I.S.R.M. Commission
on Rock Failure Mechanisms
in Underground Openings.
Greek working subgroup.

CASE HISTORIES FROM GREECE

A.I. Sofianos, Member of the Commission, coordinator
K. Karapantelakis, Geologist
N. Koronakis, Mining Engineer
D. Solomos, Mining Engineer

Abstract : Six underground projects encompassing mines, hydroschemes and highway tunnels are investigated as far as rock failure during construction is concerned. Any modes of failure are drawn and the practical measures for overcomming the difficulties are presented.

1. Introduction

Case histories of rock failures in Underground Openings have been collected by a relevant greek working subgroup. They have been collected from two underground mines i.e. the Skoumtsa Chromium and the Parnassos - Giona Bauxite Mines, from two tunnels of the Aoos springs underground Hydroelectric scheme and from two main highway tunnels i.e. the Artemissio and the Metsovo tunnel constructed during the last four years.

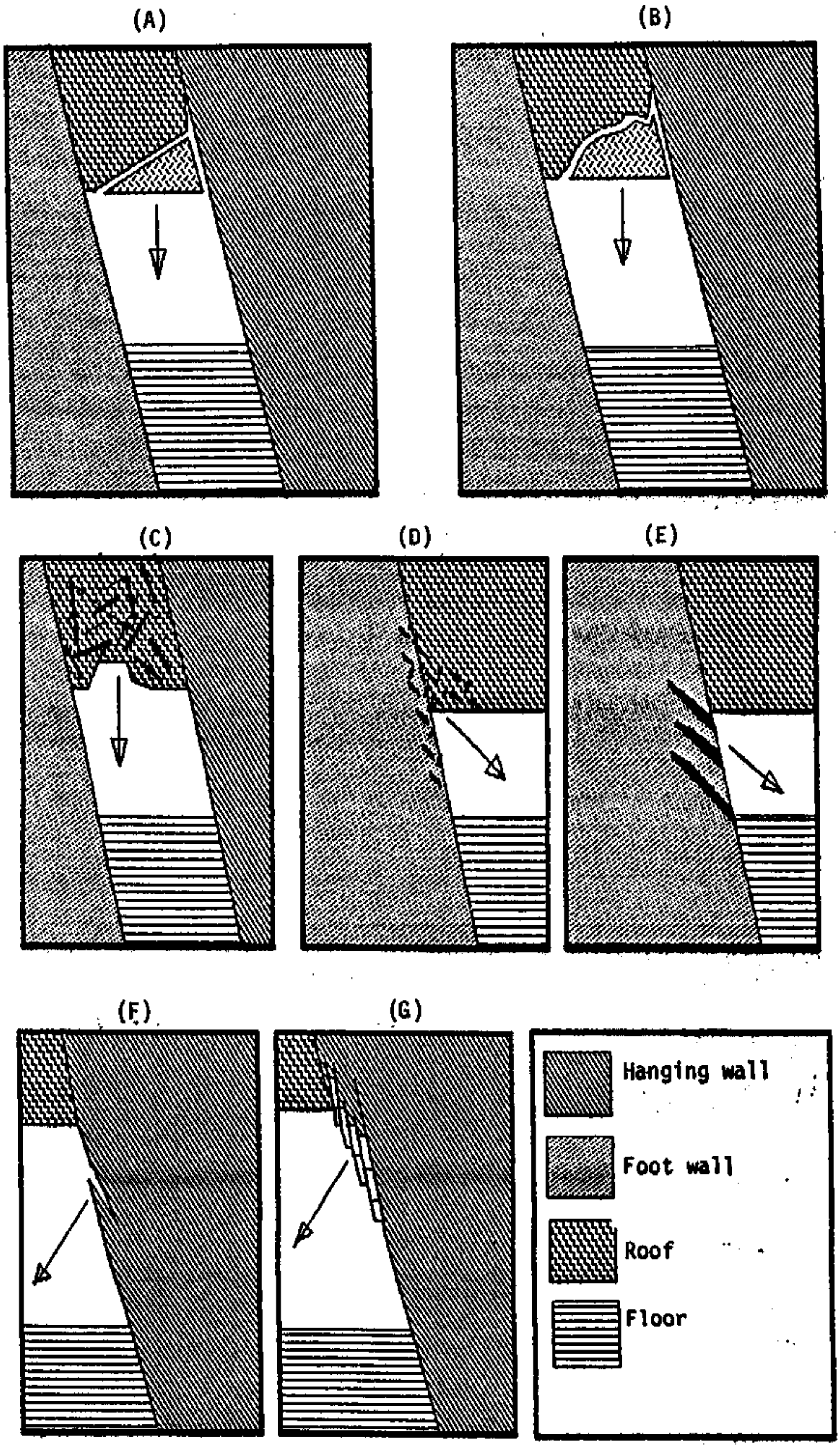
2. The projects

a. Skoumtsa chromium mine

Skoumtsa mine lies beneath the Vourinos mountain in the counties of Kozani and Grevena. The Chromium ore is extracted through an extended network of openings and other mining structures, either new or old. The mining method used usually is the Horizontal cut and fill, whereas variations of this method are tried such as the undercut and fill or the drift and cemented fill. The surrounding rock of the main mining works is dounite, which is usually serpentinized, highly tectonically fractured at all scales, with many usually wet discontinuities due to the local presence of high water table. Factors affecting adversely on the mechanical characteristics of the rock mass are, the presence or flow of magnesium hydroxide emulsion within the rock mass discontinuities which acts as a lubricant, the slickensided walls of the serpentine joints, etc.

The alternating cuts and fills extend 40 to 60 meters long

Figure 1. Typical failures at the Skoumtsa Mine



and 20 to 25 meters high. Their width equals it of the ore vein. These veins are highly inclined and extend usually 400 meter long, 120 meter high and have a width between 2.5 and 5.0 meter.

The rock mass either of the ore or the surrounding rock may be characterized according to CSIR as fair rock with an RMR value 50 to 60 and with an average stand up time between 2 to 4 weeks. High or medium faulting of the rock mass is one of the causes for any instabilities in the underground excavations. Especially difficult in mining are characterized the regions where the dounite is eroded into soft serpentinite. Rock blocks either from the hanging or the footwall fall within the mining faces and cause relative dilution of the mined product. In addition ore blocks fall from the immediate roof, whereas due to the particular prevailing conditions at the shear zones there happen larger scale roof falls.

Timber and steel is used mainly for the support of the openings, but full contact rockbolts are also employed. Ascending mining causes loosening on the hanging and footwall rock which in turn produces exfoliations and fracturing of the ore in the immediate roof. In general terms we notice an instable physicomechanical behaviour of the rock mass of the mine, which might be apart from the other mentioned causes, due to the high fluctuation of rock stress and especially of the horizontal pressure in the region. There are indications that this results from the sequence of mining. Rockmass might behave viscoplastically due to the high fracturing. When the width of the ore vein is less than this of the mining drift, rock falls are more pronounced due to the presence of new active discontinuities between the ore and the surrounding rock.

Most of the problems are due to the structural discontinuities which intersect and form blocks that fail due to their own weight. The rock mass is due to the tectonic stresses highly fractured. The excavation performed with drill and blast method causes high stresses in the surrounding the excavation rock which appear as exfolliations, minor rock block movements and opening of discontinuities. The rate of deformation is extremely high as soon as the hollow room is established, but soon after any deformation stops to appear. The most usual and important rockfall and failure cases are shown in figure 1. Cases (A) and (B) of the figure are present when there is lateral rock pressure. The cause of this type of failure may be attributed to the presence of the chromium grains arranged within bands and which act as rollers. The most abundant type of roof failure is shown as case (C). There are intersecting joints that create irregular rock blocks which equilibrate due to weak stabilizing forces such as friction or solution bonding. These forces are inadequate to bear on for the stability of the roof. Incipient collapse of large rock blocks derives from the lack of lateral constraint due to the fall of neighbouring smaller blocks. This process is progressive and may result in major rock falls. Failures that occur at the sidewalls of the hanging- or the footwall of the veinshaped deposit appear as in cases (E) or (F) of the figure. Cases (D) or (G) appear seldom, mainly in regions with high lateral pressures, such as where the inclination of the orebody is low and the fill area large, which happens in the last cuts of each sublevel.

b. Parnassos - Giona Bauxite mines

Bauxite orebodies are abundant in the Greek mainland and form a very important mineral for the economy due to their favoural bedding. The orebody has usually an irregular form in contrast to the hangingwall which has a smooth and planar surface. Mining activities expand within the geotectonic zone of Parnassos - Giona in the district of Sterea Ellada, particullarly in the geosyncline of the Parnassos - Giona the main axis of which has a southeast orientation.
The outcrops in those regions are white non crystallic limestones which stratigraphically belong to the Maistrictian period. Apart from the upper strata which is massive, the rock is usually thinly bedded. There are three bauxite horizons that belong to this stratigraphy in which the content in the minerals vaimite and diasporo varies. Nowadays the upper two horizons are mined and especially the uppermost third one, which underlies dark coloured roudistic bituminus limestones of the Touronian-Senonian period and overlies white limestones of the lower Cretaceous. The latter form an uncorfomity which is in angular discordance with the roof of the bauxite orebody.

Room and pillar is the prevailing mining method. Mining excavations must be supported on ore pillars. Design is based on the assumption that stresses at any point whithin the surrounding rock should not exceed its strength. Various types of rockbolts are used in the roof and the sidewalls of the underground openings for the prevention of rockfalls. Their design is based on empirical and generally accepted rules which have performed effectively. As a result there have happened no more than ten signifficant failure cases in the extended underground mining activities of the larger greek bauxite mining companies. Experience has shown the following :
Discontinuities and joint patterns of the surrounding the excavation rock is the main cause for roof falls. Clay material filling these discontinuities separates rock blocks which may be detached and become unstable. Microfracturing in problematic regions is intense thus leading to worse rock mass quality compared to other regions, Critical discontinuities that trigger the plate separation or other

rock blocks in the roof dip shallowly less than 30 degrees, and intersect the roof.

Miners do not perceive those key discontinuities which are perilous, due to the abundance in the roof of safe random discontinuities and limestone bedding planes that may not be distinguished from the former. Large roof falls are considered to be extremely rare in the vast quantities of greek bauxite mined roofs, if compaired to other mining activities of similar magnitude in the greek or international space. This bears on the favourable mechanical response of the limestone, its satisfactory response to the creation of the underground opening, and the room and pillar behaviour prediction design method.

Roof falls may be classified according to their magnitude in two classes. In the first class the boundary or at least its fracture initiation region of any unsafe rock block may extend deeper than the length of the used rockbolts (Fig. 2a). In the second case the rockbolt is long enough to support the rock block. A fall may happen either if there is an insufficient number of bolts which may cause their slip or overstressing, or by tension cracking (Fig. 2b) of the immediate roof between the bolts which causes the fall of usually small fragments.

In some cases failure does not happen along critical discontinuity planes with clay infill, but along fractures created after the formation of the opening due to stress concentration (Fig.2c). This new planes may combine with the structural discontinuities of the parent rock mass and form unstable rock blocks.

Cross cuts for the creation of the rooms and the junctions of the drifts are important in the creation and development of the falls. The rock mass of this region after having undertaken the double blasting effort has to act as a supporting pillar for the overlying rock mass above the junction. Any yield of the pillar affects immediately the roof and causes detachment of the roof rock plate.

Designers believe that the existing pillar faces are not adequately strong due to their fracturing or injuring during the drill and blast process (Fig. 2d). Hence the stability of the room should be based on a reduced support area of the pillar and a larger span of the room.
Although failure may sometimes occur without any warning, nevertheless rock failure or detachment is usually preceeded by noise and tremor, falls of small rock fragments, violent rock bolt failures and linear propagation of primary roof fractures. The operator of the drilling rig for the installation of the rock bolts perceives usually any weakness of the overlying rock, from the rotary and percussive speed and stress of the drill, from the created noise, from the

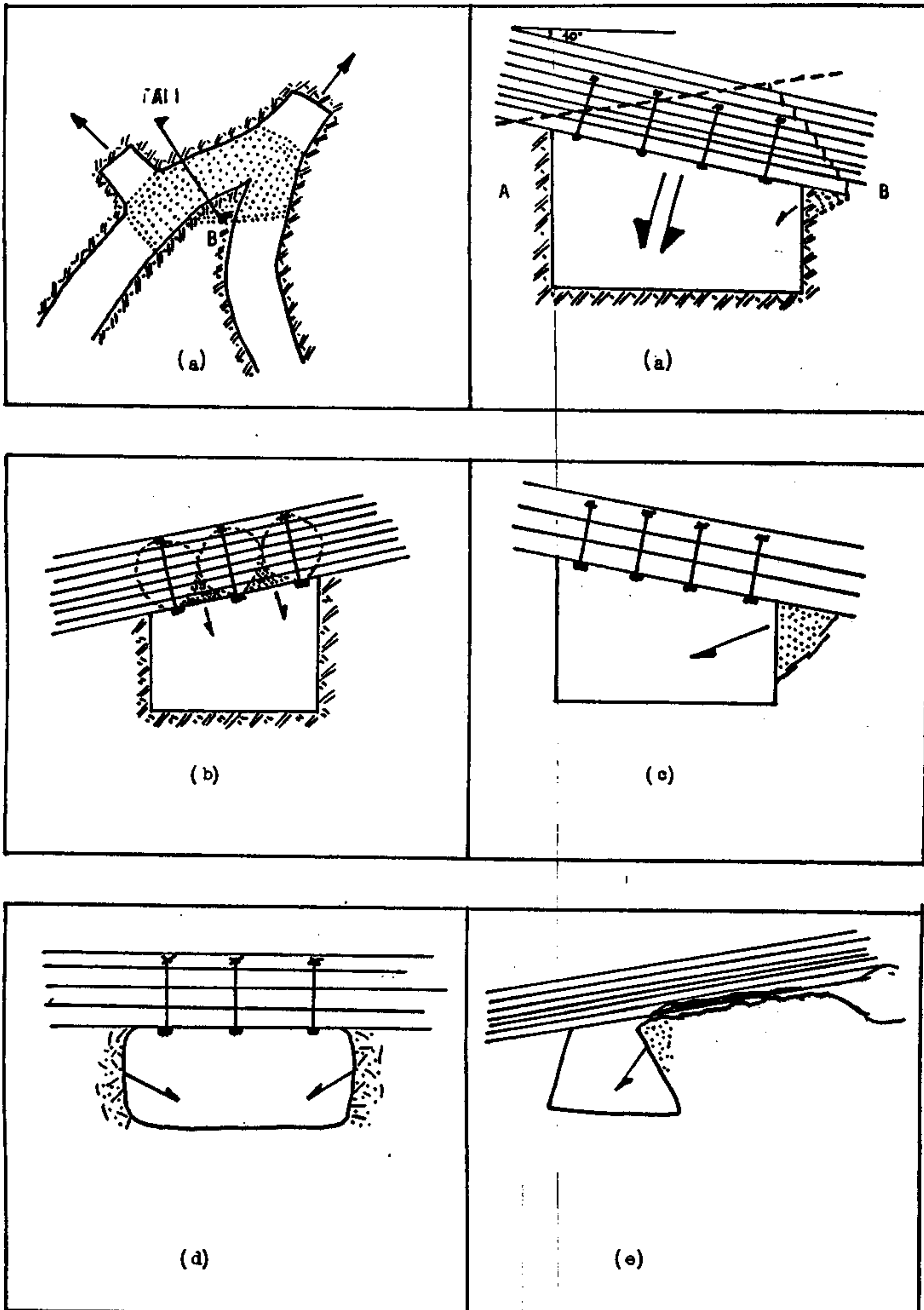


Figure 2. Failures at the Parnassos- Giona underground Bauxite mines

smell, etc.

The bolting methods used and their design principles are considered to be productive and successful for a vast range of the rocks surrounding the mining openings of the bauxite ore deposits. Any falls or roof failures are related to very peculiar rock mass conditions, as shown e.g in figure 2e, in conjunction to unusual arrangements such as cross cuts and junctions.

c. Aoos springs (Piges) hydroelectric scheme

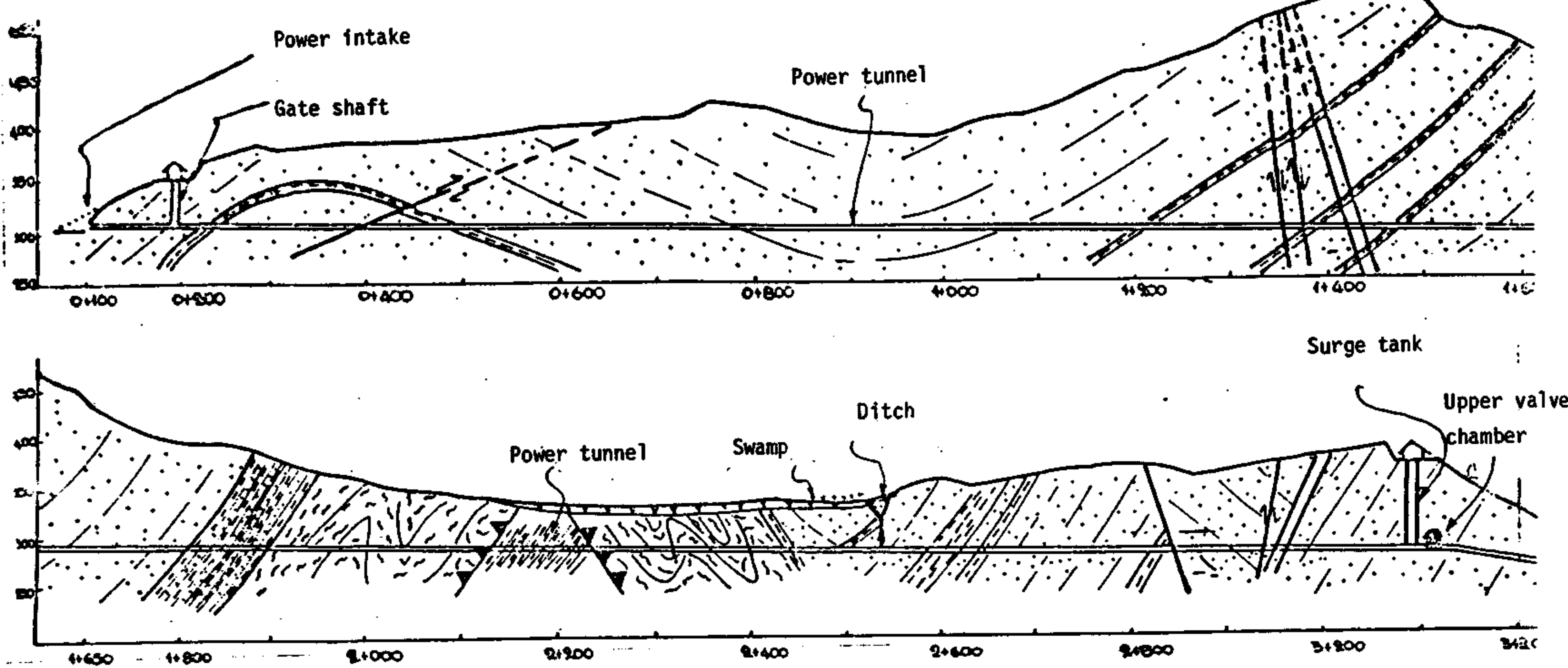
The scheme lies in Northwestern Greece. It comprises a series of dams in the Aoos river near its springs for the formation of an artificial lake, a series of underground tunnels and an underground Power house. From these works two tunnels are to be discussed, i.e the power and the diversion tunnel.

c1. The Power tunnel

This tunnel is 3200 meter long with a 15 square meter cross section. Its larger part is excavated in thick layered massive sandstones of the Pindus flysh with circa 200mm thick intercalations of siltstone. The rest is excavated within 100 meter alternations of sandstone and siltstone of the Pindus flysh, 450 meter red siltstone of Pindus flysh, and 150 meter siltstone maybe of the Ionian flysh (Fig. 3). Water ingressed through the sandstone layers. 200 to 300 cubic meter of water per hour discharged within the opening during the excavation of the siltstone and the sandstone alternations, which nevertheless ceased in a few hours as the capacity of the aquifiers was limited. The capacity at chainage 2550 was significantly higher.

The excavation method was full face drill and blast. It advanced in steps of three meters in the thick layers of the sandstone to one meter in the red siltstone. In the former region of the sandstone support consists of two to five rockbolts per meter length of the tunnel and shotcrete sprayed locally on the roof only. In the region of the siltstone or the red siltstone support consists of five to seven rockbolts per meter and a lining of shotcrete 50 to 100 mm thick. In the region from chainage 1900 to 2150 the red siltstone is heavily fractured and support consists from steel ribs spaced 1 meter apart, besides the rockbolts and the lining.

Rock failures mainly structural happened during the excavation in the red siltstone area only, particularly in the region where they appeared heavily fractured. There it



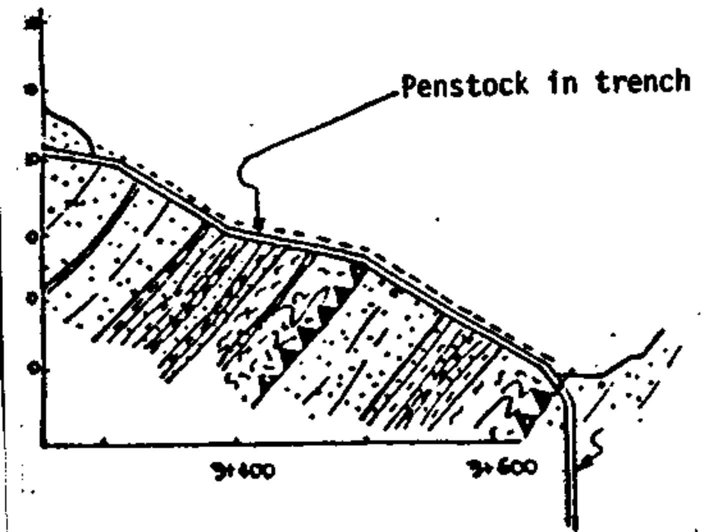


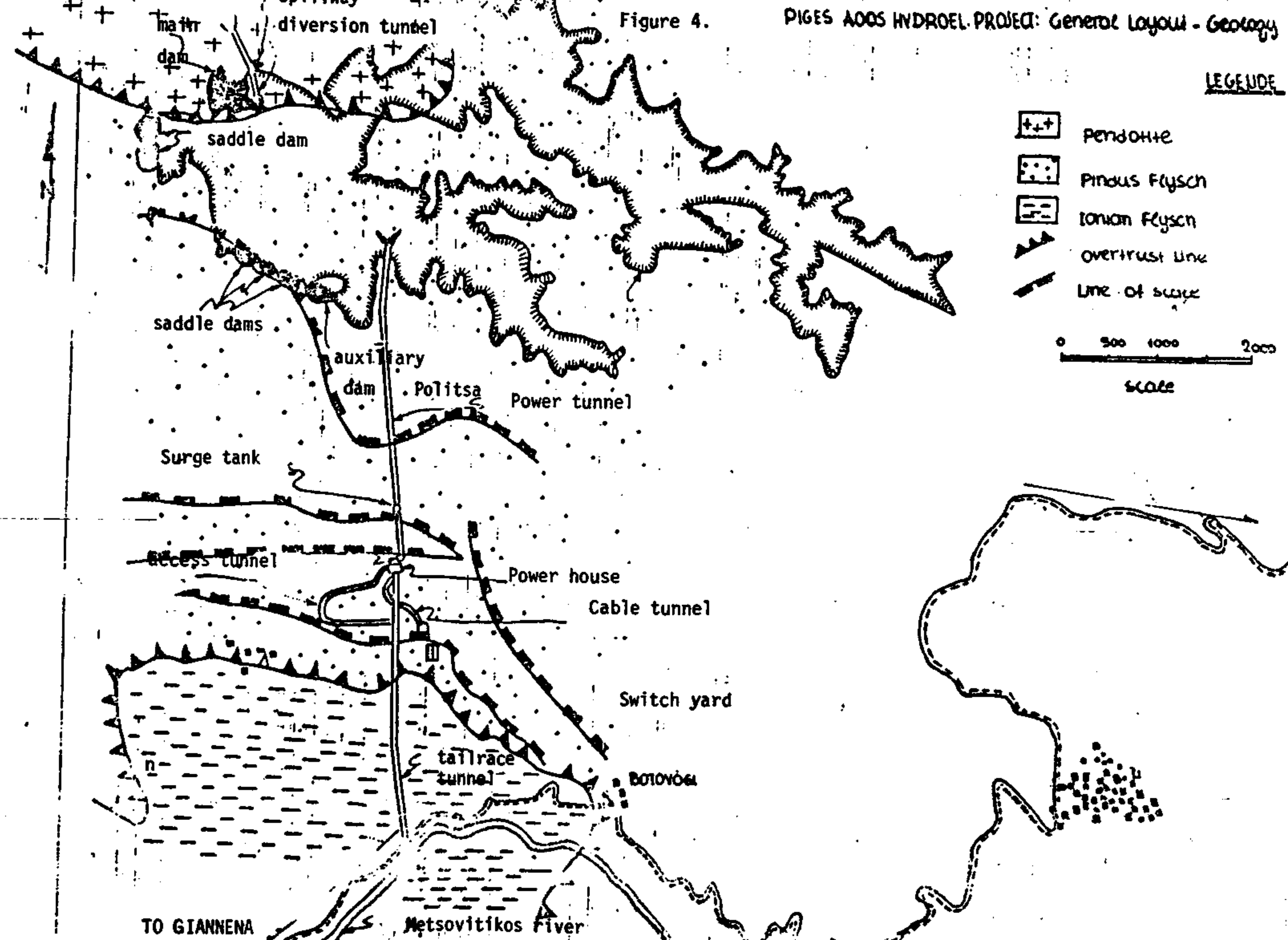
Figure 3. HYDROELECTRIC PROJECT: Geological section along waterway

LEGENDE

- Siltstone possible of Ionion Flysch
- Thick Layers sandstone of Pindus Flysch
- Alternations of sandstone and siltstone of Pindus Flysch
- Red siltstone of Pindus Flysch
- Overthrust line

0 50 100 150 200 m

Figure 4. PIGES AOOS HYDROEL.PROJECT: General Layout - Geology



happened a rockfall of approximately 150 cubic meter. The fall was attributed to the high rate of advance of three meter per step. Stability problems were successfully overcome by reducing the rate of advance to one meter and by changing the support measures by incorporating steel ribs.

After the excavation four meter deep cement grouting was injected in order to stabilize the rock which might be disturbed by blasting. The absorption rate of the grout in the sandstone region was as high as four tonnes per meter length.

c2. The diversion tunnel

The diversion tunnel is 650 meter long and its cross section is 19 square meter. It has been excavated entirely within peridotites. Its entrance lies at a distance of 250 meter from the overthrust line (Fig. 4). In the 300 first meter the rock mass was heavily fractured and serpentinized which was then reduced. Heavy stability problems were encounterd in the former region such as :

a. Full face advance could not stand unless supported within 0.50 meter.
b. The stand up time for the unsupported full face excavation was small and it ranged from a few minutes to several hours.
c. Stability reasons prehibited the use of the conventional method of drill and blast excavation, associated either with small advance steps or with partial face excavation.
d. Any attempt for rock improvement by grouting was not successful due to the small absorption rates of the rockmass. Similarly an attempt for prereinforcement of the roof with rockbolts did not succeed, due to defective grouting of the boreholes attributed to their instability.

The excavation proceeded manually with compressed air jackhammers. Drill and blast was used locally only. Excavation of the necessary circumference was performed initially in order to install each steel rib. The excavation of the rest of the cross section proceeded after the installation of the support measures. The latter comprised steel ribs at 0.7 to 1 meter intervals and shotcrete.

Many rockfalls occured of which four incorporated a volume of 40 to 50 cubic meter each. Those occured prior to the installation of the support measures. After their installation there were none but one visually observable displacement. The latter concerns a ten meter region of the tunnel at the interface of the highly fractured with the less fractured peridotite in which the water discharge was significantly higher. High convergences of the steel rib footings were measured in this region, which necessitated the application of additional support measures.

Rockfalls happened in a progressive manner. They initiated with small wedge failures from the roof and proceeded into a complete rockfall in about two hours. This was attributed to the high fracturing and the erosion of the rockmass and the slickensided nature of the walls of the discontinuities. High stress concentration was created after the removal of any rock block which accelerated the precipitation. Hence of primary importance is not the kind but the time of installation of the support measures. This ranged from a half to two hours for the particular excavation that advanced with a rate of 0.7 to 1 meter per step at the top heading.

In order to rehabilitate the region where the rockfalls occured the face was sealed with shotcrete and the rockmass was after injected with grout. Tunnel excavation with a rate of advance of one meter per day was performed again carefully followed with immediate installation of the support measures.

In the rest of the tunnel where fracturing and erosion are lower, the excavation proceeded with the full face drill and blast method with an advance rate of two meters per step and day. Support measures comprised steel ribs and locally shotcrete.

d. Artemissio highway tunnel

Artemissio highway tunnel joins the counties of Argolis and Arkadia. Its 1356 meter length passes through cretaceous limestones, cherts and clay schists (Fig. 5) which are estimated to be classified according to CSIR as rock mass classes not more than IV. The average dip of the limestone strata was 35 degrees and their dip direction crosses the tunnel diagonally. The larger overburden depth is 370 meter. The water table is above the level of the tunnel and discharges quantities of water from the pervious limestone within the tunnel. The highest water table is 100 meter high and it was measured near the overthrust line at chainage 1200.

Drill and blast is the excavation method which is performed in one to three stages. Most of the excavation proceeded from the Nestani site (right). The last part of the excavation was driven from the Neochori site (left). The immediate support comprises mesh reinforced shotcrete, rockbolts and light steel sets. Final lining consists of unreinforced concrete 300 to 400 mm thick.

All the rock failures that happened were structural. Those that embodied a rock volume larger than 10 cubic meter are

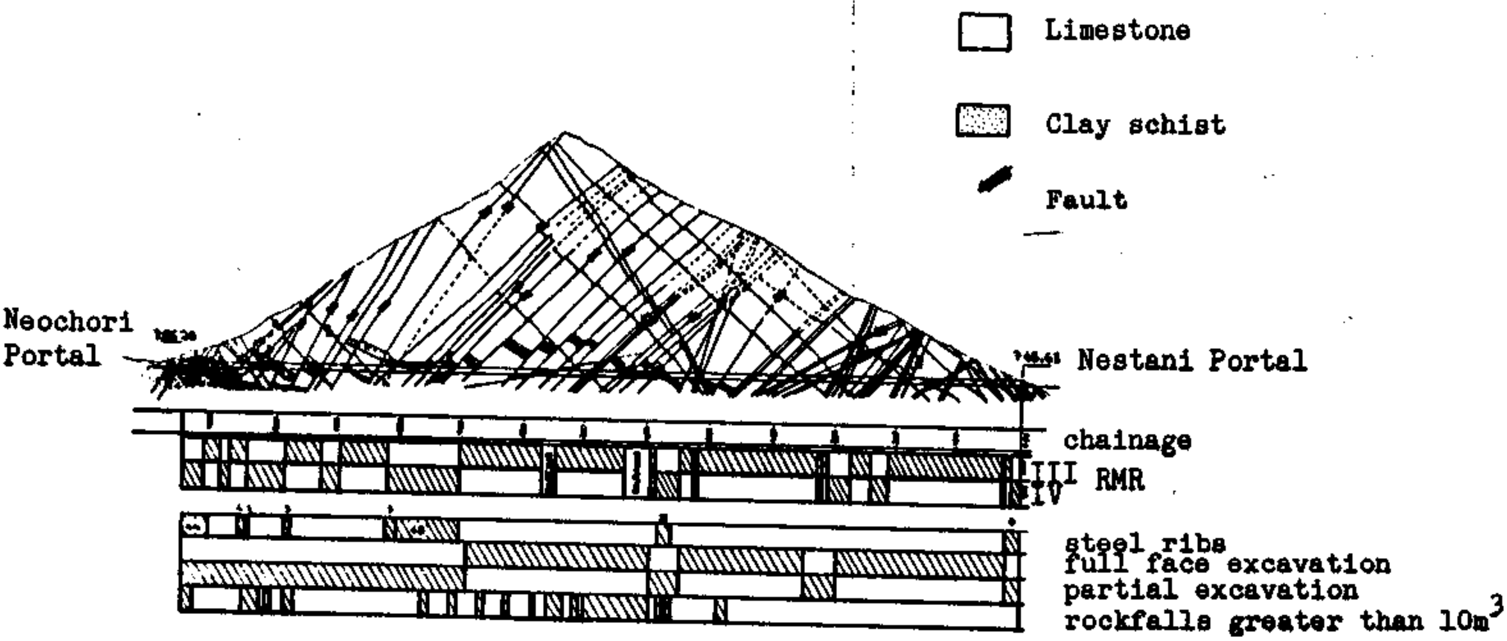
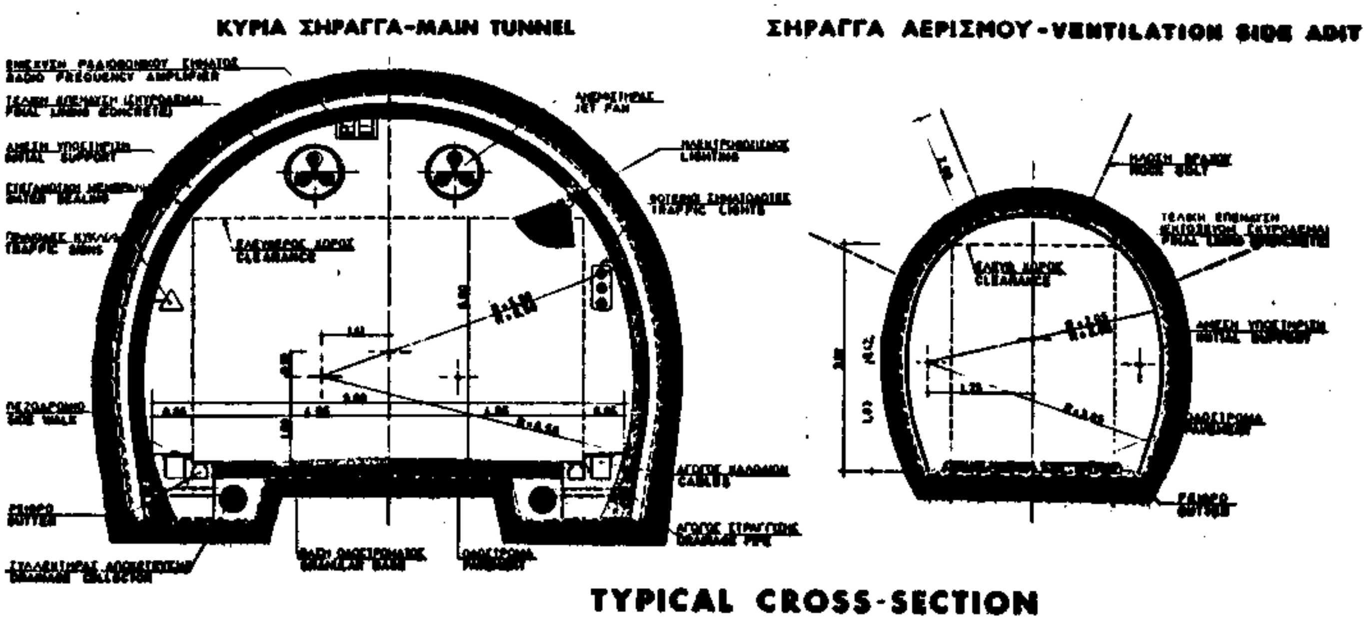


Figure 5. Geology and rockfalls in a longitudinal section of the Artemissio tunnel.



PROJECT FEATURES

- Total tunnel length	:	3.516 m.
- Horizontal alignement	:	Curved at the entrances, tangent in between
- Longitudinal gradient	:	0,5%
- Width of traffic lanes	:	2 × 3,75 m.
- Gutters width	:	2 × 0,30 m.
- Sidewalks width	:	2 × 0,85 m.
- Minimum clearance (height)	:	5,00 m.

Figure 6. Cross section of the Metsovo highway tunnel

shown in the figure. There were almost no rockfalls during the first 550 meter in which the rock is characterized as thinly bedded limestone and may be rated according to CSIR as rock mass class III (fair rock). Many rockfalls were anticipated from chainage 550 to 900 although the quality of the rock did not change. This may be attributed to the clay schist layers which appeared unpredictably in the roof or the walls of the excavation and separated limestone blocks. From chainage 900 on as the overthrust line is approached, the schist layers become abundant and the tunnel face may be termed as mixed. A slower rate of advance performed in two stages, the immediate application of support and the installation of steel ribs, prehibited almost any rockfalls untill chainage 1175. In the region of the overthrust line from chainage 1175 to 1250 many rockfalls occured although the excavation proceeded very carefully in more than one stages. In this region convergences as high as 39mm were measured. Within the clay schist area from chainage 1250 on there were almost no rockfalls although the rockmass might be classified according to CSIR as rock mass class IV (poor rock). Apart from the very cautious excavation process, this may be attributed to the period of performing it, which was a dry one that did leave the clay schists dry.

e. Metsovo highway tunnel

Metsovo tunnel is 3500 meter long and is the largest excavated highway tunnel in Greece. It is excavated in two stages from both entrances. Parallel to the highway tunnel a ventilation side adit is constructed. Excavation of the latter is complete as is the excavation of the top heading of the former. In figure 6 the cross section of both tunnels is shown. Plan view and longitudinal section of the tunnels are shown in figure 7. Six categories of excavation support type are imposed by the contract which depend on the quality of the rock.

The excavation of the main tunnel from west to east passes through Alluvium, Flysch in the form of silt and claystone, weathered altered Serpentine, Peridotite or Serpentine, red Siltstone and Gabbro. The ventilation adit started from the weathered altered serpentine on. The water table is above almost all of the excavation. Except for the Gabbro (which was excavated full face) stability problems were encountered in all the rest of the tunnel.

In the claystone of the flysch squeezing and maybe swelling of the rock caused excessive buckling of the every meter installed twin steel arches. Convergence exceeded many tens of centimeters. The weathered peridotite which lies in the overthrust zone had to be machine excavated in one meter steps, supported immediately with steel ribs and shotcrete and in a second stage with rockbolts. A bulb of the face

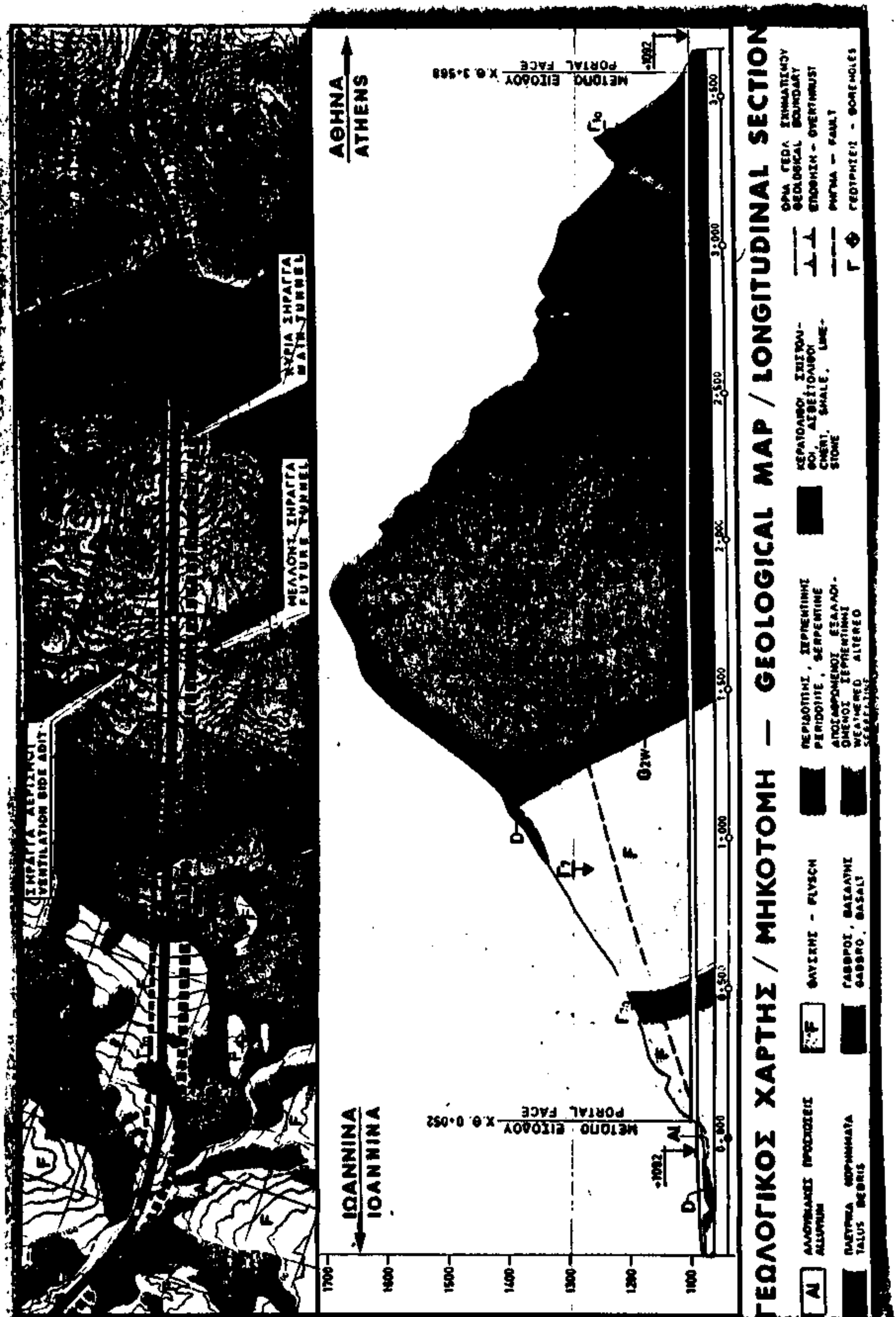


Figure 7. Plan view and longitudinal section of the Metsovo tunnel.

shotcreted rock had to be left unexcavated in order to provide support to the face. All this was necessary in order to avoid collapse.

The peridotite is strong as a material, but quite heavily fractured. It had to be immediately shotcreted in order to remain intact. The red siltstone seemed to be the most difficult to excavate. Although supported with double steel ribs every meter, it buckled them heavily producing a high convergence of them. This necessitated the application of a dense mesh of rockbolts which seem to have stoped the convergence rate. The area has to be excavated again in order to obtain the necessary profile. Benching will start now and any new failures have to be observed.

3. Conclusions

Rock failure in underground openings constructed in Greece is investigated in some of the most important current projects in the field of mining, hydroelectric energy conduits and highway tunnelling.
The overwhelming majority of rock failure may be attributed to structural causes. This is especially true for the case of highway tunnels as their positioning is tried to be based on the best possible geology avoiding as much as possible weak or soft rock. Nevertheless the large diameter of the cross section intensifies any structural failure possibility. Artemissio tunnel was such a case. Metsovo tunnel passed a considerable length within weak rock under the water table after considering various alternative routes. Rock failure is structural within the unwheathered peridotite and the sand and siltstone. The larger problems are with the squeezing claystone, serpentinized peridotite and red siltstone which are not structural failures.

Hydroelectric project tunnels have not the versatility of the highway tunnels as the positioning of the various conduits has to be put in one of the two abutments of a complete project masterplan. This necessitates the driving of the tunnel within weak and swelling rocks and within fracture and fault zones. In the case of the power tunnel it was possible to drive within the sandstone which was almost successfull. A small length driven within the red siltstones was troublesome.
In the case with mining there is almost not versatility at all as the production drifts have to be close or within the deposit. At the Bauxite mine failure is almost structural due to the limestone rock surrounding the deposit. At the Skoumtsa mine failure of the surrounding unwheathered dounite rock is structural. In the regions where the dounite is serpentinized or wheathered, failure is due to squeezing. Failure of the ore material is either structural or due to the disintegration of the ore rock.