6

The Rio Grande project - Argentina

6.1   Introduction

The Rio Grande pumped storage project is located on the Rio Grande river near the town of Santa Rosa de Calamucita in the Province of Cordoba in Argentina. It has an installed capacity of 1000 MW and provides electrical storage facilities for the power grid and, in particular, for a nuclear power plant about 50 km away from Rio Grande.

The project is owned by Agua y Energia Electrica, the principal Argentinean electrical utility organisation. Preliminary feasibility studies were carried out by the owner and these were followed by detailed design studies by Studio G. Pietrangeli of Rome. The scheme was partly financed by Italy and some of the construction was done by Condote de Agua, an Italian contractor. Golder Associates were involved in the design and supervision of support installed to control the stability of most of the major underground excavations.

The main underground facilities are located in massive gneiss of very good quality. The upper reservoir is impounded behind a rockfill dam and water is fed directly from the intakes down twin penstocks which then bifurcate to feed into the four pump-turbines. These turbines, together with valves and the control equipment, are housed in a large underground cavern with a span of 25 m and a height of 44 m.

Draft tubes from the turbines feed into twin tunnels which, with a down-stream surge shaft, form the surge control system for this project. The twin tunnels join just downstream of the surge tank and discharge into a single tailrace tunnel with a span of 12 m and height of 18 m. This tailrace tunnel is about 6 km long and was constructed by a full- face drill-and-blast top heading, with a span of 12 m and height of 8m, followed by a 10 m benching operation. A view of the top heading is given in Figure 6.1.

6.2   Tailrace tunnel support

 Because of the excellent quality of the gneiss, most of the underground excavations did not require support and minimal provision for support was made in the contract documents. Assessment of underground stability and installation of support, where required, was done on a 'design-as-you-go' basis which proved to be very effective and economical. Recent reports from site, many years after the start of construction and commissioning of the plant, show that there have been no problems with rockfalls or underground instability.

Figure 6.1: The 12 m span 8 m high top heading for the tailrace tunnel was constructed by full-face drill-and-blast and, because of the excellent quality of the massive gneiss, was largely unsupported.
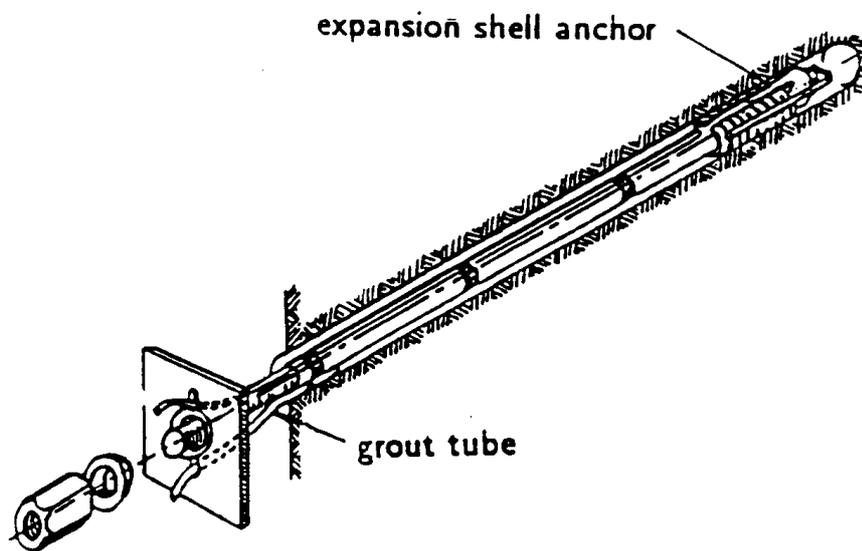

Figure 6.2: Mechanically anchored rockbolts of the type used on the Rio Grande project. These bolts were tensioned to 70% of their yield load upon installation and then, at a later stage, were re-tensioned and fully grouted.
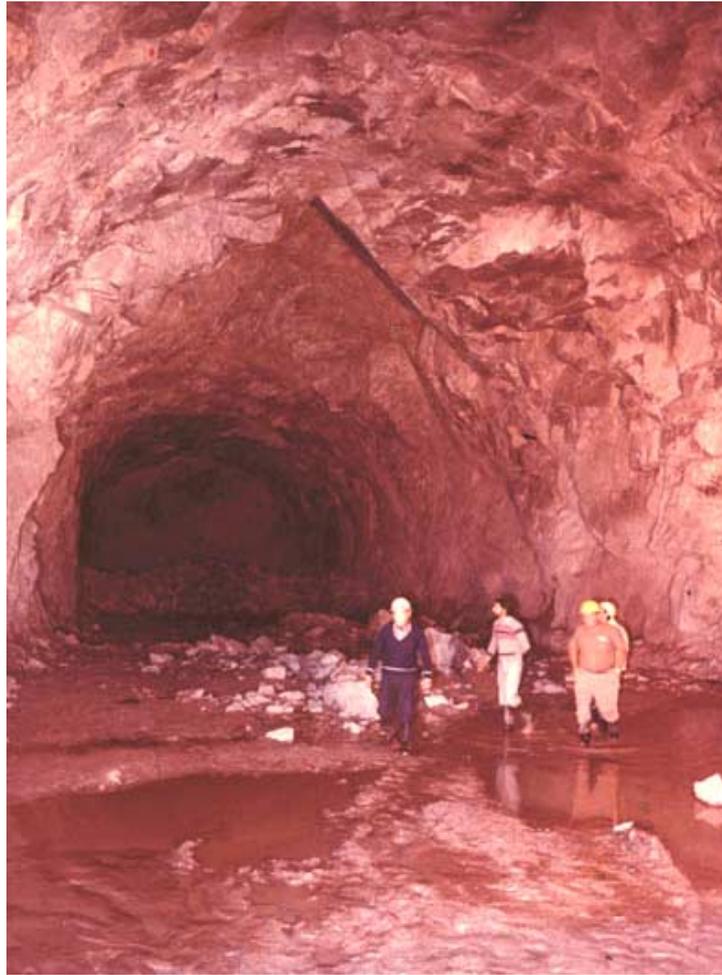
Figure 6.3: A wedge failure in the roof of the top heading of
the Rio Grande tailrace tunnel.

Decisions on support were made on the basis of inspection of the excavated faces by a resident team of geotechnical engineers. Where the appearance of the face indicated that a zone of heavily jointed rock, usually associated with faulting, was being entered, the top heading was reduced to a 6 m span by 8 m high pilot tunnel to limit the volume of unstable rock which could be released from the roof. This pilot tunnel was large enough to accommodate the seven-boom jumbo, as illustrated in Figure 6.4, but small enough to limit the size of roof falls to manageable proportions. Bolting from inside the pilot heading was used to pre-support the potentially unstable wedges and blocks in the roof.

In the case of the tailrace tunnel, which is itself a large excavation, the support comprised mechanically anchored and cement grouted rockbolts as illustrated in Figure 6.2, with mesh reinforced shotcrete where required. These bolts were generally installed to control the type of wedge failure illustrated in Figure 6.3. In the case of particularly large wedges, calculations of the factor of safety and support requirements were carried out on a programmable calculator, using an early version of the program UNWEDGE.

Figure 6.4: A 6 m wide heading driven ahead of the tunnel face to permit pre-reinforcement of potentially unstable wedges in the roof. The seven-boom jumbo is seen working in the heading.

6.3   Support for power cavern

A cross-section of the power cavern is given in Figure 6.5 and this figure includes the five main excavation stages for the cavern. Careful mapping of significant structural features in the roof and walls of the central access drive at the top of the cavern provided information for estimating potentially unstable blocks and wedges which could form in the roof of the cavern. Figure 6.6 illustrates a number of such wedges in one section of the cavern roof. At each stage of the cavern excavation, long rockbolts (up to 10 m length) were installed to stabilise wedges or blocks which had been determined as being potentially unstable.

Because gneiss has usually undergone some tectonic deformation during its geological history, projection of structural features from visible exposures tends to be an imprecise process. Consequently, the potentially unstable blocks and wedges had to be reassessed after each excavation step revealed new information. The structural plan illustrated in Figure 6.6 had to be modified many times during excavation and that shown is the final plan prepared after the full cavern roof had been exposed.

A general view of the cavern excavation is given in Figure 6.7. This photograph was taken when the bulk of the cavern had been completed and only a few benches in the bottom of the cavern remained to be excavated. The enlarged top of the cavern is to accommodate the overhanging crane that is supported on columns from the cavern floor. An alternative design for this cavern would have been to support the crane on concrete beams anchored to the walls as is commonly done in good quality rock.
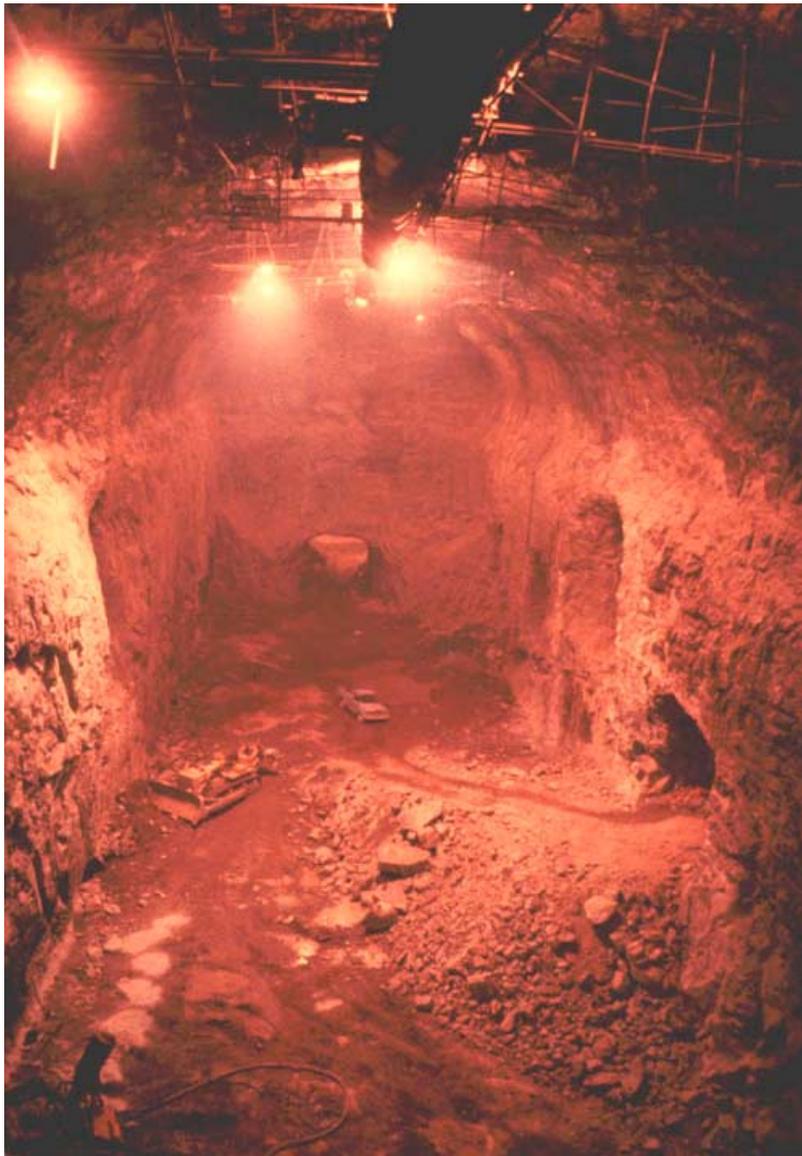
Figure 6.5: Cavern profile and excavation stages.

Figure 6.6: A plan of the traces of geological features mapped in part of the cavern roof. The shaded areas represent potentially unstable wedges requiring reinforcement.

Figure 6.7: A view of the 25 m span Rio Grande power
cavern during excavation of the lower benches.

6.4   Discussion of support design and costs

Apart from rockbolts installed to control isolated structurally controlled blocks and
wedges in the roof and sidewalls and some areas of closely jointed rock which were
shotcreted, the cavern was unsupported. While this was successful for this particular
project, it is not the approach which should generally be used for a critical excavation
such as an underground powerhouse.

The damage resulting from even a small rockfall in such a cavern is out of all
proportion to the savings achieved by eliminating pattern rockbolting and full
shotcrete lining. Hence, in addition to the rockbolts installed to control structural
instability, as described earlier, I would recommend a normal pattern of 25 mm
diameter, 5 m long bolts (20% of the excavation span) on a 2.5 m grid. In addition, I
would recommend the placement of 50 mm of fibre-reinforced micro-silica shotcrete

over the entire roof and upper sidewalls of the cavern. Based on current north American costs, this additional support, involving approximately 600 rockbolts and about 300 m^3 of shotcrete, would have cost approximately US $200,000. In terms of the overall project cost and the increased long-term security in the cavern, this would normally be regarded as a good investment.

In contrast, consider the 6 km long tailrace tunnel in which the consequences of a small rockfall are minimal. Assume that a pattern of 4 m long bolts on a 2 m grid (say 10 bolts per section) and a 50 mm shotcrete thickness had been specified for the roof and upper sidewalls of the tailrace tunnel. This would involve 30,000 bolts and 5,400 m^3 of shotcrete at a total cost approaching US $5 million. This example illustrates the need to give careful consideration to the function and risks associated with each underground excavation before deciding upon the support system to be used.

6.5   Analysis using UNWEDGE program

UNWEDGE, described in the previous chapter, is a user-friendly micro-computer program which can be used to analyse the geometry and the stability of wedges defined by intersecting structural discontinuities in the rock mass surrounding an underground excavation. The analysis is based upon the assumption that the wedges, defined by three intersecting discontinuities, are subjected to gravitational loading only. In other words, the stress field in the rock mass surrounding the excavation is not taken into account. While this assumption leads to some inaccuracy in the analysis, it generally leads to a lower factor of safety than that which would occur if the in situ stresses were taken into account.

The application of the program UNWEDGE to the analysis of a potentially unstable wedge in the Rio Grande cavern is illustrated in the following discussion.

6.5.1 Input Data

The dips and dip directions of a number of planes (maximum 20) can be entered directly into the pop-up table which appears when the 'Input data' option is chosen or this information can be entered in the form of a DIPS file. Once the data has been read into the program, the great circles representing the discontinuities are displayed on the screen as illustrated in Figure 6.8 and the user is prompted to select the three joint planes to be included in the analysis. Once the information on these planes has been entered, the unit weight of the rock and the shear strengths of the joints are entered. Finally, the water pressure acting on the joint surface is entered. In most cases, the default water pressure of 0 will be chosen but the user may check the sensitivity of the wedge to pore water pressure by entering appropriate values.

In the case of the rock mass surrounding the Rio Grande Cavern, the dips and dip directions of the following four sets of joints are included in Figure 6.8:

> 1 50/131   infrequently occurring joints
> 2 88/225   shear joint set
> 3 85/264   shear joint set
> 4 50/345   tension joint set

Joints 2, 3 and 4 form the central wedge illustrated in Figure 6.6 and these joints have been included in the analysis which follows.
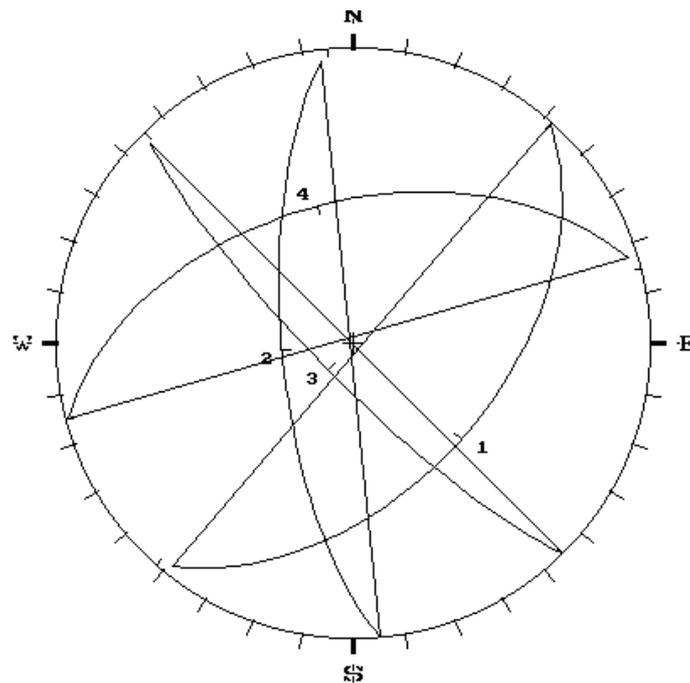
Figure 6.8: Great circles representing four joint sets which
occur in the rock mass surrounding the Rio Grande cavern -
imported as a DIPS file.

6.5.2 Input of excavation cross-section

In setting up this analysis, the co-ordinates shown in Figure 6.9 were used to define
the cavern profile. These co-ordinates must be entered sequentially and must form a
closed figure. The profile is formed from straight line and arc segments and a
sufficient number of co-ordinates should be entered to ensure that a smooth profile is
generated.

6.5.3 Determination of wedge geometry

Depending upon the shape of the cross-section, a maximum of six wedges can be
formed with three intersecting joint planes. Selecting the 'View Wedges' option
initiates the calculation which determines the shape and size of these wedges. The
two wedges formed on the cavern end walls can be viewed by activating the 'View
Ends' option.

   Figure 6.10 shows the wedges formed in the case of the Rio Grande power cavern
for the three joint planes 2, 3 and 4 defined in Figure 6.8. The right hand bar contains
information on the weight of each of these wedges, the failure mode and the
calculated factor of safety. Obviously, the most dangerous wedge in this situation is
the wedge formed in the roof while the wedge formed in the floor is stable and need
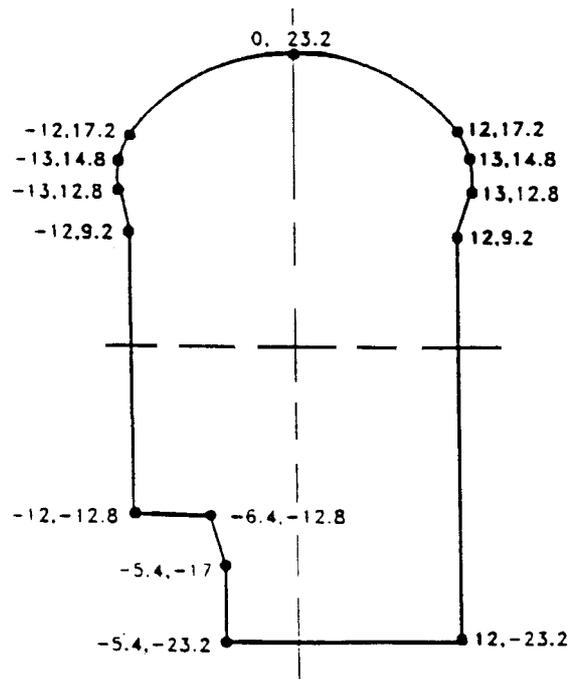not be considered further in this analysis.

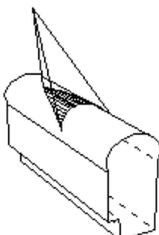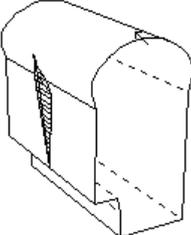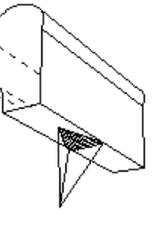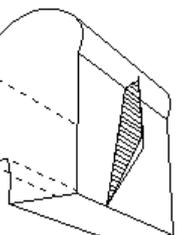Figure 6.9: Co-ordinates used to define the profile of the cavern.

Figure 6.10: Perspective view of the wedges formed in the rock mass surrounding the Rio Grande power cavern.

6.5.4 Installation and analysis of rockbolts

The program UNWEDGE automatically determined the largest wedge which can occur in the rock mass adjacent to the excavation profile. In the case of the roof wedge, shown in Figure 6.10, the wedge extends over the full 25 m span of the cavern and weighs 11,610 tonnes. While, in exceptional circumstances, such wedges may occur, the limited extent of joints in many rock masses will restrict the size of the wedges to much smaller dimensions than those determined by UNWEDGE for the large excavations. As illustrated in Figure 6.6, the span of the central roof wedge (measured normal to the cavern axis) is about 5.5 m and the trace length of joint number 4 (50/345) is approximately 10 m.

When one of the wedges is selected for more detailed analysis a 'Size' option is displayed and this allows the user to define the size of the wedge in terms of the area of the face on the excavation surface, the volume of the wedge, the height of the apex of the wedge or the length of one of the joint traces. In this case a trace length of 10 m is entered for the joint defined by 50/345 and the resulting wedge is illustrated in Figure 6.11. This wedge weighs 255 tonnes and will require about seven 50 tonne capacity fully grouted cables to give a factor of safety of about 1.5 which is considered appropriate for a cavern of this type.
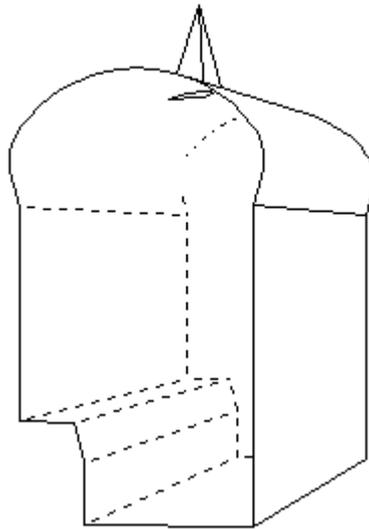

Figure 6.11: Perspective view of roof wedge in the Rig Grande cavern roof. The size of this wedge has been defined by setting the trace length of the 50/345 joint to 10 m.

UNWEDGE allows the user to add a layer of shotcrete and calculates the factor of safety increase as a result of such an addition. Since the shotcrete can only be added once the surface of the wedge is fully exposed it is not taken into account in calculating the support required to stabilise the wedge. The increase in safety factor which occurs after the shotcrete has set can be regarded as a long term bonus and it does allow the user to choose a slightly lower factor of safety for the immediate support of the wedge.