point of the tunnels is also required. There are three entry/exit ramps running parallel and connecting with the tunnels at the eastern end.

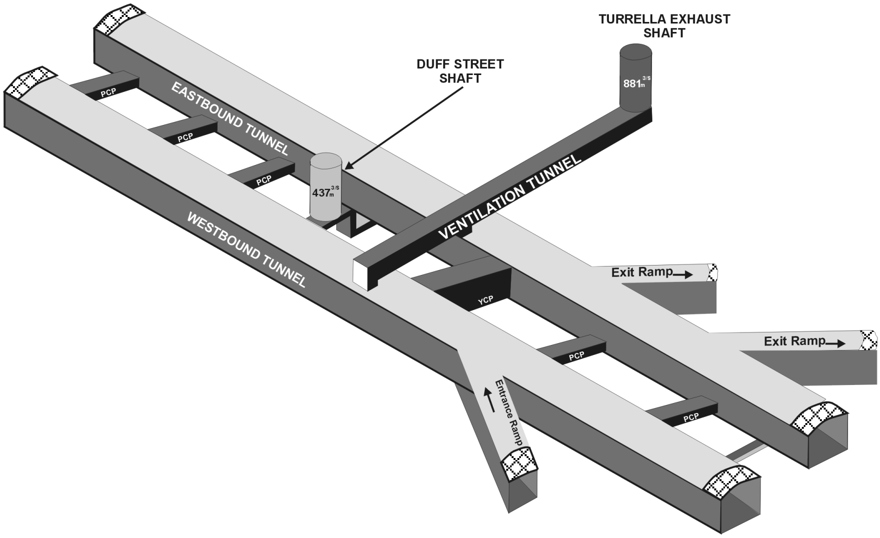

Figure 1. M5 tunnel layout

One 400 m long exit ramp connects at the Princes Highway, and 100 and 200 m long entry and exit ramps, respectively, at Marsh Street. At these ramps, the tunnel cross section increases to 80 m^2. In addition to the tunnels associated with autos, the ventilation system required that fresh air be supplied, and exhaust removed via an intake/exhaust tunnel and shafts. The intake shaft is 66 m deep, and the exhaust shaft, located at the other end of a 700 m long exhaust tunnel is 43 m deep. A simplified view of the tunnel and its ancillary structures is shown on Figure 1.

GEOLOGY

The M5 East tunnels are being driven for the most part through Hawkesbury Sandstone. The Hawkesbury, a sedimentary rock of the Triassic Sydney Sedimentary basin, is overlain by Quaternary alluvial deposits consisting of sands and clays deposited during the Pleistocene period prior to the last sea level rise. It consists of massive, laminated and cross bedded fine to coarse grained quartz sandstone in near horizontal layers. Bedding thickness is generally between 1 to 3 m, however, cross beds are typically thinner than 1 m. Within the sandstone beds, shale, or siltstone fragments can occur, forming thin layers or beds of shale breccia. Locally shale breccias may occur as infill within erosional depressions or channel features and may be up to 4m deep and greater than 10m wide. Mudstone facies, or laminites, also occur in the Hawkesbury Sandstone and comprise laminated or finely interbedded mudstone, siltstone and fine sandstone and are typically 0.3 to 3m thick. Slaking can occur within the laminite on exposure to wetting and drying effects. Iron cementation is common in the upper weathered areas and can occur as very high strength thin bands, which have been referred to as ironstone.

The M5 tunnels have been driven trough the eastern end of the Fairfield Basin, which is oriented roughly east-west. During excavation, bedding was observed to be gently undulating and dipping to the northwest, north and northeast. Cross bedding dips up to 30°, to the northeast, north, and southwest. The alignment of the tunnels is about normal to the strike of the bedding.

The predominant rock mass defect is bedding. Bedding partings are commonly lined with extremely weathered rock or even clay veneer, in the upper weathered rocks to clean below this. However, some decomposed particles do occur throughout the whole depth of the formations, ranging in thickness from a few millimeters up to 20 mm. Low strength bedding partings also occur along thin silty carbonaceous and/or micaceous laminations or beds of generally less than 30 mm thick. Jointing observed during excavation is typically poorly developed and when present occurs in local zones of typically parallel striking, steeply dipping joint sets spaced less than 1 m apart. Near surface joints are weathered, with iron oxide coatings and clayey infills of up to 10 mm and may be open. Joint spacing in the shallow excavations ranges from 1 to 8 m. At depth the joints are tight, planar, typically rough, greater than 8.6 m long and are generally not weathered.

Faulting through the sandstone in the M5 tunnels appears generally as either low angle shearing moving along and between bedding and cross bedding partings or steep normal faulting along individual polished surfaces or multiple surfaces in sheared zones of up to 6 m in width. Offset on the faults is generally less than 1 m, however, at one location over 6 m of offset was apparent. The steep faults contain clayey crush ranging from a few mm in thickness up to about 250 mm and sheared rock zones of up to 1.5 m in width. The main fault zones encountered strike generally sub-perpendicular to the tunnel alignment and are parallel to the local jointing orientations. The exception to this is one main fault zone approximately 8 m wide comprising parallel polished surfaces and splays with up to 250 mm of clayey crush and which strikes subparallel to the tunnel direction. A dyke swarm was encountered in the M5 tunnels and comprised five individual dykes ranging in width from 0.2 to 1.2 m. The dykes are typically deeply weathered with consistencies ranging from stiff clay to high strength rock. Within this swarm the individual dykes dip steeply toward either the east southeast or west northwest and are coincidental with local faulting. Elsewhere in the Hawkesbury Sandstone weathered dykes of up to 16 m in width have been encountered.

Another feature in the Hawkesbury that may locally disrupt the horizontal continuity of bedding, occurs where zones of intra-formational slumping or turbulent environment deposits occur. These deposits appear to form due to the disturbance of thick mudstone or siltstone beds, lenses or channel depression infill materials. This disturbance results in the development of a rock mass comprising large low strength discontinuous mudstone or siltstone bodies with irregular internal polished and slickensided surfaces in a variable strength sandstone matrix. Such conditions were encountered in the M5 tunnels as deposits of up to 3 m thick.

Within Hawkesbury Sandstone the horizontal in situ stress is typically much higher than the vertical overburden stress. Measurement of the in situ stress field along the route of the M5 tunnels gave values for the major horizontal principal stress ranging from about 4 to 7.5 MPa for depths of up to about 65 m. Two main orientations were measured during the testing, one toward the north-northeast, which is consistent with the regional major horizontal principal stress direction, and one toward the east-southeast. The latter orientation is considered to result from local paleochannel development and runs parallel to the tunnel alignment.

The Hawkesbury typically has very low rock mass permeability, and groundwater flow through it is dominated by flow along rock defects, generally occurring along near horizontal bedding partings and the vertical joints which connect them. Higher tunnel water inflows do occur along geological structures, dykes, sheared zones, etc. or along bedding partings which may have opened up due to unloading or stress relief as a result of valley bulging effects, particularly within 10 to 20 m of valley floors.

Unconfined compressive strength of the rock ranges from less than 5 MPa where it is extremely weathered to greater than 60 MPa at depth. Generally, the sandstone ranged in strength from 15 to 50 MPa, with an average strength of 30 MPa. The interbedded sandstone and siltstone, and siltstone ranged in rock strength between 10 to 25 MPa, with an average of 20 MPa.

DESIGN

Because of the structure, strength and in-situ stress conditions, it was determined at tender that the traditional arch shaped tunnel was not the most effective design. Forcing a round shape into a stressed, horizontally bedded rock would lead to unstable "corners," as well as be less efficient with respect to minimum cross sectional area. During the preliminary design, the team used the principles presented in the SME Mining Engineering Handbook, to both evaluate, and then educate other team members that a flat roof would provide the most stable opening in most cases, allowing for minimum support. In addition to these structural issues, using a flat roof was very efficient – it minimized excavation requirements outside of the roadway working widths and signage clearances.

As part of the design effort, the team made several site visits to tunnels currently under construction within the Hawkesbury. These visits generally confirmed our preliminary assessment of the rock, and gave the team confidence with the shotcrete and rockbolt design. The site visits also lead the team to develop a single pass operation, where the permanent lining is installed as the excavation progressed.

Permanent Lining of Rockbolts & Shotcrete

Combining geotechnical assessment with practicality of tunnel construction, the team determined that three different ground classifications would best suit both inputs. Put simply, Type 1 consist of massive Sandstone with Siltstone laminations less then 5 mm thick, Type 2 consists of Interbedded Siltstone and Sandstone or Siltstone, and Type 3 consists of Dyke, Fault, or Crushed rock zones. The rock support for both Type 1 and Type 2 conditions was determined to rely on rock bolts for the majority of support. It was assessed that running or raveling would not be an issue. Bolts would hold the laminations together, and shotcrete could be used generally speaking to maintain the rock conditions – namely keep the rock mass from weathering. A cross sectional view of Type 1 & 2 rock support is shown in Figure 2. The rock support for Type 3 conditions consists of rockbolts and shotcrete, in a more traditional arched shape. This Type 3 support was anticipated to occur over 5% of project.

Rock Bolt Design. Design of the rock support system included use of empirical relationships by Sharp, Endersbee, and Mellors, 1984; Stacy and Page, 1986; and others to assess rock bolt lengths, as well as an in-depth review of case histories in and around Sydney. To determine rock bolt capacity requirements both suspension bolting and arch building analysis were utilized. Between these, it was determined 200 kN rock bolts were required to support all rock classes. The most challenging part of the rock bolt design was accommodating the RTA's 100 year design life constraints. There is a

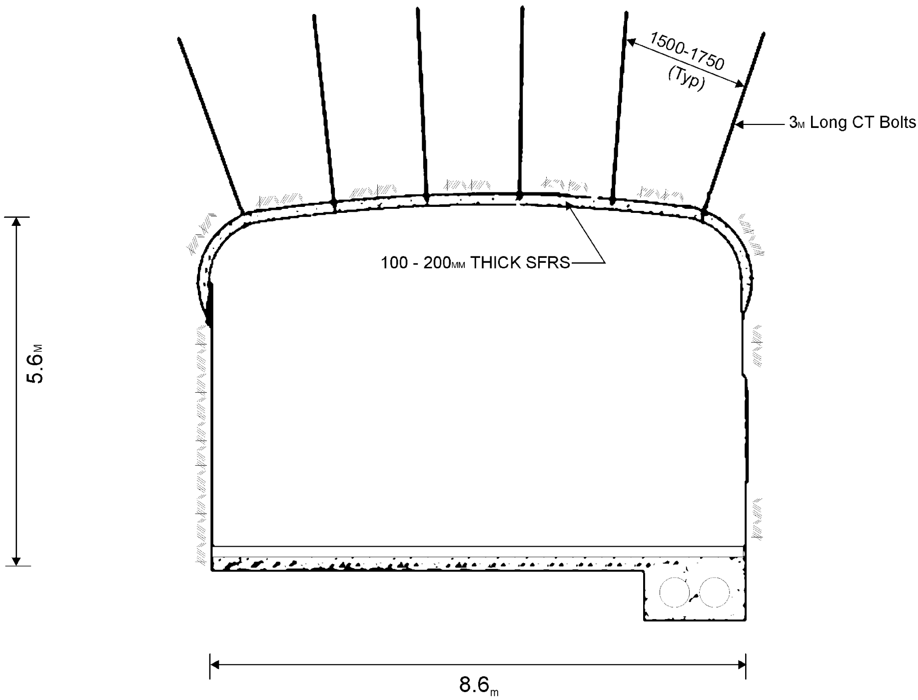

Figure 2. Typical tunnel support

considerable amount of concern and uncertainty associated with the long-term durability of rock bolts (Baxter 1996). It was determined that double-corrosion protected rockbolts would be required to meet the 100 year design life. The team reviewed other projects where galvanizing and epoxy coating seemed to be the most routine method of achieving this, however, the effectiveness of either as double protection is questionable, and the ability to install epoxy coated bolts efficiently without scratching and chipping, in a one-pass environment, is very difficult.

With these design inputs 3 m long CT Bolts, with 211 kN yield were specified. The CT bolt, as shown in Figure 3, consists of a steel bolt contained within a deformed plastic tube, and a helical "loading bulb" for grouting. For M5, mechanical anchors were added to provide active rock support upon installation. Once installed and tensioned to 50 kN, the bolts are then grouted, using thixotropic grout, injected through the load bulb, up the inner shaft, and down the outside of the bolt, providing true double protection. Once the grout cured, the bolts behaved like a pre-tensioned dowel. Throughout the design, Strata Control Systems, of Australia, assisted the team as the anchors and details were added to the bolt. They also supplied the CT to the project.

Shotcrete Design. Shotcrete played an important role in the support systems for the tunnels. For Type 1 and Type 2 conditions identifying the load requirements for the 100 yr design life was elusive. After several iterations, it was determined that in addition to protecting the rock from weathering, the shotcrete must be capable of carrying a "computer monitor" sized block of rock which could loosen along cross beds within any individual bed. For Type 3 Rock, the development was more straightforward.

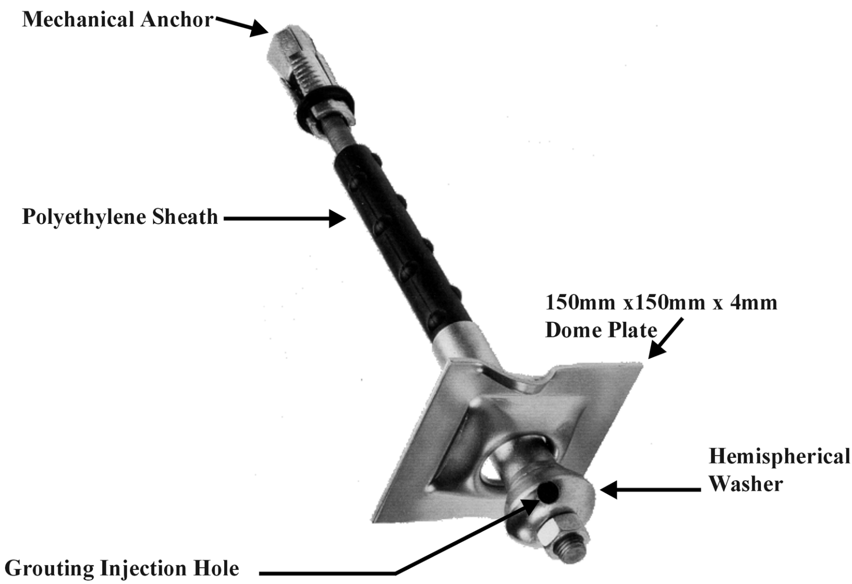

Figure 3. CT bolt

Shotcrete in this case was designed to carry rock that was assumed to loosen to gravel like consistency between rockbolts.

With the nearly flat roof, traditional "arching" was not a relevant resolution of load for Type 1 and 2 ground. Using principles similar to those presented by Barrett, S.V., and McCreath, D.R., 1995, it was determined that the dominant failure mechanisms for the shotcrete, given the design load were adhesive failure and flexural failure. With respect to adhesion, the design team used data from previous tests on the Hawkesbury, for design values for the bond strength between rock and shotcrete. To ensure that the construction team achieved the required bond, the design included specifications requiring the jet-washing of the rock surface, and inspection, prior to spraying. Where the required bond strength could not be achieved, rock dowels were placed subsequently through the shotcrete.

Assessing the flexural capacity of shotcrete was not straightforward, as the current state of practice is not clear. Some designers insist on referring to reinforced concrete code where others insist on a simplified classification system of shotcrete capacity. Existing literature ranges from guidelines such as "use 65 kg drawn wire fibres to handle flexural loading," to more specific guidelines which tie ASTM C1018 toughness indices and unconfined compressive strengths to a precise flexural capacity. Using the latter, in a variety of formats, enabled the team to quantify the flexural requirements of the shotcrete lining, and specify criteria to be met by the Contractor to achieve the required flexural capacity.

With the bolts and shotcrete defined in this fashion, the system was modeled using FLAC. Models were run for over 40 different variations of rock conditions. Overall, the results showed that the system provided adequate support for all conditions. Convergence was estimated at between 6 and 8 mm at the crown centerline and 13 mm at the side-walls. Rock bolt loads were anticipated to range from

16 to 90 Kn, with the latter only occurring in the worst Type 2 variation. Models using the Type 1 conditions showed that most of the bolts in a given array carried no load. These models also showed that the shotcrete lining carried no load.

Groundwater Control. As described above, groundwater was anticipated to initially flow through discrete bedding planes, and reduce to nil over time. The tunnel was designed to have the water drain away from the lining, by using a system of stripdrains placed between the rock surface and the shotcrete. By putting a very shallow arch in the roof, the drains, encapsulated in shotcrete, carry water to the walls, laterally. Once at the side-walls, they were connected into the carriageway drainage. Water played a large role in the durability assessment of the shotcrete. This added requirements to the shotcrete mix design, to minimize any leaching of cementitious material.

Wide Roof at Ramp Intersection. Perhaps the most interesting aspect of the design was the ramp intersections. These are the three places where the tunnel widens to allow for ramp bifurcations. Each of these intersection areas received a higher degree of both investigation and analysis, with boreholes at several locations within each one, and in-situ stress tests being performed. The design at the ramps proceeded as described above, leading to a system that included 6 m long bolts and 200 mm of shotcrete. However the FLAC modeling indicated that with 6 m bolts, the rock mass yielded in tension directly above the bolts across most of the crown. It also revealed that the opening was un-stable unless support was put in sequentially, and that there was significant inter-bed shear expected within the first 3 m of the bolts in each shoulder. In response to this, 7 m long units were modeled, which resolved the tensile failure. However, putting a 7 m rigid bolt in a 6 m high opening is impractical. So, a flexible cable bolt was adopted. The second problem was handled by requiring the excavation to proceed in three-stage fashion. The cross section was to be excavated with the left heading, support installed, and then a right heading, support installed, and finally the central drift removed and the remaining support installed. Several model iterations were used to assess how big and how long each cycle could be, and these requirements were matched with capabilities in the field.

With a cablebolt, shear capacity is far less than a rigid bolt. However, the model indicated that the high shear loads occurred in the shoulders, and not the central section of the tunnel. To resolve this problem, the cablebolts were inclined to be as near to horizontal across the shoulder area as possible, hence transferring the shearing along the cable, and not through it. This reduced shear across the cables to acceptable levels and lead to the support array shown in Figure 4. However, shifting the alignment resulted in an anticipated axial bolt load of up to 400 kN. With this load, a pre-established project criteria of double corrosion protection, and the Construction group insisting that they would not approve a two piece bar nor any post-tensioned two-stage-grouted cables, the team set out to design something new. Working with Megabolt Pty. Ltd., of Melbourne Australia, the team developed the 600 Kn cablebolt shown in Figure 5. The bolt, called the Megabolt, consists of a 9-strand anchor, wrapped in a corrugated polyethylene sheathing which is sealed at the bottom to the inside of a stainless steel cup and saucer bearing assembly. The bolt was fitted with large mechanical anchor which is set in the hole using the central strand. The setting strand allows for placement precisely in the hole, eliminating the risk of “setting early”. The stainless steel bearing assembly includes an isolating gasket, to separate the stainless portions from of the carbon steel strand seat. Grouting is performed using the same grout (thixotropic) as the CT bolt and the same philosophy – injection through a threaded hole in the cup portion of the bearing plate, and venting, down the outside of the sheathing though the saucer portion of the bearing plate.

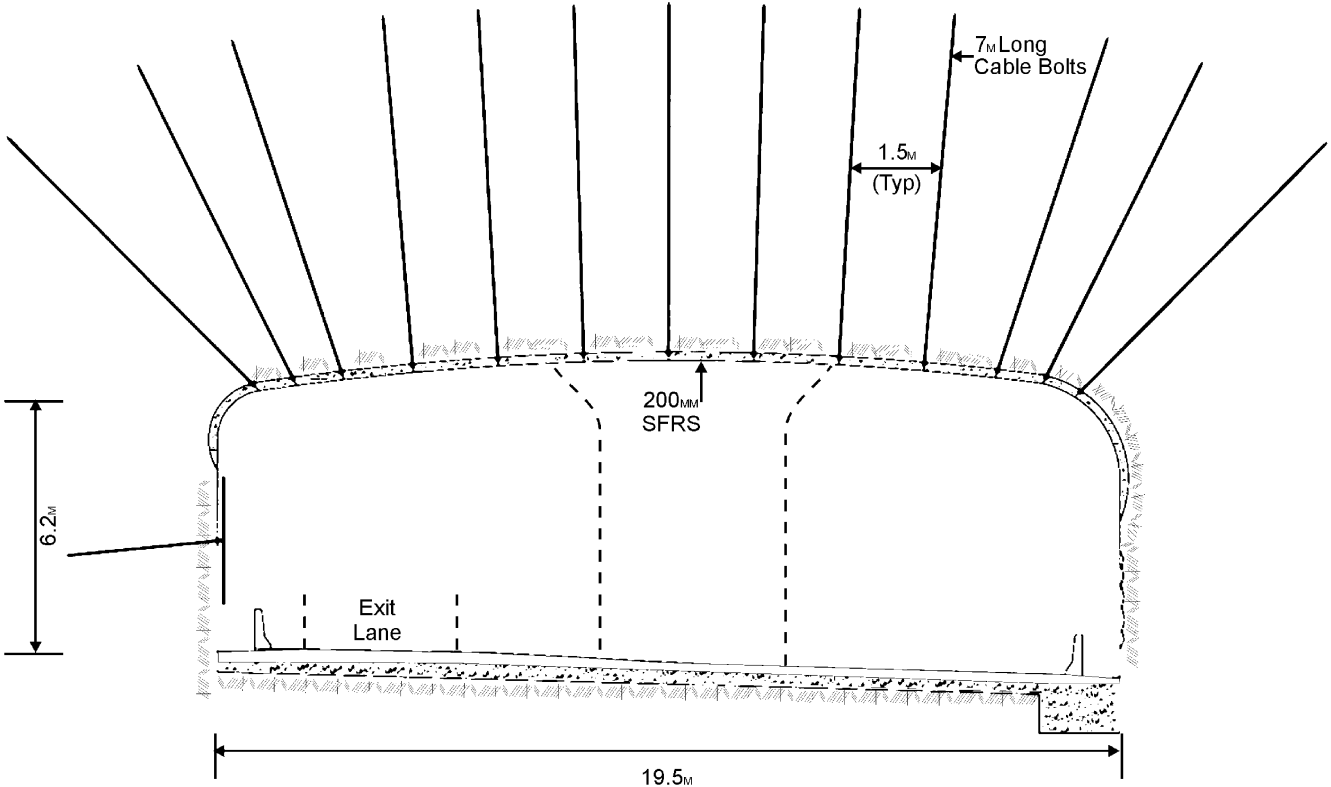

Figure 4. M5 ramp intersection support

Pre-Construction Testing & Quality Control

Rockbolt and Cablebolt Trials. Prior to the execution of the works, it was required to conduct an extensive field trial program to confirm the design assumptions and to gain certainty that the proposed support elements are meeting the requirements for a 100-year design life. For both the CT Bolt and the Megabolt, the critical protective element of the sheathing is a full encapsulation in the grout. The fact that thixotropic grout (water/cement ratio approx. 0.35) had to be used to meet the design requirements gave rise to concerns about the grout's ability to fill both the inner and outer annulus of the tubes – this type of grout is not commonly used with rockbolts. To demonstrate the performance of the grouting operation, one fully assembled CT-bolt, and two fully assembled Megabolts were placed vertically in transparent plastic tubes (one cablebolt was in steel pipe) with the same diameter as the borehole. Grout was injected through the grout bulb/injection ports of the bolts after they were "tensioned" in the testing frame.

The project team were present during these trials. The grout did flow up until the mechanical anchor was fully encapsulated and then returned on the outside of the sheathing until the full length of the bolt was encapsulated. For the one megabolt installed in the steel pipe, the pipe was cut into some 50 pieces, all of which showed a fully encapsulated strands and sheathing on both sides.

Upon acceptance of the grouting procedures, further field trials were required to confirm mechanical anchorage. Twelve CT bolts and three Megabolts were installed in in-situ conditions. All bolts were tensioned to 50 kN as required by the design. Then the test units were pulled to 100 kN with the displacement measured during application of the load. In all but one case, no significant displacement has been recorded – confirming design assumptions on the mechanical anchorage for both units. One Megabolt failed at the mechanical anchorage at about 80 kN, however, it was determined that this was due to the anchorage zone being within weathered rock. The units were not rejected, however, the failure was a catalyst for developing a plan for future failed anchorage of these units. Four of the CT bolts were fully grouted as part of the trials, and successfully pulled to failure at loads in excess of the required test load.

Steel Fiber Reinforced Shotcrete. Initially, a number of different mixes were trialed in order to fulfill two basic requirements: constructability (successful spraying and immediate adhering to the rock statum) and meeting the requirements for a 100 year design life, which included mix design criteria for durability, as well as a significant post crack performance criteria. Since these two features are often in conflict with each other, the range for a suitable mix was limited. After a suite of pre-production spraying and testing, a mix with a total cementitious content of 520 kg/m^3 was selected. The mix included 60 kg/m^3 fly ash and 40 kg/m^3 silica fume, 60 kg/m^3 of drawn wire fibres, and had a water/cementitious ratio of 0.38. It was also determined during these trials that the concrete would be delivered from a standard mixing plant as a wet-mix shotcrete with fibres added at the plant. The dosage of alkali-free accelerator was set at 4% of the total cementitious content, and the slump was 90 mm at the pump.

As indicated above, the shotcrete plays a vital role in achieving the 100 year design life of the project. Due to this, a rigorous testing regime was specified during construction, which included unconfined compressive strength and toughness testing for every 250 m^2 of schotcrete surface area, adhesion testing (bond) for every 10 m^2 placed, permeability testing, and shrinkage testing. Most of the tests procedures were based on generally recognized standards. However, based on the results of the testing and their repeatability, both toughness and adhesion testing required alternative testing methods. As described above, the toughness testing, as it relates to flexural

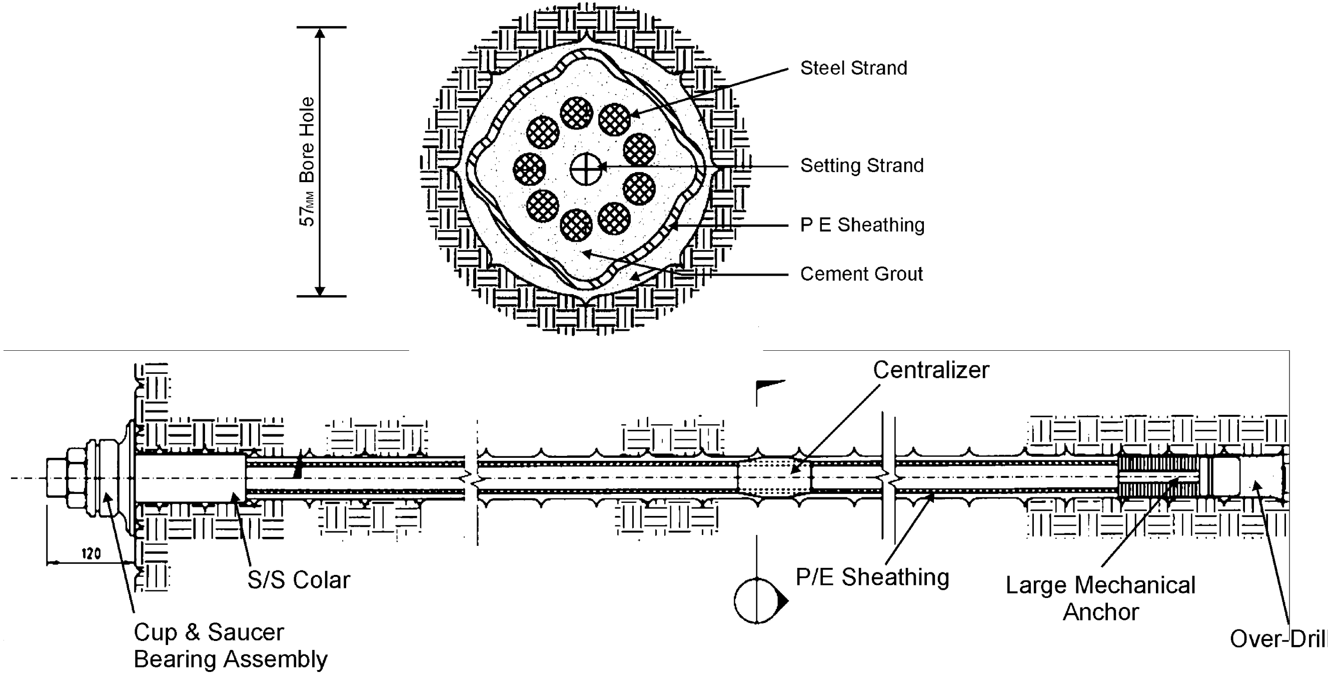

Figure 5. 600 kN Megabolt

capacity or post-crack performance of the shotcrete is a key feature for achieving the design intent. Various standards exist for testing the toughness using beams. For the M5 East, initially a “cross-breed” between ASTM C 1018 and the European Specification for Sprayed Concrete (EFNARC 1993) was selected as the testing method, however, difficulties in the statistical analysis of the results called for a revisiting of the testing. In cooperation with Dr. S. Bernard of the University of Western Sydney, shortly into the project, toughness testing was changed from beans to a method using Round Determinate panels. This testing has proven to be more reliable than the conventional beam testing and gave confidence that the selected shotcrete mix does fulfil the requirements of a 100 year design life. In a similar vein, the testing for adhesion initially involved coring and testing in accordance with EN 1542. However, since the conventional coring did not follow statistical principles as required for a quality controlled production testing, it was decided to change adhesion testing into the hammering method as proposed by the Norwegian Concrete Association. Here, the shotcrete is examined for any defects in adhering to the rock by the acoustic output of hammering against it. In order to maintain correlation to EN 1542, a significant number of cores were taken.

CONSTRUCTION

The excavation was mainly done using roadheaders. A total of six machines were in service on the project, 3 Mitsui S 300 machines, and 2 AM 105’s both of which have cutting power of 300 kW. The 6th machine was a 200 kW Mitsui S 200 which was mainly used for the single lane ramp tunnels. Excavation rates ranged from 4 m/day to 8 m/day depending upon the ground conditions encountered. Excavation of the $10m^2$ pedestrian cross passages between the twin tunnels was done by drill & blast method. Rockbolts were installed using Tamrock robolters which reduced the time for drilling, placing, tensioning and grouting of the rockbolts significantly. The permanent shotcrete was placed with remotely controlled spaying manipulators from MBT Meyco.

At most sections of the tunnels the actual rock conditions encountered required the rockbolts to be installed immediately behind the face. With the equipment utilized, this operation could be performed simultaneously to the excavation process. As indicated in the design, permanent shotcrete was placed parallel to excavation works as well. The muck has been hauled from the tunnel using conventional tunnel dumpers to a location from where it has been used as fill on the other road sections of the M5 project. Due to the environmental constraints in an urban area, it was required to store material from the night shift within the tunnel and muck out at daytime only.

Groundwater Monitoring. Another environmental requirement was that due to the tunneling no significant adverse impacts on the natural or built environment shall occur. Since the M5 is characterised by a drained, driven rock tunnel which passes under five alluvial infilled valleys, analysis and groundwater modeling indicated that there is a variable degree of hydraulic connectivity between the Hawkesbury Sandstone rock mass and overlying alluvial infill sediments within these valleys. In order to manage potential surface settlements, an Observational Method has been adopted. This method included the following elements:

- Carrying out of dilapidation surveys of structures which may be within the anticipated zones of surface influence,

- Monitoring groundwater levels, groundwater pressures, surface settlements and in-tunnel groundwater flows,
- Carrying out in-tunnel grouting as required to control significant flows,
- Carrying out minor repairs / remediation to surface assets as required.

With the actual groundwater inflow at approximately only 10 percent of the predicted and the connectivity only at a limited value, the surface settlements were kept at a minimum with values below 10 mm in general.

Quality Control. During construction, a comprehensive Quality Management Plan has been implemented. Part of this plan was to manage design related quality issues through a suite of measures:

- Requests for Information to existing design lots
- Engineering Change Proposals based on alterations to the existing design
- Non Conformance Reports for the managing of works executed not in accordance with the design.

The design consultant and its Construction Phase Service staff permanently on site played an active role in the quality management and its implementation

REFERENCES

Report on M5 East Motorway Geotechnical Investigations Driven Tunnels. To Hyder Consulting. March 1999. Golder Associates.

Cummins, A.B., and Given, I.A. 1973. *SME Mining Engineering Handbook, Volume 1*. New York: The American Institute of Mining, Metallurgical, and Petroleum Engineers, Inc.

Itasca Consulting Group, 1998, FLAC Fast Lagrangian Analysis of Continua, Itasca Consulting Group, Minneapolis, MN.

Sharp, J.C., Endersbee L.A., and Mellors, T.W. 1984. Design and Observed Performance of Permanent Cavern Excavations in Weak, Bedded Strata. P*roceedings of the ISRM/BGS.* Cambridge, 1984.

Page, C.H., and Stacy, T.R. 1986. *Practical Hanbook for Underground Rock Mechanics*. Clausthal-Zellerfeld, Germany: Trans Tech Publications.

Baxter, D.A. 1996. Do All Rockbolts Rust? Can Q.A. Help? But Does It Matter? Proceedings of the 9th Australian Tunnelling Conference, Sydney, Australia. AIM 1996.

Barrett, S.V.L. and McCreath, D.R. 1995. Shotcrete Support Design in Blocky Ground: Towards A Deterministic Approach. *Tunnelling and Underground Space Technology*, Vol.10, No. 1.