Alternative ground control strategies in underground construction

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ABSTRACT

Underground works vary from shallow urban tunnels to very deep tunnels and caverns in the world's great mountain ranges. The problems encountered at and between these extremes are entirely different and require appropriate approaches to site investigation, design and construction. The establishment of reliable financial estimates, construction schedules and contract proposals can only be done once a realistic geological model has been prepared and a clear understanding of the likely behaviour of the rock mass and the groundwater conditions has been established.

The conditions that control the behaviour of different kinds of excavations in a variety of geological environments are presented in the context of case histories. The aim is to provide project owners, financial managers, insurance companies and contractors with a road map that may assist them in avoiding some of the pitfalls and in considering some of the alternative strategies in the development of underground projects.

1 INTRODUCTION

Tunnels have been built for hundreds of years as part of transportation systems for people, goods, water and services. Until the middle of the last century these tunnels were generally small in size and the builders sought out the most favourable geology and topography in which to build them. With increasing population densities and growing international trade came the need for larger, longer and deeper tunnels through increasingly complex geological conditions. In addition, development of underground hydropower projects, gas and oil and dry goods storage facilities, as well as defence facilities, created a demand for large underground caverns, sometimes at considerable depth below surface. In parallel with these civil engineering projects, the mining industry has gradually moved toward deeper and larger underground operations with some gold mines in South Africa operating at depths of approximately 4 km below surface (Anonymous, 2011). In order to mine low grade deposits economically many underground mines employ mass mining techniques, such as block caving, in which the orebody is undercut and the ore drawn downward through ore-passes to extraction levels.

These advances have placed huge demands on the geologists and engineers who have to assemble the information and carry out the designs for the excavations required to meet the needs described above. Early texts on rock tunnelling (e.g. Terzaghi, 1946), while still useful for understanding some of the general concepts of tunnelling, are no longer appropriate for the design of many of underground excavations in use today or planned for tomorrow. In the following text an attempt is made to summarize the advances that have been made or which still have to be made to meet these challenges.

2 THE YACAMBÚ-QUIBOR TUNNEL IN VENEZUELA

2.1 Project background

The Yacambú-Quibor tunnel in the State of Lara in Venezuela will transfer water from the wet tropical Orinoco basin, on the eastern flank of the Andes, to the semi-arid Quibor valley on the western flank of the Andes. The agricultural and urban requirements of this semi-arid agricultural area, near the city of Barquisimeto, exceed currently available fresh water supplies and have resulted in a significant depletion of aquifers in the Quibor region.

The 4.0 m average internal diameter 24.3 kilometre long tunnel finally broke through on 27 July 2008 after 32 years of technical, financial and contractual problems. The principal technical issues that had to be overcome were the severe squeezing problems in very weak graphitic phyllites at depths of up to 1270 m below surface. Initial attempts to use an open-face TBM in 1976 failed as did attempts to use heavy support to resist squeezing. It was only after the introduction of yielding support in about 1991 that reasonable progress was made. Difficulties continued with floor heave in sections of the tunnel in which horseshoe profiles were used, even after the introduction of yielding support. Finally, in 2004, slow but steady progress was achieved after the Owner and the Contractor agreed that only a circular section, supported by steel sets with sliding joints and a 60 cm shotcrete lining, would be used. Emphasis was placed on developing a routine construction procedure, irrespective of the rock conditions encountered at the face. A detailed discussion on these problems and on methods used to overcome them has been published by Hoek and Guevara (2009).

A total of eight contracts were required to complete the driving of the tunnel. These are briefly described as follows:

First Contract (1976 to 1977). Two 4.8 m diameter open face Robbins hard rock Tunnel Boring Machines (TBMs) were mobilised for excavation from the Intake (Entrada) Portal and the Outlet (Salida) Portal. These machines were selected on the assumption that most of the rock that would be encountered would be of reasonable quality and strength, similar to that seen in the silicified phyllites at the dam site. In 1973 a consultant’s report contained the following statement "I imagine that much of the rock along the tunnel alignment will be fairly good phyllite, similar to that seen in the river channel at the dam site. In fact, possibly except for the Bocono Fault and some of the smaller ones, this could probably be essentially an unlined tunnel". An inclined access adit, with a portal located about 7.6 km from the Outlet Portal, was mined by conventional drill and blast methods. The purpose of this adit was to provide early access to the Bocono Fault so that this could be mined manually before the TBM arrived. In later years this included adit was utilised for ventilation.

Second Contract (1977 to 1979). The first and second contracts were operated by the same contractor and resulted in the Intake drive being advanced to a total of 1,700 m and the Outlet drive to a total of 1,850 m. In 1979 it became evident that the occurrence of the graphitic phyllite in the tunnel route was a serious problem. According to Dr Siegmund Babendererde (2002), the site manager for the TBM contract, the machine operated very well but significant convergence and floor heave started 50 to 100 m behind the TBM. The ground support system, designed for better rock conditions than those encountered, could not cope with the squeezing conditions. After the Intake drive TBM had advanced 1,700 m and was operating at a depth of 425 m below surface, the work was suspended during technical and contractual discussions. The TBM in the Outlet drive was removed from the tunnel at this time but the Inlet drive TBM was left in place and it was eventually trapped in the squeezing rock. It was excavated in 1987 during the fourth contract. It is interesting that the inclined adit was advanced a total distance of 1,200 m during the second contract and that, in order to deal with squeezing conditions, yielding support was used (Babendererde, 2002). Unfortunately, this European technique for dealing with squeezing conditions was not used in the main drives until the fifth contract (1991 to 1997).

Third Contract (1981 to 1984) and Fourth Contract (1984 to 1988). The same contractor used drill and blast excavation in the Outlet drive and the inclined adit. The Intake drive, blocked by the TBM, was not worked on during the third contract and the TBM was removed in 1987 during the fourth contract.

Fifth Contract (1991 to 1997), awarded to a new contractor, utilised conventional drill and blast in the Outlet drive and a roadheader in the Intake drive. This roadheader operated with mixed results and it was eventually abandoned. The contractual period expired and the project was re-bid.

The Sixth Contract (1997 to 2002), Seventh Contract (2002 to 2005) and Eighth Contract (2005 to 2008) were all carried out by the same Venezuelan contractor using conventional drill and blast methods. The final break-through occurred on 27 July 2008.

A ninth contract for the repair and final lining of some sections of the tunnel is currently in progress.

2.2 Lessons learned

Many important lessons related to the theme of this symposium were learned during the Yacambú-Quibor project. Probably the most important of these was that, in spite of complexity of the tectonic environment in which the project is located, shown in Figure 1, there was no reliable Geological model and that no serious effort was made to define the geotechnical characteristics of the rock types encountered along the tunnel. Surface mapping had revealed the presence of two major faults, one of which was encountered in the tunnel. One vertical borehole from surface was attempted at close to the maximum depth of the tunnel but this was abandoned at about 300 m depths due to drilling problems.

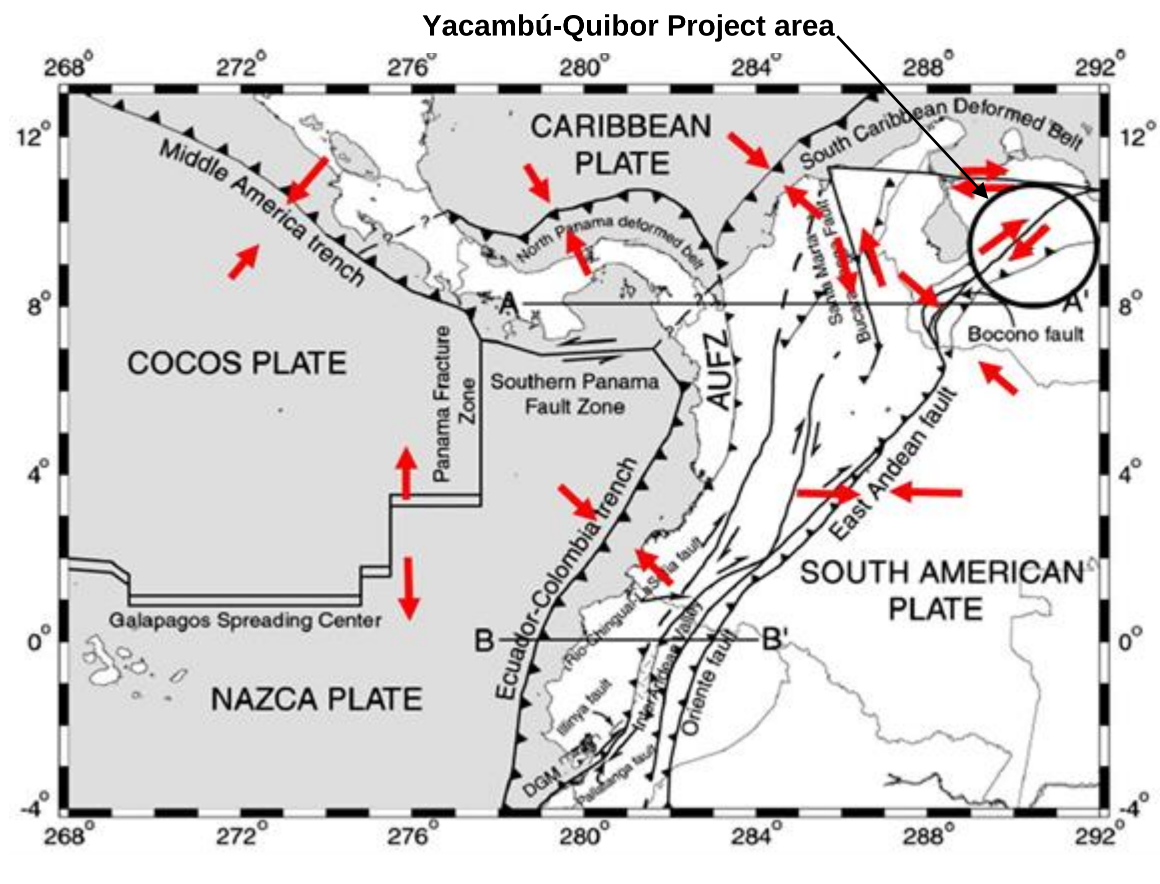


Figure 1 Tectonic plates in the north-western region of South America and Panama. The Yacambú-Quibor project is located in the circled area in the upper right of the figure. After Trekamp et al (2002) with additions by Diederichs (2008).

The graphitic phyllite encountered for significant lengths of the tunnel had been severely sheared by tectonic activity and its strength was very low. The resulting deformation of the tunnel overwhelmed the support, designed for much lower deformations. Even when the presence and behaviour of this graphitic phyllite had become obvious during the first and second contracts, the warning signs had not been heeded and tunnelling continued with horseshoe shaped tunnels using inadequate steel sets and shotcrete linings (Figure 2). The basic principles of tunnel support in squeezing ground were not understood by the designers until the fifth contract, in spite of the fact that these principles had been applied during the driving of the inclined adit in the second contract as described above. The use of a circular tunnel profile with yielding steel sets and a full shotcrete lining (Figure 3) was only fully implemented during the final three contracts (Hoek ad Guevara, 2009).

The original five contracts were all traditional fixed price contracts with disputes resolved by Disputed Review Boards or by litigation. The final three contracts were based on an agreed fixed price per metre of tunnel mined. With almost 20 years of tunnelling experience, the actual cost of mining the tunnel was well known and this was used as a basis for contract negotiations.

Figure 2 Re-mining and re-lining of a collapsed section of horseshoe shaped tunnel supported by steel sets and a thin shotcrete shell.

Figure 3 A circular tunnel over-excavated to 5.2 m diameter and supported by steel sets fitted with two sliding joints which allowed 60 cm of movement resulting in a final diameter of 5.0 m. The sets are embedded in 20 cm of shotcrete except for 1 m wide windows over the sliding joints. These windows were filled 15 m behind the face, when the sliding joints had generally closed, and an additional 40 cm of shotcrete was added. The sheared nature of the graphitic phyllite is evident in the face.

3 THE OLMOS TRANSANDINO TUNNEL IN PERU

3.1 Project background

The Olmos Transandino Tunnel is part of a multi-phase hydroelectric and irrigation project currently being developed by The Regional Government of Lambayeque, Peru. The project consists of a recently constructed dam on the Huancabamba River and a 19.3 km long water diversion tunnel that will convey water from the east side of the Andes to the west, providing irrigation for towns on the Peruvian Pacific coast. Future phases will include increasing the height of the dam and the construction of a hydroelectric dam downstream of the tunnel outlet.

The original proposal for the project dates back to 1924 but feasibility studies were only conducted in the 1960s. Tunnel excavation commenced in the late 1970s but work was halted in the 1980s due to a lack of funding. Construction of the dam and excavation of the remaining 13.9 km of tunnel was opened to international public bidding in the early 2000s. In July 2004 an agreement was signed with Concesionaria Trasvase Olmos with the contractor Odebrecht Peru, Engineering and Construction, responsible for driving the tunnel by means of a TBM.

This concession is in the form of a 20 year Design, Build, Own, Operate and Transfer contract. This arrangement gives the contractor a very high incentive for constructing the tunnel as quickly and efficiently as possible while, at the same time, ensuring that it can be operated safely and economically for a long period of time.

3.2 Project details

Excavation of the tunnel commenced in March 2007 and it was completed on 20 December, 2011. A 5.3 m Robbins Main Beam TBM was used for the 13.9 km drive under a rock cover reaching approximately 2000 m (Roby et al, 2009).

The main geological units include metamorphic basement rocks (schists) of Paleozoic age, flows of extrusive rock including andesites and dacites of Jurassic, Tertiary and Paleocene age, and intrusive rocks including granodiorite and volcanic flow rocks (tuffs) of Cretaceous and Paleocene age. The original interpretation of the geology specific to the tunnel alignment was based upon detailed mapping of the topography directly above the tunnel and upon two exploratory boreholes. This interpretation, published in 1982, was made by Russian engineers who were responsible for the first contract. The geological cross-section was re-interpreted by Concesionaria Trasvase Olmos who also constructed an as-built geological cross-section. The geologic units encountered are those which were predicted by the Russian geologists although the actual distribution of the rock units varied from those predicted.

Spalling and rockbursting have been an issue throughout the driving of the tunnel with more than 10,000 events being quoted in some publications. Care has to be taken to differentiate between these phenomena. Spalling or popping is a relatively local brittle failure of the excavation boundaries which is sometimes accompanied by snapping or popping sounds with a relatively minor energy release. Rockbursts result in “damage to an excavation that occurs in a sudden or violent manner and is associated with a seismic event” (Kaiser et al., 1995 and Kaiser and Tannant, 1999).

Both spalling and rockbursting are induced by high in situ stresses. The location of spalling in the roof, as was common in Olmos, indicates that the horizontal stresses are higher than the vertical stresses. Bursting of the face, which was one of the more serious types of failure in Olmos, occurs when the horizontal stress parallel to the tunnel axis is higher than the vertical stress. Reliable measurement of all the in situ stresses at depths in excess of 1000 m is not practical and hence it is not possible to predict the location and magnitude of rockburst events. However, in the case of Olmos, it was found that transitions from rhyolite, latite and granodiorite into dacite were marked by severe bursting. The dacites contain persistent sub-vertical structures that interact with the accumulation of stress-induced fractures to guide the fracturing process outwards creating large volumes of damaged rock that can then fail instantaneously along the structures creating rockbursts with extensive overbreak.

Until December 2008 the tunnel suffered from ongoing spalling and popping but this was not a serious impediment to progress and an advance rate of 12.6 m per day was maintained with over 8.4 km of tunnel being completed in 22 months. On 22 December 2008 the tunnel encountered serious rockbursting in dacites and the advance rate dropped to 2.7 m per day. Several serious rockbursts occurred and the largest of these, on 29 April 2010, resulted in significant damage to the TBM which was not able to restart operations until 8 August, 2010.

In May 2011 a transition from dacite into basement schist occurred and the rockburst problem was reduced. Advance rates picked up again and a completion date for the tunnel was projected for November 2011. However, Consortium Trasvase Olmos suspended work in June, claiming it had suffered a loss of revenue of US $70 million as a result of delays arising from the rockburst problems about which they had not been adequately informed. Work resumed in October and the tunnel broke through on 20 December 2011 (Vigo, M. 2011).

3.3 Lessons learned

The high stress problems in the strong brittle rock mass through which the Olmos tunnel was driven were responsible for spalling and rockbursts which resulted in significant delays in completion of the tunnel. Reasonable geological predictions were available and the maximum cover of 2000 m suggested that stress induced failure could be a problem in driving this tunnel. However, the magnitude of the rockbursts and the overbreak that occurred could not be predicted and this presented a major challenge in excavating this tunnel and will continue to present similar challenges in driving future tunnels in hard rock at these depths.

The World Stress Map (Heidbach, 2008) of the project area, reproduced in Figure 4, shows that the major horizontal stress is generally parallel to the trans Andean Olmos tunnel axis and this is confirmed by the experience of rockbursting ahead of the TBM face. However, the World Stress Map gives only stress directions and the magnitudes are very difficult to establish at these depths.

Direct in situ stress measurements from surface are typically limited to a depth of less than 100 m. Measurements have been carried out successfully to depths of 500 m but, due to the complexity of manipulating equipment at that depth, the success rate is very low. Hydraulic fracture techniques for stress measurement only give reliable measurements of the minimum principal stress and, where this is vertical as in the case of the Olmos tunnel, these techniques do not help. Consequently, at this time, horizontal in situ stresses in the rock surrounding very deep tunnels cannot be measured directly during site investigations and this makes it very difficult to predict spalling and rockbursting accurately and to plan for dealing with these problems when encountered.

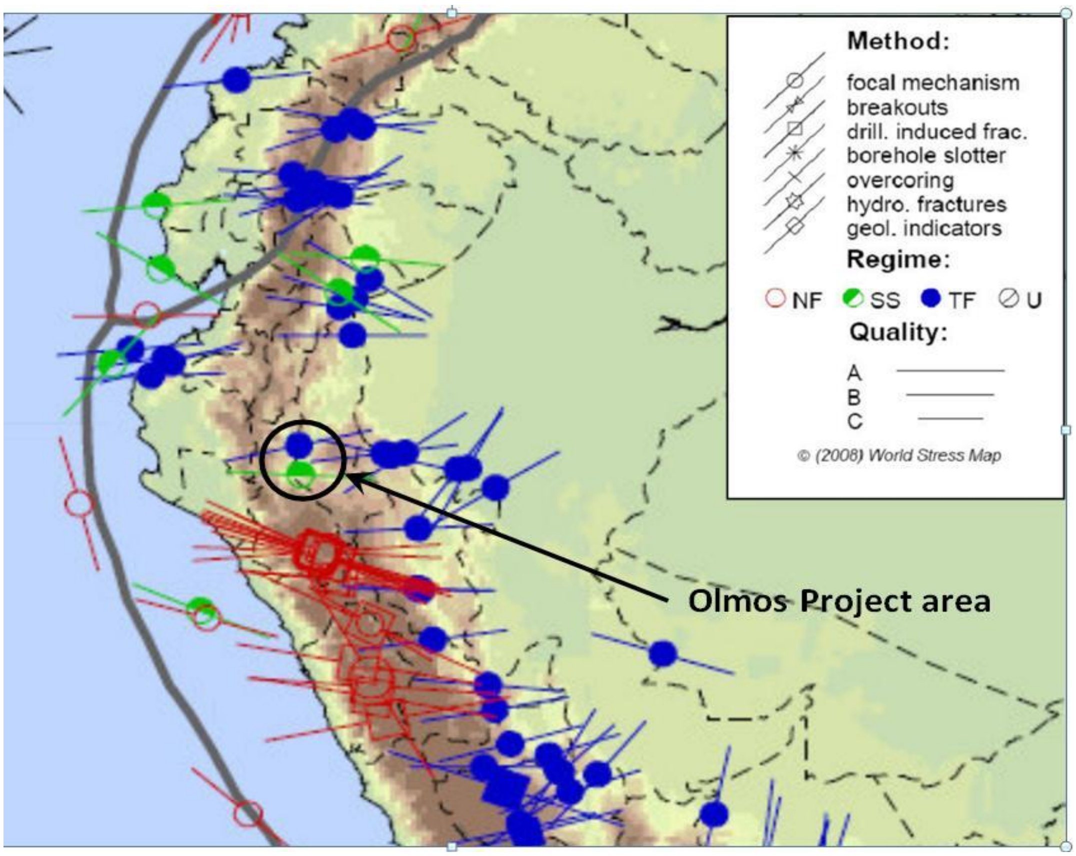


Figure 4 World Stress Map detail of the Olmos project area in Peru.

In planning the excavation and support of the 13.9 km of TBM driven tunnel the contractor chose a robust and powerful open face hard rock TBM. The machine was fitted with a short shield in order to minimise the danger of the machine being trapped by surrounding debris in the event of a rockfall, spall or burst. The support system, illustrated in Figure 5, consists of a precast concrete invert, with a drainage channel and rail mounts included, and continuous steel sets spaced at 1 m placed in spaces in this invert.

The advantage of this support system is that the tunnel invert water is controlled and the inset rail mounts allow accurate alignment of the rails which, in turn, translates into reliable high speed train movements which are critical in maintaining delivery of materials and equipment and in removal of muck from the tunnel. These are important practical considerations since time lost in drainage and in derailments or slow travel can have a major cumulative impact on the construction schedule.

The steel sets, while not sufficiently robust to withstand major loads from a rockburst or rockfall, provide a safe canopy under which the miners can operate. Wire mesh or rebar mats placed over the top half of the sets prevent small pieces of rock falling on the miners. In the event of a damaging burst or fall the sets can be severely deformed but they still provide some protection and are relatively simple to replace once the area has been stabilised. The one benefit of rockbursts is that once the energy has been released the rock tends to stabilise and further events in the same location are unlikely. Hence, by allowing the ground to settle for approximately 30 minutes after a burst, the area can be re-entered safely. The steel sets are fully embedded in high quality robot applied shotcrete immediately behind the trailing gear of the TBM and this results in a completed tunnel as shown in Figure 6.

This system of precast concrete inverts, regular steel set installation and shotcrete application as an off-line activity behind the TBM trailing gear is a highly efficient process in which each miner knows exactly what to do and the overall schedule can be tightly controlled as in a factory production line operation. Of course, when a serious rockburst or rockfall occurs, the advance of the face stops but the facilities to drain the tunnel and to move the equipment required for repair to the face remain fully operational, allowing the time required for the repair to be minimised.

Figure 5 Precast concrete invert sections and steel sets used to support the Olmos tunnel.

Figure 6 Fully shotcreted tunnel behind the TBM trailing gear.

The design-build-own-operate type of contract used in the construction of the Olmos tunnel creates a very high incentive for the contractor to work quickly and efficiently and to produce a high quality end product which will operate safely and efficiently during the concession period and beyond. As was the case in Olmos, this type of contract does not guarantee that there will be no disputes or claims but these are generally limited to very specific issues which may be simpler to resolve than in conventional fixed price contracts.

Finally, it is worth exploring whether the Geological Data Report and the Geotechnical Baseline Report concept, which has been widely adopted in North America and is gaining acceptance in other countries, would have helped in the case of the Olmos tunnel? The Geological Data Report, which is a compilation of all of the results of the site investigation process, has been in use for many years. However, this report is generally restricted to factual information and it does not include very much interpretation. The contractor is left to assess the factual information and draw conclusions on the probable groundwater and rock mass behaviour; tasks that may be very difficult to accommodate during the bidding process.

The Geotechnical Baseline Report (URTC, 1997, 2007) takes this process one step further. It is an interpretative report in which all the factual data collected during the site investigation stages are analysed in terms of potential groundwater and rock mass behaviour and other issues that could cause problems during construction. These interpretations and recommended solutions are presented in the report and form a behavioural baseline which can be used in setting contractual limits. The contractor cannot make claims for ground behaviour which falls at or above the baseline while the owner has to accept responsibility for problems resulting from rock mass behaviour which is worse than that predicted in the baseline report.

Even if all the questions cannot be fully resolved, the preparation of the Geotechnical Baseline Report forces the geologists, geotechnical engineers and design engineers to consider the questions that they are required to address very carefully. Has a reliable geological model been prepared? Has the pre-construction groundwater distribution been studied and the rock mass permeability investigated so that predictions of groundwater movement during construction can be made? Have sufficient high quality diamond drill cores been recovered, logged and tested in the laboratory? Have the in situ stresses been measured or, if not, has an attempt been made to assess these stresses from measured stresses on nearby projects or from geological reasoning?

In the case of the Olmos project and similar deep tunnels, the problem of determining the in situ stresses is a difficult one to resolve. There are several examples of tunnels where high (and sometimes low) in situ stresses have caused significant construction problems. In most cases, the in situ stresses had not been accurately predicted nor the danger of spalling or rockbursts fully assessed before the start of construction. As discussed earlier, reliable direct measurement of in situ stresses is a complex problem in high cover situations with no intermediate access to the deepest sections of the tunnel alignment. It is anticipated that this will remain a technical problem for years to come. It is hoped that the presentation of case histories such as that of the Olmos tunnel will alert owners, contract managers and insurance companies to these problems but also show that they can be overcome by logical contractual procedures.

4 REDUCING GEOLOGIAL RISK IN TBM TUNNELLING

4.1 Background

The geological conditions and the stability of the rock mass in which a tunnel or cavern is to be excavated are probably the greatest sources of risk in a project involving underground construction. In the absence of a reliable geological model the project can go seriously wrong. Even when a good geological model is available, the interpretations of the rock mass characteristics and of the behaviour of the excavations are not trivial tasks and construction problems cannot be avoided completely, irrespective of the type of contract adopted.

Given this situation it is appropriate to ask whether there is anything else that can be done to alleviate the risk, particularly for long, deep tunnels which will become more common as the demands for more transportation routes, water diversion projects, hydropower developments continue to grow. Fortunately, there is a viable option that involves making the tunnelling less sensitive to geological and geotechnical uncertainty by adopting a tunnel lining strategy that is as independent as possible from the geological conditions.

One example of this approach has already been discussed in the case history of the Olmos tunnel in Peru. While it had been anticipated that there would be problems due to over-stressing of the rock mass surrounding the tunnel, the magnitude and frequency of the spalling and rockbursts could not be estimated with any degree of reliability. It was therefore decided to utilise a support system that could be installed routinely throughout the tunnel, irrespective of the conditions encountered. This support system, using precast concrete invert segments, steel sets and full embedment in shotcrete, was designed to cope with typical overstressing problems and it was set up to maximise production in the tunnel. Unusually heavy rockbursts overwhelmed this support system from time to time but the resulting problems proved possible to repair and the tunnel was completed successfully, albeit with significant delays.

There are several other examples where this approach has been applied deliberately and where very good results were obtained in a wide variety of geological conditions. One of these projects is discussed in the following section.

4.2 Yellow River Diversion Project in Shanxi Province, China

The Yellow River diversion project includes more than 300 km of tunnels and conduits, treatment plants and pumping stations. It is designed to divert water from the Yellow River to meet the critical water supply needs of the Shanxi provincial capital of Tai Yuan and, in the future, the city of Da Tong. The project component dealt with here covers four 4.9 m diameter TBM driven tunnels with a total length of 88.7 km (Wallis, 2009, Kolić et al, 2009, Lampiano et al, 2001).

The project is located in the Gobi desert in the dry north-west corner of China. A summary of the topography and predominant rock types as well as the performance of the 4 TBMs is presented in Figure 7. It can be seen that the main rock types encountered by Tunnels 4, 5 and 6 are limestones and dolomites with frequent karst features and bands of soft plastic clay. Tunnel 7 encountered coal measure rocks with gas as well as Triassic sandstone and mudstone. Water inflow was limited in Tunnels 4, 5 and 6 but was abundant in Tunnel 7.

Starting in 1989/1999, four double shield TBMs were deployed to excavate these tunnels. Two new Robbins machines, one refurbished Robbins machine and one new NFM-Boretec machine were used. All the TBMs were fitted with back-loading 17 inch (432 mm) disk cutters and the new machines were all equipped with variable speed electric motors. The used Robbins TBM was fitted with two-speed electric motors with gear reducers and hydraulic clutches. All the machines were operated on three 8 hour shifts per 24 hours for 6 days per week.

The outstanding performance of these four TBMs is due largely to the use of precast concrete Honeycomb segments, illustrated in Figures 8 and 9. These segments are installed within the tail shield of the TBM with two opposing segments being installed while thrust to move the machine forward is reacted by the other two segments. Pea gravel is pumped into the gap between the tunnel walls and the tail shield, immediately behind the machine. About 60 m behind the machine this pea gravel is grouted to complete the lining installation.

Having the rails fixed accurately to the precast invert allowed train speeds of 20 km/hour to be maintained to ensure timely delivery of segments and supplies to the face and the efficient removal of muck from the tunnel. These are critical factors in maintaining a tightly controlled schedule in this type of tunnelling operation.

4.3 Lessons learned

The example of the Yellow River diversion project in China and a very similar outcome in driving a 12.2 km long 4.88 m diameter tunnel for the Guadiaro-Majaceite water project in Spain (Castello et al, 1999) demonstrates the utilisation of double shield TBMs with simultaneous installation of precast concrete liners within the tail shield of the machines. The ability to maintain a continuous supply of concrete segments and equipment and to remove the muck from the tunnel by means of trains running on accurately aligned rail set on the invert segment was a critical factor in achieving the very high excavation rates.

More importantly, the utilization of this system meant that the tunnel drives were effectively independent of the geological conditions through which the tunnels were excavated. There was no need for endless discussions at the tunnel face about the class of the ground, the type of support to be installed, whether rockbolts should be used and how long they should be. The segments were designed to deal with all of the support issues and to provide a water-tight one pass lining.

Of course there were problems and delays in all of these tunnels. Cutting heads were replaced, gearboxes repaired, TBMs trapped in squeezing ground had to be freed and some soft ground sections had to be excavated by hand. However, the delays caused by these problems were of minor significance in terms of the overall project schedule.

It is not suggested that this approach is universally applicable to tunnelling. However, these examples do demonstrate that “thinking outside the box” can sometimes reduce the number of impediments encountered in tunnel driving and, in particular, isolate the tunnel driving process from some of the geological and geotechnical uncertainties or changed ground conditions that can cause so many problems.

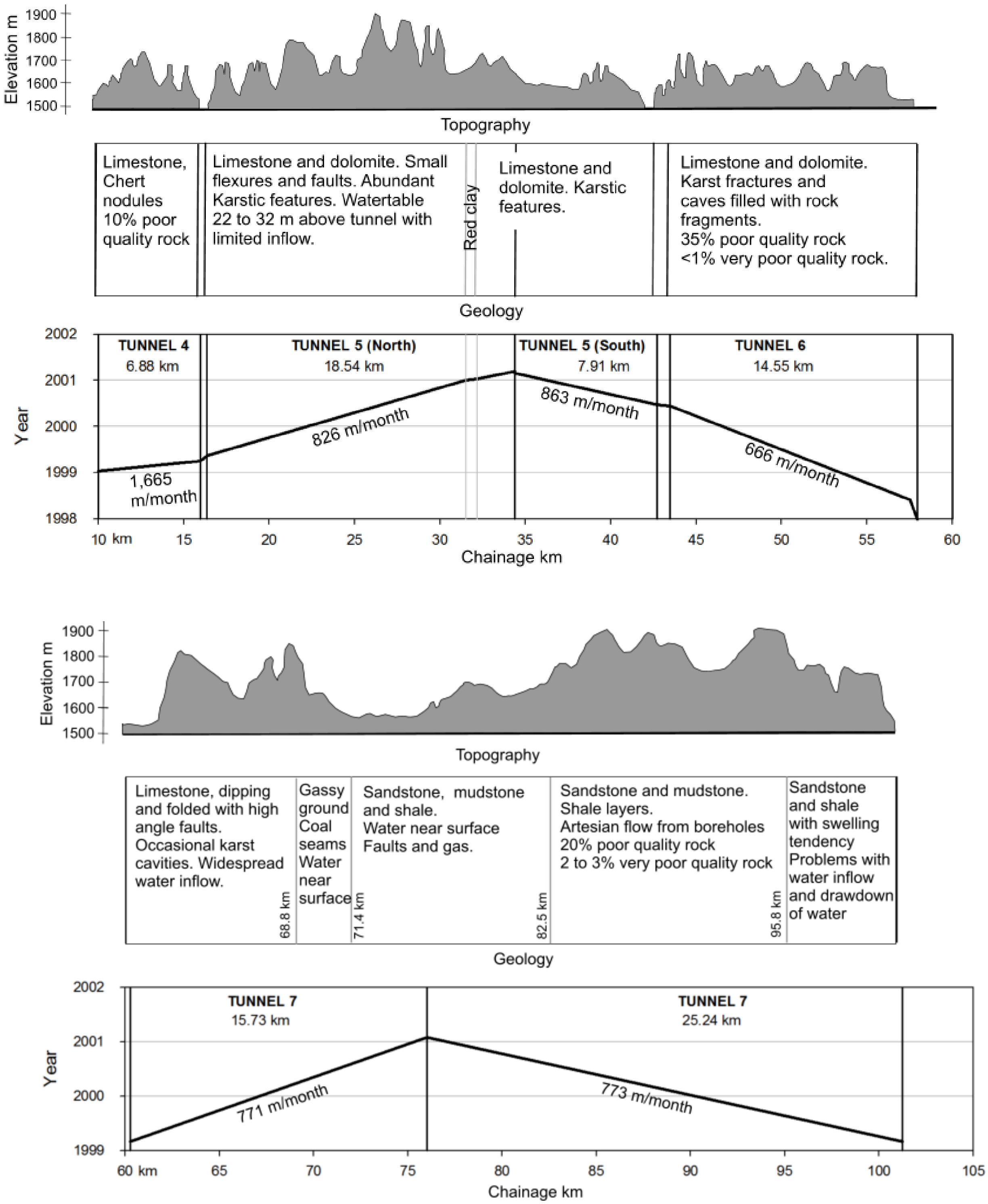


Figure 7 Topography, geology and tunnel performance for Tunnels 4, 5, 6 and 7 of the Yellow River Diversion Project in China. (After Babendererde, 2007)

Figure 8 Assembly of Honeycomb pre-cast concrete segmental lining showing the interlocking of the segments. The rail mounts and drainage channel are cast into the invert segment. Photograph provided by Dr Siegmund Babendererde.

Figure 9 Segments with rubber sealing gaskets to allow grouting of the space between the bored tunnel walls and the lining and also to prevent loss of water from the operating tunnel.

5 NATHPA-JHAKRI HYDROELECTRIC PROJECT IN INDIA

5.1 Project background

The Nathpa-Jhakri hydroelectric project is located in the Himalayan foothills in the state of Himachal Pradesh in India and it consists of the following components:

- 40 m high concrete gravity dam across the Satluj river
- 4 x 525 m long x 27.5 m high x 16.3 m wide desilting chambers,
- 27.4 km long 10 m diameter headrace tunnel,
- 301 m deep 21.6 m diameter surge shaft,
- 222 m long, 20 m span x 49 m high underground powerhouse,
- 196 m long x 18 m span x 27.5 m high underground transformer hall and
- 983 m long 10 m diameter tailrace tunnel.

Construction commenced in 1993 with commissioning in May 2004. The project operates at a head of 428 m and produces 1500 MW of power. A comprehensive description of the project by the Geological Survey of India entitled “Nathpa-Jhakri hydroelectric project, Himachal Pradesh, India” can be found at http://en.wikipedia.org/wiki/Nathpa_Jhakri_Dam.

The Geological Survey of India was responsible for the geological site investigations which included 24 boreholes (2575 m of core) and 7 exploratory adits. Excellent geological maps were produced and the conditions encountered during construction were generally in accordance with these maps.

A traditional fixed-price contract was used in accordance with the owner’s normal procedure. International bids were invited and three separate contracts were awarded for the dam and upstream works, the headrace tunnel and the surge shaft and the underground caverns and tailrace tunnel.

A complete discussion on this project exceeds the scope of this paper and the following presentation is limited to the excavation of the headrace tunnel through the Daj Khad fault zone.

5.2 Daj Khad fault zone

The 400 m wide Daj Khad fault zone had been accurately predicted in the geological model but the characteristics of the rock mass were not well defined and it was anticipated that conventional steel set support would be sufficient for the excavation of this zone. Figure 10 shows significant deformation in the saturated and heavily sheared gouge encountered in the top heading of the tunnel. The contractor was unable to stabilize the fault zone using steel set support and other methods available to him.

After lengthy discussions between the designers, the contractor and the project owner’s Panel of Experts, it was decided to bring in the Italian consulting company Geodata to assist. They recommended stabilization of the tunnel face by means of drainage and the use of 12 m long grouted pipe forepoles as illustrated in Figures 11 and 12. This method has been used successfully in the past (Carrieri et al, 1991) and, although very expensive, it was considered to be the most appropriate approach for this situation. The zone was excavated successfully with the contractor being paid on a time and materials basis for his work.

5.3 Lessons learned

The Nathpa-Jhakri hydroelectric project is a very large and complex project which was successfully completed using site investigation, design and construction methods which are typical of those used by large state-owned hydroelectric power corporations. An issue that required outside help was the stabilisation of the headrace tunnel through the Daj Khad fault as described above. This was handled as a special item in the contract and paid for on a time and materials basis.

Figure 10 Squeezing of the headrace tunnel top heading in the Daj Khad fault zone.

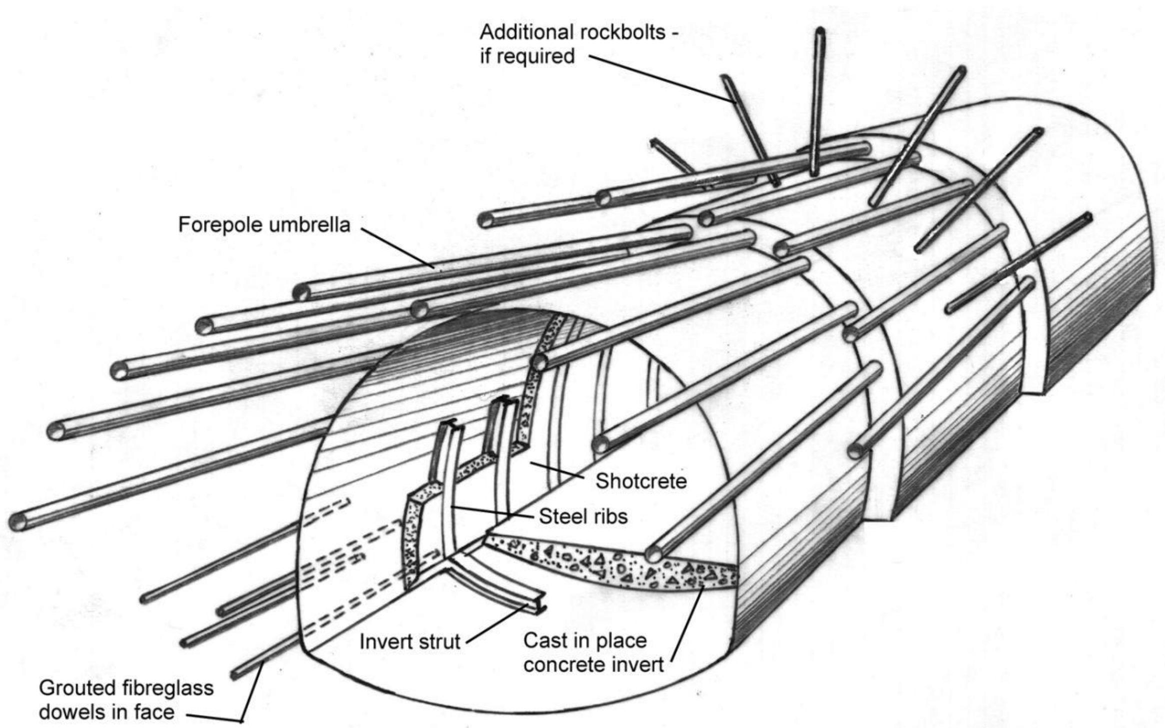


Figure 11 Advancing a tunnel under a forepole umbrella. Note that not all of these components were used in the Nathpa Jhakri project.

Figure 12 Excavating through the Daj Khad fault zone using forepoles.

Within the three contracts for the complete project there were problems and delays of the type that can be anticipated in large complex underground projects of this kind. These were resolved with the aid of a Disputes Review Board that met regularly. Where an unanticipated problem is encountered and this problem has a relatively minor impact on the cost and schedule of the entire project, as in the case of the excavation of the Daj Khad fault, dealing with this problem by means of a change order or a small sub-contract is probably the simplest and most efficient solution. When the unanticipated problems cannot be resolved with the tools available, as for the Yacambú-Quibor case discussed earlier, it may be more effective to terminate the contract as soon as possible and to reassess the entire project before proceeding. Of course, changing contracts mid-way is never a simple process and all the implications of this course of action have to be considered very carefully before taking this route.

In many countries the state-owned organisations, of the type responsible for the Nathpa-Jhakri project, have been largely disbanded. Some of these organisations have been privatised while others have been broken up and the components privatised. It is not unusual to find a small group of administrative staff managing a variety of consultants and contractors who are responsible for most of the tasks originally performed by the organisations themselves. While there is nothing fundamentally wrong with this new model, the lack of a of a pool of experienced people, who have worked together for many years, can give rise to technical and contractual problems that will be more difficult to resolve and which may require different types of contractual arrangements.

6 MINGTAN PUMPED STORAGE PROJECT IN TAIWAN

6.1 Project background

The Mingtan Pumped Storage Project in Taiwan has an installed capacity of 1600 MW with six reversible pump-turbines housed in an underground cavern 300 m below the ground surface. The upper reservoir is the existing Sun Moon Lake which provides a maximum static head of 403 m.

The underground powerhouse complex consists of two caverns. The main powerhouse cavern is horseshoe shaped with a span of 22 m and a height of 46 m. The transformer cavern is located 45 m downstream of the powerhouse cavern and is also horseshoe shaped with a span of 12 m and a height of 17 m. These caverns are located in a predominantly sandstone formation, dipping at 35 degrees, with relatively weak siltstone layers up to 2 m thick. Most of the bedding planes and contacts between rock types are sheared as a result of previous tectonic movements. (Cheng and Lui, 1990, Liu and Hsieh, 1991). A detailed description of the geotechnical aspects of the project will be found in Hoek (2007).

6.2 Improving the rock mass above the underground caverns

While the overall contract for the project was a typical unit price contract, an unusual feature was that there was a relative large preliminary contract during which extensive site investigation and construction were carried out. This included site investigation and in situ tests in existing exploration/drainage adits, 10 m above the powerhouse and transformer caverns, as well as the construction of many of the access roads, the laydown areas and the contractor's camp site. This preliminary contract also provided the opportunity for significant rock improvement works to be carried out in the rock above the powerhouse and transformer caverns, so that main contractor could work efficiently in "good rock" conditions.

Detailed mapping during site investigation had defined a significant number of dipping fault structures crossing both the powerhouse and the transformer caverns. An isometric view of a typical fault plane is reproduced in Figure 13. The influence of these faults on the stability of the cavern was of major concern. It was decided that pre-treatment of the cavern roof was necessary in order to ensure that the main contract could proceed without severe problems due to roof instability. This pre-treatment consisted of removal and replacement of the clay seams in the faults to the maximum extent possible, followed by reinforcement of the rock mass in the roof by means of grouted cables.

The treatment of the faults involved high pressure washing of the clay seams and backfilling the voids with non-shrinking concrete. This technique was developed for the treatment of similar faults in the foundation of the Feitsui arch dam near Taipei (Cheng, 1987). Figure 14 shows the arrangement of longitudinal working galleries and cross-cuts used to access the clay seams. It was found that the clay washing and replacement could be carried out to a depth of about 4 m. The thickest and weakest fault was excavated manually and backfilled to a similar depth.

Once the clay seam treatment process had been completed the rock mass above the caverns was reinforced by means of 50 tonne capacity cables as shown in Figure 15. These cables were installed downwards from the central exploration/drainage adits and upwards from the two longitudinal working galleries. Since these cables were installed before any excavation had taken place in the caverns they were untensioned except for a few tons of straightening load. Deformation of the rock mass during excavation of the caverns resulted in tensioning of the cables. The fully excavated powerhouse cavern roof is illustrated in Figure 16. The sequence of excavation and reinforcement of the powerhouse cavern is illustrated in Figure 17.

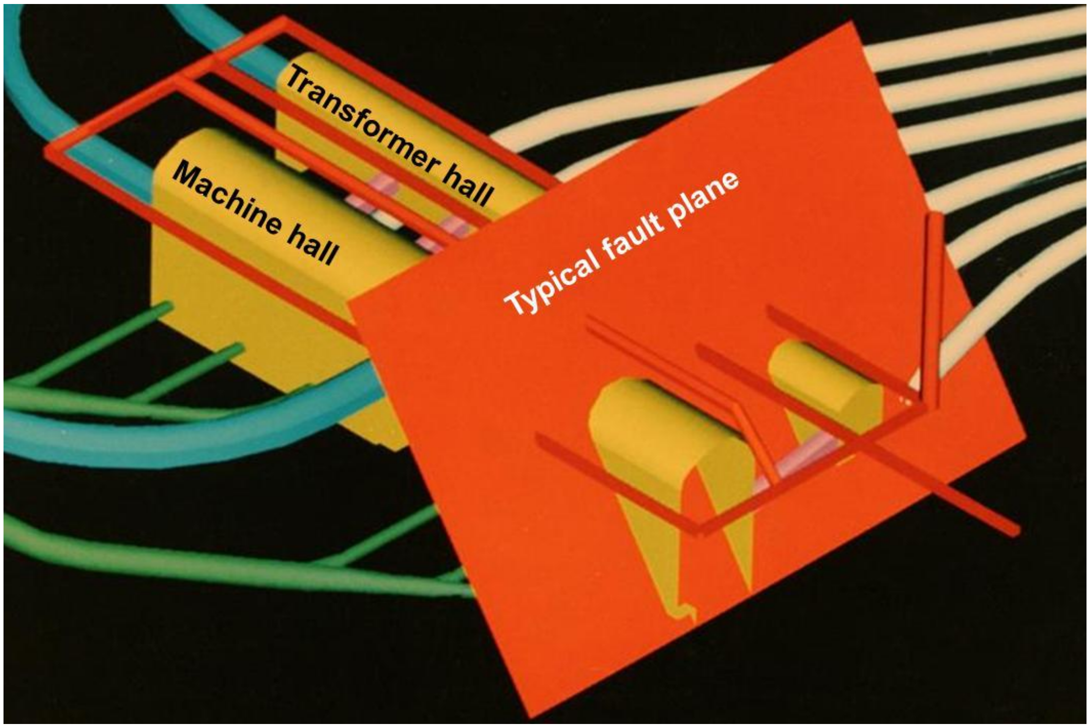


Figure 13 Isometric view of underground power and transformer caverns showing a typical fault plane crossing the caverns.

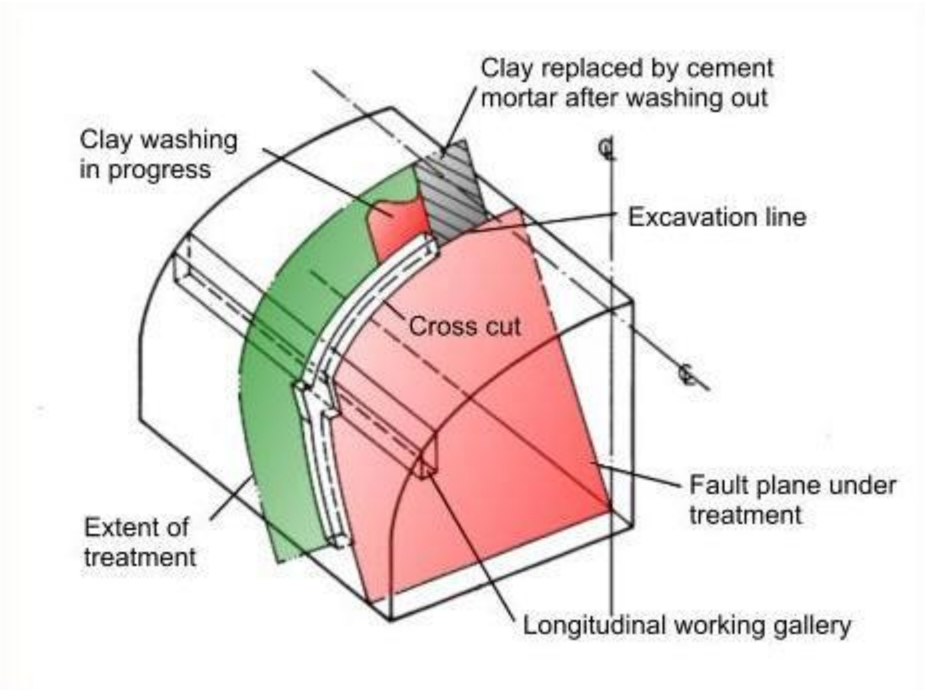


Figure 14 Washing and replacement of clay seams in the faults encountered in the roof and upper sidewalls of the Mingtan power cavern.

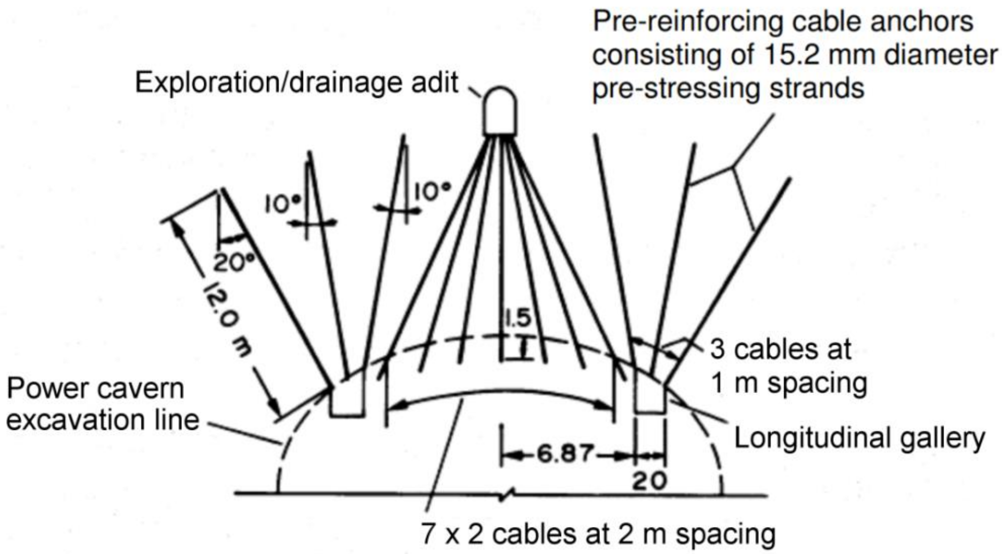


Figure 15 Pre-reinforcement of the power cavern roof by means of grouted untensioned cables placed from the longitudinal working galleries and from an existing exploration and drainage gallery 10 m above the cavern roof.

Figure 16 Excavated Mingtan powerhouse arch showing some of the reinforcing cables before they were trimmed and shotcrete applied.

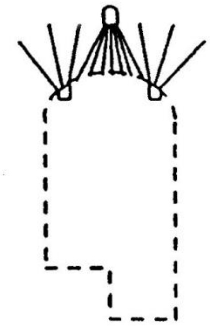
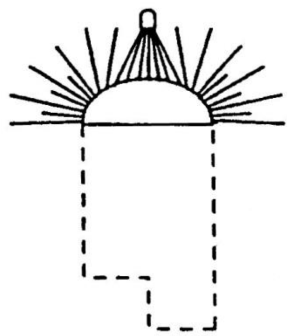
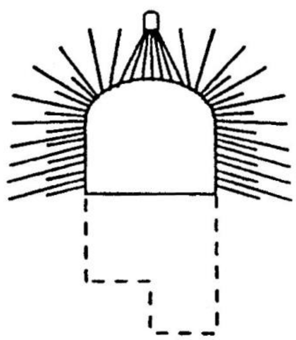
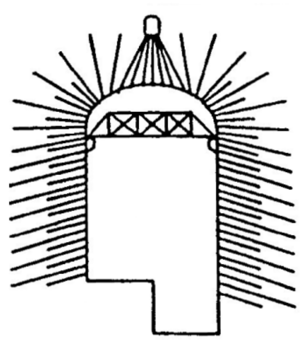
	Installation of double corrosion protected cables from exploration/drainage gallery located 10 m above centre of roof arch and from two longitudinal working galleries. The 50 tonne capacity cables were installed on a 2 m x 2 m grid pattern and a small straightening load of 5 tonnes was applied before grouting. Dashed line shows cavern profile before excavation
	Excavation of cavern roof from a centre heading with slashing of the sides and the application of the first 50 mm of steel fibre reinforced micro-silica shotcrete. End fixings and faceplates were added to projecting ends of the cables which were tensioned to 20% of ultimate capacity to ensure positive anchorage. Where required, 5 m long 25 mm mechanically anchored, tensioned and grouted rockbolts were installed at the centres of the 2 m x 2 m grid of reinforcing cables.
	Excavation of the cavern by 2.5 m vertical benches. Double corrosion protected 112 tonne capacity cables, inclined downwards at 15° to cross dipping bedding planes, were installed on a 3 m x 3 m grid in the sidewalls. Before grouting these were tensioned to 38 to 45% of yield strength, depending upon their location relative to the bench. Intermediate 6 m long 25 mm diameter tensioned and grouted rockbolts were installed at the centres of the pattern of reinforcing cables. Final shotcreting of the roof was carried out at an early stage of benching.
	Complete excavation of the cavern with 150 mm total thickness of steel fibre reinforced micro-silica shotcrete on the roof and upper sidewalls and 50 mm thickness on the lower sidewalls. Access to the roof for inspection and minor remedial work was provided by a temporary construction crane.

Figure 17 Sequence of excavation and reinforcement of the powerhouse cavern.

Note that a 22 m span powerhouse cavern cannot be supported by means of steel sets or thin shotcrete linings since these do not have sufficient capacity to resist rock movements. In the past the arches of many underground powerhouses have been supported by reinforced concrete arches but, in deformable sedimentary rock such as that in which the Mingtan cavern was excavated, these arches are too stiff and can fail as a result of the lateral pinching action which occurs as the cavern walls converge during excavation of the lower benches. Reinforcement by means of cables improves the overall strength of the rock mass and results in a much more flexible system which can accommodate the progressive convergence of large caverns during excavation.

Cables, such as those used in the rock mass above the Mingtan cavern arches, can only be left untensioned if they are installed before excavation of the cavern. Once excavation of the cavern commences, the cables in the lower portion of the arch and in the sidewalls must be tensioned to a load calculated on the basis of the amount of deformation to which each cable will be subjected

The shotcrete used as a final internal lining is designed to support the rock pieces that can become detached between the cable faceplates, typically installed on a 2 m x 2 m grid pattern. Wire mesh or steel fibre reinforcement is generally used to improve the tensile capacity of these shotcrete layers. The support provided by the thin layer of shotcrete, typically about 150 mm thick, is ignored in calculating the required capacity of the reinforcing cables.

6.3 Lessons learned

The identification and treatment of the vulnerable rock masses above the powerhouse and transformer caverns during a preliminary contract meant that a conventional fixed price contract could be applied with confidence to the main contract and that it worked successfully. This option is not always available in underground construction, particularly in long, deep tunnels where it is difficult to gather sufficient information before construction and where the opportunity to implement such measures during construction is very rare. However, in the construction of underground caverns it is worth examining this type of option since the simplification of the main contract has significant cost and schedule advantages.

In accordance with underground cavern design procedures, no allowance was made for earthquake loading in the design of the Mingtan underground complex. Hence the loading imposed by the 7.6 magnitude Chi-Chi earthquake of 21 September 1999, with its epicentre at a depth of 7 km about 15 km from the Mingtan site, represented a good test of the validity of this design approach.

Charlwood el al (2000) report that thousands of buildings were damaged, 2,200 people were killed and more than 8,000 were injured in the area surrounding the epicentre. The concrete gravity dam on the Mingtan project was undamaged but, at a penstock river crossing, some components of expansion couplings in the penstocks were deformed due to longitudinal movements. These couplings did not fail, the deformed components were replaced and the penstocks quickly returned to service. The project was in operation at the time of the earthquake and the underground excavations were undamaged, although there was a loss of power and lighting underground. Several people were working in the plant at the time and apparently felt only minor shaking. These observations confirm that deep underground excavations are much less vulnerable to seismic ground motions than those at surface.

7 CONCLUSIONS

The examples presented demonstrate that the increasing demand for long, deep tunnels creates new problems for the construction industry. Because of limited access, it is difficult to apply traditional site investigation techniques so that, in many cases, the amount of information available is very limited and the preparation of detailed designs for differing ground conditions occurring along the tunnels are not practical. This means that engineers and contractors have had to develop approaches that permit the tunnels to be constructed in such a way that geological and geotechnical variations do not play a dominant role in the process.

The Yellow River Diversion tunnels in China and the Guadiaro-Majaceite tunnels in Spain are excellent examples of the use of a tunnelling method, based on double-shield TBMs with simultaneous installation of precast concrete linings. This makes the process largely independent of the geological conditions. To a lesser extent, the Olmos Transandino tunnel in Peru and the last stages of the Yacambú-Quibor tunnel in Venezuela are also examples where single support systems were installed routinely in order to permit the tunnels to be advanced without the need for frequent changes in methodology to deal with differing ground conditions.

One of the most serious impediments to rapid and efficient tunnel construction is the endless tinkering with tunnel support in an attempt to optimize these designs to the ground conditions encountered. This is also one of the main sources of claims and disputes since it is very seldom that the various parties involved will agree on the definition of the geological and geotechnical conditions and the methods that should be used to stabilize the tunnel. In the cases mentioned above, support systems designed to deal with most of the conditions encountered were installed routinely and the field engineers and geologists were not permitted to interfere with this process. Their advice was only sought when exceptional conditions occurred.

I am entirely in agreement with this process and I foresee that, as TBMs continue to develop, the tendency to use lining systems installed simultaneously with the advance of the machine will become more and more common.

In direct contrast to these trends is the increasing sophistication of site investigation and design methods for large underground caverns. These caverns are concentrated in a limited volume of rock and it is justified to devote significant resources to the detailed definition of this rock volume. Exploration adits and test galleries are general constructed to allow detailed geological mapping, in situ stress measurement and deformation modulus testing. Comprehensive geological and geotechnical models are compiled, usually well in advance of the start of construction. This means that excavation sequences and support methods can be prepared and, in some cases such as the Mingtan project in Taiwan, work can be done during preliminary contracts to make the tasks of the main contractor simpler and safer.

Again, I am in complete agreement with this approach and I see no contradiction between this approach and the hands-off approach for driving tunnels where it is difficult or impractical to collect sufficient reliable information.

It would be nice to end this paper with a neat list of recommendations for different types of contract that have been found to work well for differing ground conditions. Unfortunately, having worked on a large number of projects in every conceivable set of ground conditions, I am forced to conclude that the compilation of such a list is not possible. The form of contract adopted on a particular project depends, to a very large extent, on the limitations imposed on the project management by the ultimate owner and by the organisations providing funding for the project. Even when these constraints and limitations do not exist, it is very difficult to decide what type of contract is best suited to a project. In fact, my experience suggests that the success of an underground project has less to do with the type of contract used than it does with both the owner and the contractor having experienced and competent project managers, geologists and engineers who are prepared to discuss technical issues in a logical and non-confrontational way.

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