Progress with the excavation and support of the Mingtan power cavern roof

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Abstract

This paper gives summary details of the geological conditions and support design adopted for the roof of the Mingtan power cavern in Taiwan. The presence of 35 degree dipping faults in the roof and hunches of the cavern led to the adoption of pre-reinforcement of the roof and pre-treatment of the faults in advance of general cavern excavation. The response of the roof to excavation is described as revealed by visual observation and instrumental data, and summary details are given of computer modelling carried out to back analyse the behaviour of the roof.

Introduction

The 1600 MW Mingtan Pumped Storage Project is currently under construction in central Taiwan. The scheme is being constructed for the Taiwan Power Company by RSEA, which is the largest publicly owned construction company in Taiwan. It will be Taipei's second pumped storage project, following the commissioning of the 1000 MW Minghu project in 1985. Detailed design of both these schemes was carried out by Sinotech Engineering Consultants on behalf of Taipower. Golder Associates have provided assistance to Sinotech with rock engineering aspects of the design and construction of the Mingtan project.

The site has two characteristics which make it ideally suited for a pumped storage project. These are the presence of the existing Sun Moon Lake, which will be utilized for the upper reservoir of the scheme, and the availability of a static head differential of more than 400 m at the level of the Shuili river, at a distance of 4 KM from the lake. Sun Moon Lake is of very large volume and has been utilized as a source of hydro electric energy for over 40 years. It already serves as the upper reservoir for the Minghu project. A concrete gravity dam is under construction in the Shuili valley to form the Mintan lower reservoir about 3 KM downstream from that of the Minghu project. Adjacent to the lower reservoir, a 22 m span x 170 m long x 19 m high underground transformer hall are being provided.

The least favourable aspect of the site is the geological condition of the rocks in the vicinity of the major caverns. Consideration of these conditions led to the adoption of pre-support and pre-treatment measures around the roofs of the power cavern and transformer hall in advance of excavation. The performance of these measures during excavation to the end of August 1989 will be described in this paper.

Power cavern excavation will be completed in early 1990, following a 2 year construction period. The first turbine is expected to be commissioned in 1992. At the end of August, 1989, cavern excavation was 65% complete.

Geology

The geomorphology of Taiwan is heavily influenced by tectonic activity. The 140 KM wide island of Taiwan is situated on the eastern edge of the Eurasian continental plate. This butts up against the Philippines Sea plate near the east coast of Taiwan. Movements on this boundary have given rise to the formation of a range of 3,000 metre high mountains in the centre and on the eastern side of the island. The major lineation of the mountains in central Taiwan is North/South and there is an associated series of steeply dipping, North/South striking thrust faults. Aerial photography indicates that these run the entire length of the island.

One of these faults lies to the west of the Shiulili river. Its exact location has not been identified, due to the rough nature of the terrain. However, its presence has given rise to a high level of disturbance to the rocks in the area of the underground power cavern. These belong to the Waichecheng Sandtone formation, which forms a syncline between Sun Moon Lake in the east and the Shuili river in the west.

During Basic Design Revision studies for Mingtan, a 2 metre x 2 metre exploratory gallery was excavated along the longitudinal axis of the power cavern with its invert 10m above the proposed elevation of the crown. This gallery was intended to serve as a drainage gallery during the operating life of the project. Observations in the gallery revealed that the majority of the cavern would be excavated in a slightly to moderately weathered, frequently jointed, medium to very thickly bedded, strong, coarse grained quartzitic sandstone, exhibiting a low grade of metamorphism. The beds dip at 35 degrees to the south east, and strike directly across the cavern, which was oriented specifically to obtain this relationship with the bedding.

At several locations in the cavern roof, the presence of weaker beds of siltstone up to 2 metres thick appeared to have caused a concentration of shear movements during tectonic activity, so that fault zones had developed parallel to the bedding. The common feature observed for all these faults was the presence of continuous clay filled bedding planes, with the thickness of clay varying from a few millimetres to 20 centimetres. The thickness of the individual seams was variable between these limits. The clay could be more correctly described as a gouge, varying from plastic clay to a mixture of clay and platy or irregular intact rock fragments. For most of the faults, 2 such clay seams were separated by a layer of sandstone or siltstone of variable condition. Frequently, these were shattered and intact rock varied from strong to weak as a result of weathering or alteration effects. The spacing of the clay seams varied from 29 to 100 centimetres for the largest fault, identified as fault H/H1, the overall width of the fault was 5 metres and several intermediate clay planes were observed. The rock between these planes was variable, being highly altered and very weak in places, but strong and intact in others. For all of the faults, the rocks in immediate contact with the clay on the outer contacts of the fault were strong. Some of the faults appeared to be dry, but others were damp or dripping to the extent that softened material had slid out of the roof of the gallery along the bottom bedding plane of the fault. The location of the faults in relation to the cavern centreline is shown in Figure 1.

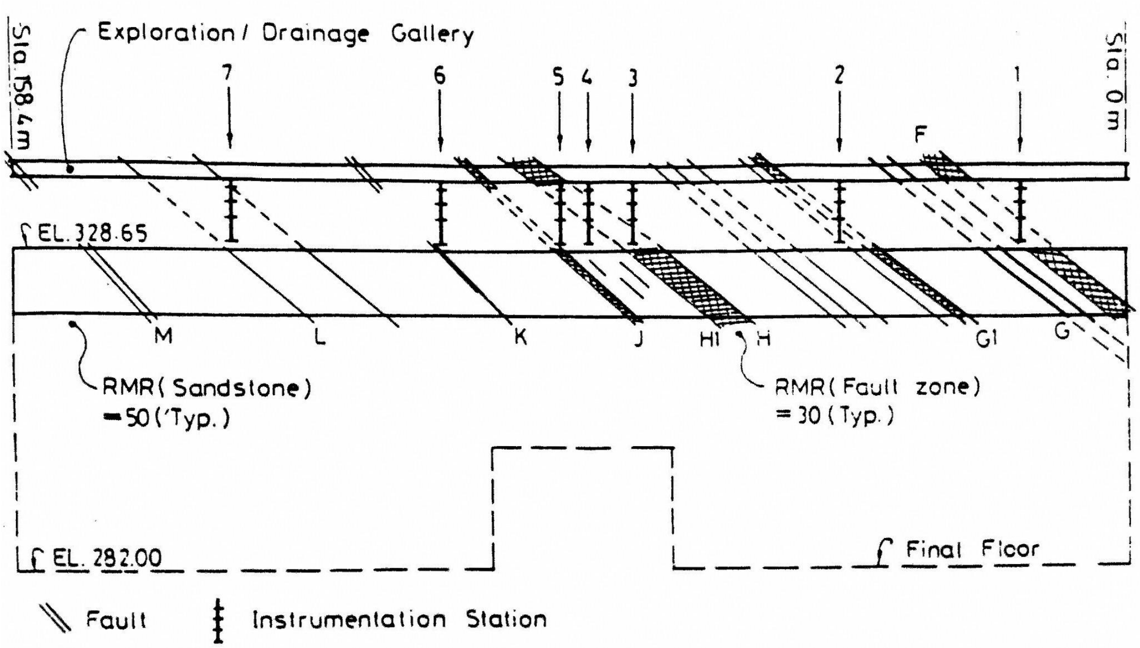

Figure 1. Longitudinal section of power cavern showing encountered geology.

Outside the fault zones, individual bedding planes with the sandstone units sometimes exhibited thin clay seams, and areas of altered and weakened rock could be observed. Jointing within the sandstone was orthogonal to the bedding , as shown in Figure 2. The persistence of jointing was generally low, of the order of 0.5 to 1 metre. Adjacent to the faults, however, the persistence of the K1 joints appeared to increase to more than 2 metres. The joints were generally tight, rough and irregular.

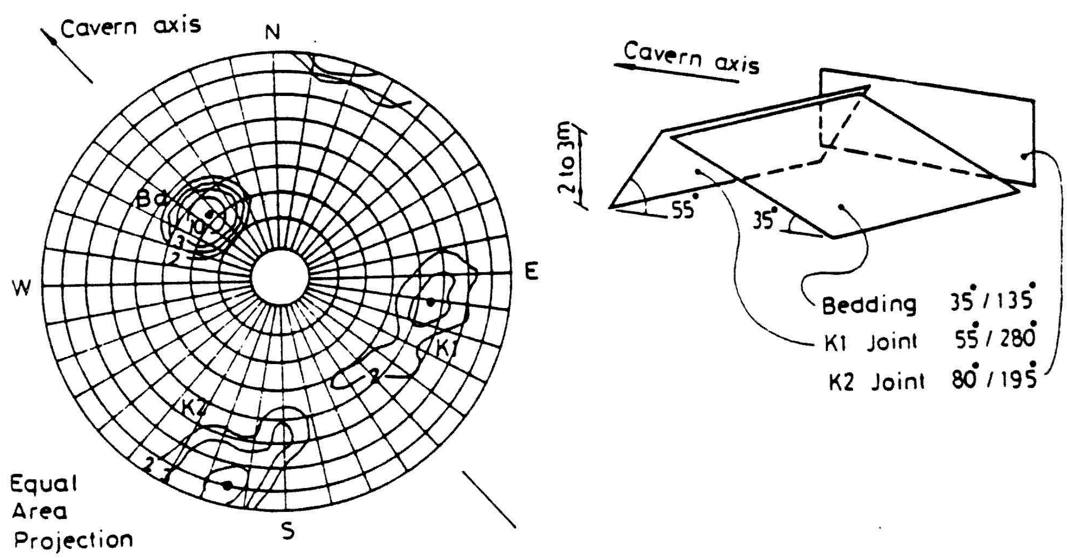

Figure 2. Jointing and wedge formation in cavern roof.

Design considerations

Full details of the cavern design have been given by Liu, Cheng and Chang (1988). For clarity, some aspects of the design of the roof supports are repeated here. The major design concern related to the anticipated behaviour of the fault zones. Firstly, these effects were considered likely to be concentrated on the weak clay filled planes. Secondly, the soft fault zones could reduce the stiffness of the rock mass and lead to a loss of confinement of the near surface rock. Thirdly, wedges of rock up to 2 or 3m high could be formed in the immediate roof and haunch through the interaction of the fault planes with the K1 and K2 joints, which tended to be more continuous adjacent to the faults. These possibilities suggested that, adjacent to the fault zones, the temporary support of the roof and haunches during excavation could be problematical.

At Minghu, excavation had taken place using temporary rock bolt support, and a mass concrete arch had been constructed prior to benching. At Mingtan, it was considered that this traditional method of cavern construction might be less appropriate than the use of a flexible option. Access and drainage galleries had already been excavated in the cavern area and there was a long lead in time to the commencement of the main underground excavation contract. Thus, the opportunity existed to carry out works designed to pre-treat the fault zones and pre-reinforce the rock mass around them in advance of excavation. This would provide security during the initial excavation phase and allow decisions relating to further supports to be made as excavation proceeded.

For this purpose, provision could be made of the installation of rockbolts or anchors during excavation and the application of a steel fibre reinforced, micro silica shotcrete lining. The thickness of this lining could be variable, depending upon the conditions encountered.

Pre-Treatment and Pre-Reinforcement Works

Longitudinal, 2 metre x 2 metre galleries were excavated along the entire length of the cavern at about the mid point of each haunch. The crown of each gallery coincided with the final profile of the haunch in this area. Where these galleries crossed the major faults, short cross cuts were excavated along the strike of the faults, ie. across the power cavern, immediately outside the final profile of the roof. See Figure 3. From the access provide by the crosscuts, clay was washed out of the faults to a depth of approximately 4 metres using a high pressure jetting system developed for the treatment of similar faults in the abutments of the Feitsui dam in northern Taiwan. See Cheng (1987) and Fu, Lee and Lee (1987). Following the removal of the clay (working in 3m panels), non shrink cementitious mortar was pumped back in to the void. Subsequently, the crosscuts were backfilled with mass concrete. No problems were encountered with the stability of the hanging wall of the faults during this operation.

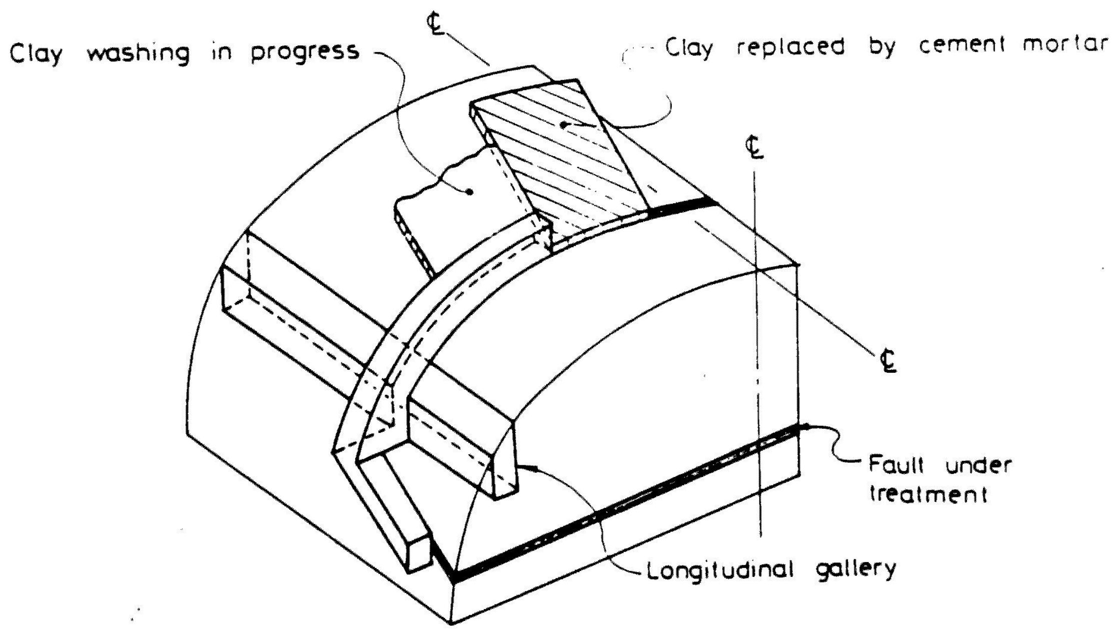

Figure 3. Schematic layout of fault treatment works

Following the pre-treatment work, the roof and haunches were pre-reinforced with corrosion protected cable anchors installed downwards from the exploratory/drainage gallery above the central crown and upwards from the longitudinal galleries in the haunches. See Figure 4. It was decided to extend this treatment over the entire cavern roof, instead of limiting it to the vicinity of the faults, as this would optimize the efficiency of the reinforcement and lead to thin savings on support works during the main phase of excavation.

Over the majority of the roof, anchors of 50 Tonne capacity were installed at 2 metre centre. It is known that this distribution of anchors would allow the dead weight support of 2.5m high blocks or wedges delineated by bedding planes and K1 joints, with a factor of safety of 2. In addition, resistance would be offered to shear displacement along fault or bedding planes and general loosening of the rock mass. In one area of better than average conditions, 25 tonne anchors were used at the same 2 metre x 2 metre spacing.

Although the anchors were fully grouted, it was decided to leave then untensioned, except for a nominal load of a few tonnes to straighten them in the hole, as they would develop load during excavation and might become overstressed if a high tension was induced at the time of installation. Several anchors were selected for the installation of load cells, which were fitted to the top of the cables in the drainage gallery. These were grouted over a 3 metre length adjacent to the cavern roof only, so that they could record the average response of the anchor to any deformation.

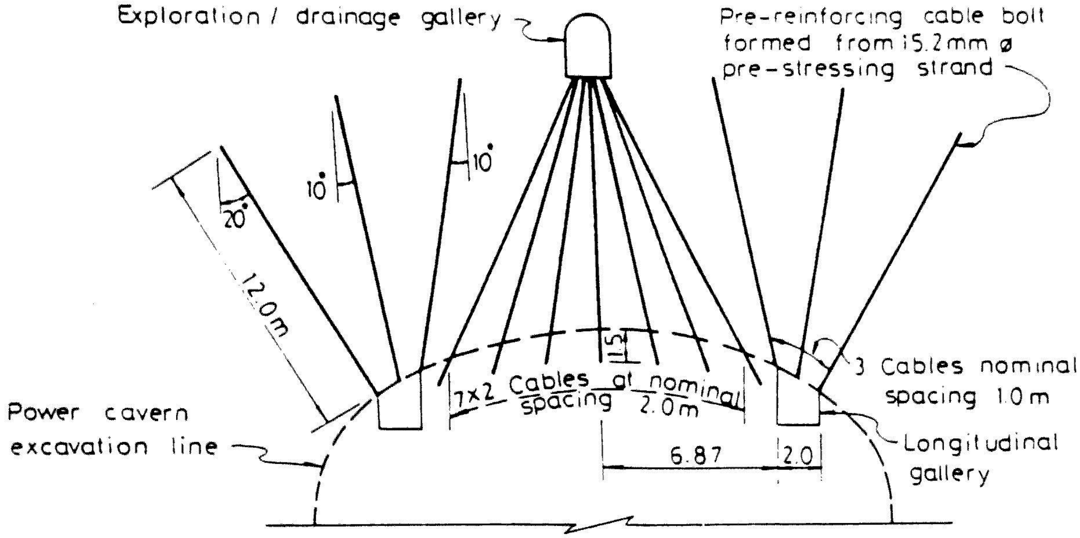

Figure 4. Typical arrangement of pre-reinforcement cables

During the implementation of the pre-treatment works, insufficient time existed to pre-treat all of the major faults, so the treatment was omitted for Fault F adjacent to the southern wall of the cavern. In this area, additional pre-reinforcement was provided to reduce the overall spacing to approximately 1.5 metres. At fault H/H1, washing of the seams was not implemented, as the condition of the fault was poor over its entire width. Instead, the crosscuts were extended right across the roof and haunches at about 3 metre x 3 metre section, so that a complete, mass concrete rib could be constructed across the roof. The same pre-reinforcement pattern was used at fault H/H1 as was provided for the untreated fault F.

Similar measures were taken in the smaller transformer hall, with a single longitudinal gallery in the central crown for pre-treatment work, and the installation of pre-reinforcement from a drainage gallery 8 metres above the crown.

Extensometers were installed in the drainage gallery and also in the longitudinal adits in the haunches, so that the movements resulting from excavation could be monitored form the commencement of excavation. See Figures 1 and 5.

Response to excavation of crown haunches

Excavation of the power cavern commenced in December 1987, with a 6 metre x 6 metre top heading in the central crown. This was advanced from the south wall and passed underneath the untreated but heavily pre-supported Fault F after 15 metres of excavation. At this location, a passing bay was slashed out on one side of the hading so that the total excavated span was of the order of 12 metres.

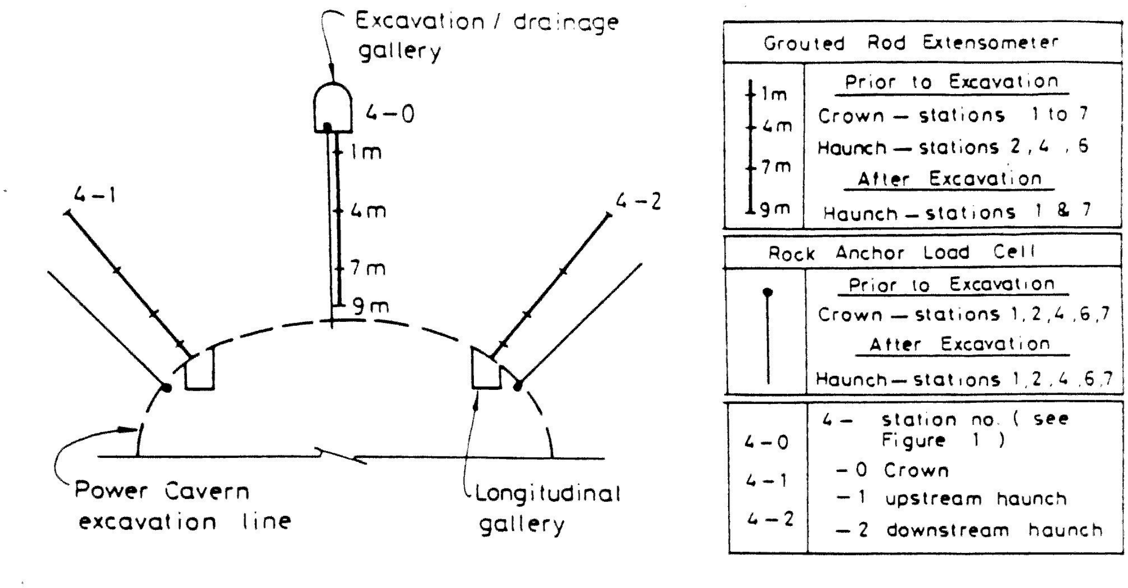

Figure 5. Instrumentation layout

The roof responded rapidly to this initial phase of excavation. During blasting, the immediate collapse of untreated fault material left a void over 1.5 metres deep in the roof of the passing bay. In the next 6 days, a rapid downward movement of over 40 millimetres was recorded by extensometer 1-0 as the excavation of the heading progressed to station 24 metres. Subsequently the rate of movement slowed and died away after a total dilation of 57 millimetres had occurred between the drainage gallery and the roof of the heading. See Figure 6. This movement was accompanied by a build up in load in the adjacent load cell to approximately 16 tonnes, and by the observation of cracking in a concrete lined section of the drainage gallery immediately above Fault F. This cracking was consistent with overstressing of the concrete induced by horizontal stress, with cracks opening in the middle of the walls and small pieces of concrete bursting of the reinforcement in the haunches.

Rapid movements were recorded by the other pre-installed extensometers as the heading passed underneath them, but significantly less movement occurred than that recorded at Fault F. The majority of the movement occurred close to the cavern roof. Some of the load cells on the rock anchors indicated lower loads than would be expected from the extensometer data, despite their close proximity. One possible explanation of this is that slippage occurred at the cable/grout or grout/rock interface for these partly grouted cables, despite the fact that face plates were fitted to the exposed ends of all the pre-reinforcement cables during excavation.

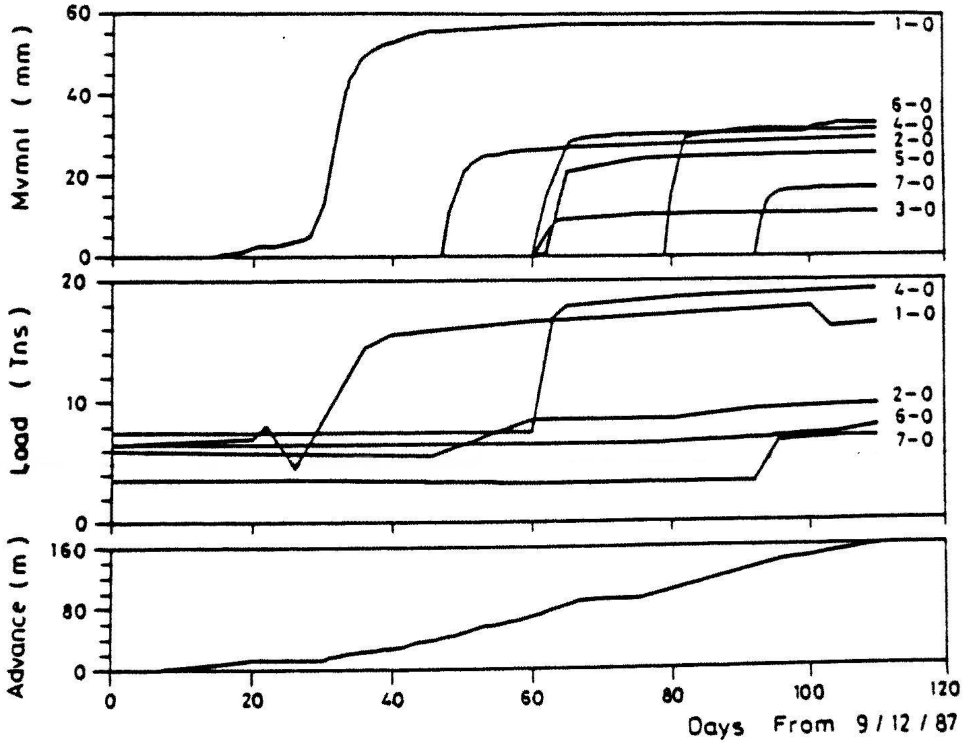

Figure 6. Instrumentation data, top heading excavation

Despite the pre-reinforcement, an irregular profile was produced with the roof tending to follow the profile of the bedding planes and K1 joints. In contrast to the overbreak occurring at Fault F, generally good profiles were obtained in the pre-treated fault areas. In some areas, 5 metre rockbolts were placed intermediate to the pre-reinforcement cables during excavation. A layer of fibre reinforced shotcrete was applied all over the roof.

Following the completion of top heading excavation, slashing of the haunches commenced immediately. The sequence of excavation involved several faces advancing from six initial slashes. By comparison with the top heading, this produced a much smaller response for extensometer 1-0, but a similar or larger response for the rest of the crown instruments. See Figure 7. Detailed analysis of the crown extensometer data indicated that, during haunch excavation, a significant proportion of the rock mass dilation occurred between pairs of extensometer anchors which straddled a fault plane. This was particularly notable of the top plane of Fault H, which was intersected by 2 extensometers exhibiting this trend, and for which approximately 5 mm of dilation could be seen in exposure on the walls of the drainage gallery, which was unlined in this area. At this location, small signs of shear movement along the plane were apparent, which may have given rise to the dilation of the plane. Evidence of differential (ie shear) movements along fault planes was also noted inside the cavern, as shotcrete cracking occurred where faults G1 and J daylighted in the crown.

By the end of haunch excavation, smaller movements had occurred adjacent to all of the fault treatment areas, (particularly fault H/H1) than at the untreated Fault F.

The overall response of the haunch extensometers is shown in Figure 8. Two instruments gave unreliable readings. The general behaviour was consistent with that described above for the crown. The transformer hall also behaved in a similar manner, but with movements of reduced magnitude consistent with its smaller size.

Numerical analysis of crown and haunch excavation

On the basis of the instrumentation data, an attempt was made to back analyse the behaviour of the cavern up to the end of haunch excavation, in order to allow prediction of the final stresses and displacement. This analysis was performed with the 2 dimensional finite difference program FLAC, distributed by the Itasca Consulting Group, Inc. of Minneapolis. FLAC allows the modelling of elasto-plastic, non-linear, Mohr-Coulomb material behaviour, and the incorporation of support elements to which strength and bond stiffness parameters applicable to rock anchors can be applied.

It was realised from the outset that this process could only yield approximate results, as the response of the crown and haunches to excavation was evidently a 3 dimensional process involving differential movements down the dip of the fault planes. Nevertheless, it was considered that the exercise would provide useful input into the process of engineering appraisal of the response of the cavern to future excavation.

The section chosen for modelling corresponded to the model used for finite element studies during the design phase. See Liu, Cheng and Chang (1988). This incorporated a fault zone in the roof of the cavern at an elevation consistent with the situation of most of the crown extensometers. Mohr-Coulomb friction and cohesion values for the sandstone surrounding the fault zone were selected from a tangent to a non-linear Hoek-Brown envelope, based on m and s values chosen from intact core tests factored down by reference to RMR and Q values.

Young's modulus was selected by factoring down intact laboratory values in accordance with RMR values, but also bearing in mind the results of plate bearing tests carried out prior to the design studies. The fault zone was modelled as an equivalent material with strength of the clay layers on the top and bottom surface of each fault.

From initial runs, it became apparent that, in order to match the extensometer results, the strength and modulus of the faults would have to be significantly lower than that of the sandstone and that these parameters would have a large influence on the displacements predicted by the model. In addition, the large movements exhibited by the anchors closest to the excavation surface suggested that the strength and modulus of the sandstone in this area should also be factored down to take account of the loosening induced by the excavation process. These considerations led to the performance of a series of analyses in which the properties were adjusted by trial and error. In the final analysis, the cohesion and friction angle assigned to the mass of the sandstone were lower than the values originally selected from the Hoek-Brown envelope, by 200 kilopascals and 5 degrees. In the disturbed, near surface zone, reductions of the cohesion value were made to match the instrumentation data. The final parameters here were a cohesion comparable to that which could be obtained from m and s values suggested by Hoek and Brown for a disturbed rock mass (1988), but a friction angle 5 degrees higher. Details of the final analysis are given in Figure 9.

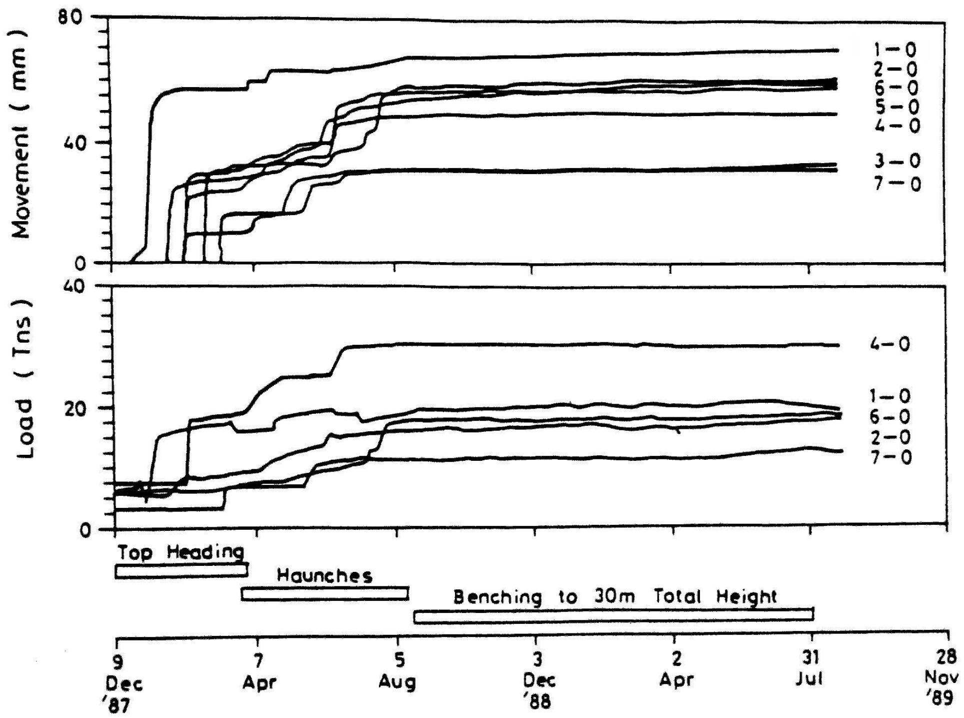

Figure 7. Crown instrumentation data to August 1989

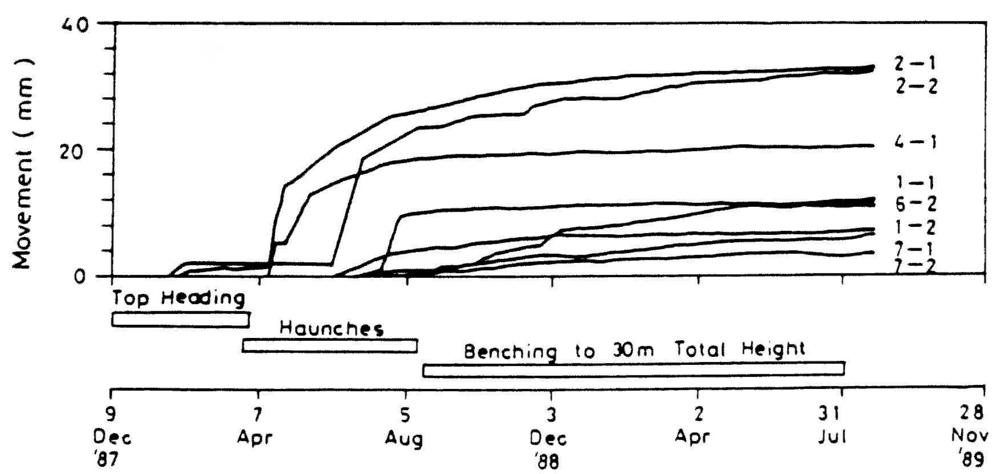

Figure 8. Haunch extensometer data to August 1989

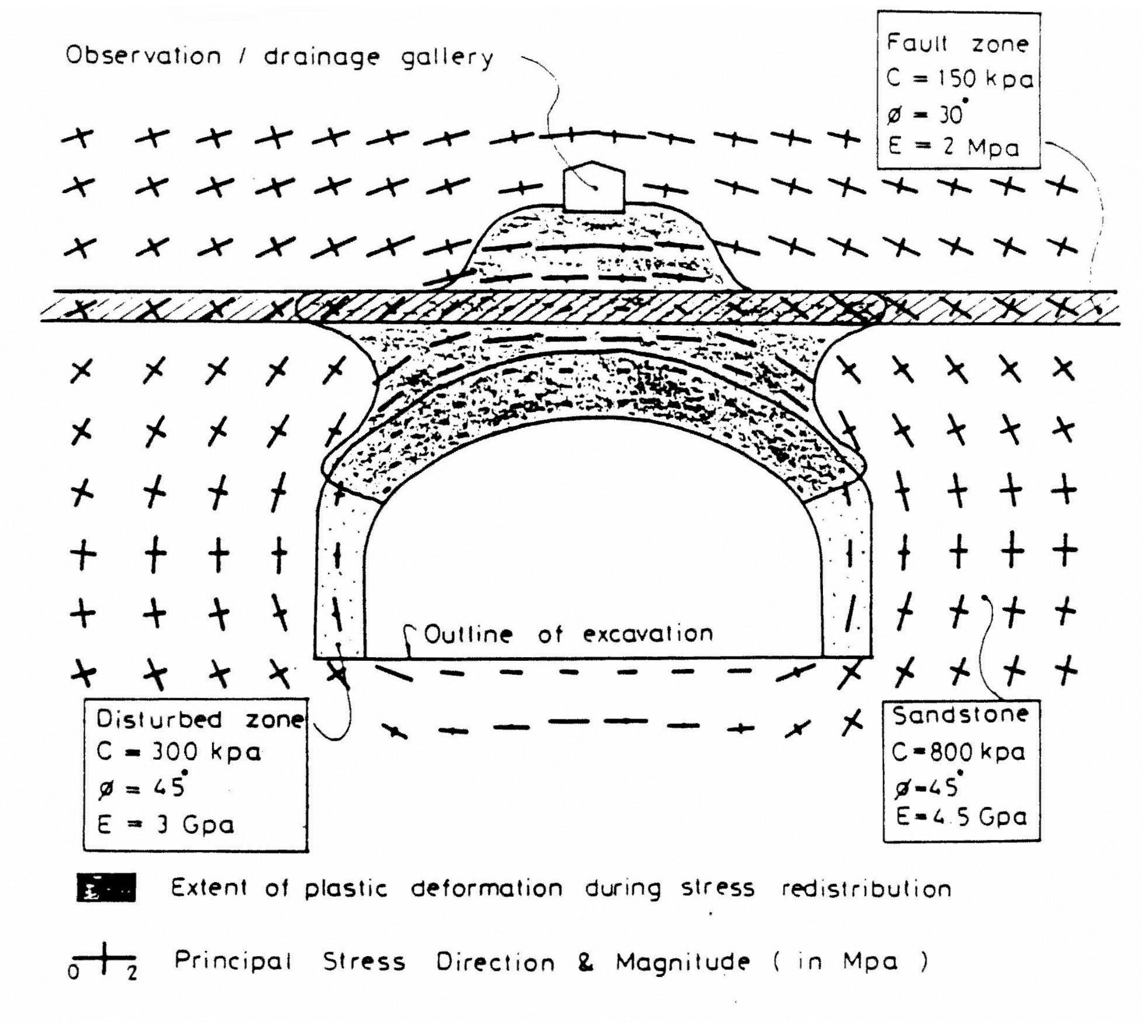

Figure 9. Input/output data for FLAC analysis

Predicted cable anchor loads in the crown and haunches were significantly higher than those monitored underground, possibly because of the limitations of 2 dimensional modelling for the Mingtan situation, (ie. the 35 degree dipping faults were represented by a horizontal layer in the model).

Supports installed prior to benching

Although central crown cable loads were satisfactory at the end of haunch excavation, the deformation which had accompanied excavation up to that point gave rise to concern about the likely magnitude of further deformations during benching. Consequently, the decision was taken to install additional cable anchors adjacent to most of the faults prior to benching. This was implemented over about 30% of the roof. Typically, these were 131 tonne capacity anchors tensioned up to 60 tonnes, and varying in length from 8 to 12 metres. These were installed radially in the middle or upper haunch so that they were anchored above the top plane of the fault. No additional supports were provided in the transformer hall. The initial thin layer of the fibre reinforced shotcrete applied during crown and haunch development was built up to a minimum thickness of 150 mm prior to benching.

Response to benching

The response of the central crown to benching was in marked contrast to its earlier behaviour. With the exception of minor surface dilation in the vicinity of Faults F and G1, little or no dilation of the reinforced zone occurred as the height of the cavern increased to 30 metres, this being approximately 65% of its final size. This suggests that the movements occurring during crown and benching occurred at greater depth. This is consistent with the FALC results shown in Figure 9, which indicate low principal stress levels above the crown and plastic yielding occurring over a large area. The response to benching indicates that the additional supports installed prior to benching may not have been necessary. Some movement has been observed in the haunches but these have shown a stabilizing trend.

Careful observations of the condition of the roof has been maintained as benching has progressed, access being possible via a platform on top of a temporary crane erected by the contractor. The contractor was encouraged to install this crane at an early stage of excavation to allow continued access to the roof during benching. Isolated signs of small, differential shear movements have been observed on faults and bedding planes, both in the roof and the drainage gallery, but no problems are anticipated during outstanding excavation.

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