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Keyonote Lecture

TUNNELLING AND MINING IN KARSTIC TERRANE; AN ENGINEERING CHALLENGE

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ABSTRACT

Although limestone and most carbonate rocks exhibit good geotechnical behavior, when karstic, they may induce hazards during tunnelling operations, which may evolve into huge problems. Groundwater is the main source of problems and so is the crossing of voids and caverns, either empty, aquiferous or filled. In order to estimate the probability of encountering such conditions and be prepared to face them, a thorough hydrogeological study should complement the traditional site investigation program. This study has to consider a broader area embracing the whole hydrogeological basin of the karstic aquifer with background knowledge of the tectonic and paleogeographic evolution. In this paper a series of hydrogeological models are discussed depending on the internal karstic geometry of the aquifer and the position of the tunnel, either in the transfer or the inundation zone. Each model is associated with its own tunnelling particularities in terms of hazards and countermeasures. The crossing of big limestone mountains is discussed through a case history since it is very likely that the interior of similar mountains is not affected by karstification and in order to present a number of deviations from the persisting hydrogeologic regime. A discussion on the solutions to be engineered in order to cross big karstic cavities is also presented. For mining in karst, two case histories on the efforts to achieve effective dewatering illustrate the scale and the size of such difficult operations, along with the associated environmental implications.

INTRODUCTION

Tunnelling can be a high risk business

Groundwater is often the main source of problems in tunnel construction associated with stability and safety issues. Groundwater control during both construction and operation of the tunnel, is one of the most challenging problems faced by tunnel designers and contractors. Drainage facilities from the headings may be required and when the necessary invert grades are not available, the additional trouble and expense of pumping are unavoidable. Water can affect roof and face stability and in appreciable quantity will impede construction. If the host ground is soft and prone to erosion the risk is further increased.

Seepages, or leakages, into underground works from the surrounding aquifer can also affect the surrounding ground and adjacent facilities. Depending on local geology, hydrogeology and geotechnical parameters of the material, a severe environmental impact may be expected. In the opposite case, i.e. when leakages from underground works to the aquifer are possible, the hazard of groundwater contamination has to be considered.

Mining works due to the extension of the underground void space involved and to the absence of lining, can affect more seriously the surrounding hydrogeological conditions and increase the risk either for the development of instability or for the depletion of the groundwater resources.

The crossing of voids and caverns, either empty, aquiferous or filled with erodable material causes difficulties and the solutions that should be engineered, are often site specific.

Hence, although limestone and carbonate rocks in general exhibit a good geotechnical behavior, when karstic, they may induce all the aforementioned problems in tunnelling operations[*]. Many large engineering projects involving tunnels are currently under construction in countries where limestones are a very common geological formation. The design of underground excavations in these materials requires knowledge of the geological and hydrogeologic model in which these excavations are carried out.

INTERACTION WITH GROUNDWATER; GENERAL CONSIDERATIONS

The interaction of tunnelling and mining works with groundwater can be summarized as follows:

During construction

- Inflows of water in the underground space, affecting normal construction procedures and possibly induce face and roof stability.

[*] In the following text the term limestone refers also to all carbonate rocks that undergo karstification.

- Sudden inflows associated with specific and localized geological features, e.g. faults, crushed zones, big karstic conduits etc.
- Decline in yields of springs, decrease of groundwater discharge to wells.
- Development of sinkholes in susceptible areas due to piping or internal erosion.
- Acceleration of dissolution of soluble sediments (e.g. gypsum).
- Unacceptable settlements, where compressible fine-grained soils or heavily fractured rock masses are present, due to the increase of effective stresses by lowering of the groundwater table.
- Temporary contamination of groundwater occurring at lower elevations, by infiltration of polluting substances used for the construction.

During operation

- Infiltration of used chemically and organically contaminated waters from road or rail tunnels can affect the quality of the groundwater if the tunnels are crossing the non saturated zone.
- Rise of piezometric levels by the obstruction of groundwater flow by lined tunnels; the rise is effective when the tunnel is located at a shallow depth under a shallow water table and can affect the built environment (foundation, basements) and/or mobilize contaminants in case of saturation.
- Influence of the hydrostatic head on the lining of the tunnel.
- Tunnel collapse by wide fluctuation in hydrostatic pressure associated with normal operation of hydraulic unlined tunnels.
- In the case of water conveyance tunnels with lining deficiencies, the relation between the head of the waters flowing in the tunnel and the head of the surrounding aquifer can cause:
    - Inflow of eventually polluted waters in the tunnel and/or development of all the related and abovementioned risks (internal head lower than the head of the aquifer). Underground excavations containing fluids such as petroleum products at near-atmospheric pressures can be left unlined if the rock quality is high and if the excavation is below the water table since the fluids are contained by inward seepage of groundwater
    - leakages from sewer tunnels can contaminate the surrounding aquifer (interior head higher than head of the aquifer); leakage is a major concern when tunnels carry high-pressure water with toxic ingredients. Such fluids must be contained by an impervious liner.

INVESTIGATION

General considerations

It is essential to have accurate preconstruction assessment of groundwater conditions. No major underground engineering operation should be initiated before a comprehensive knowledge about the loads and flow regime of groundwater is established.

In the case of a tunnelling project close to the surface or in urban areas a good number of investigation techniques suitable to provide direct information and measurements are available (on piezometric heads, permeabilities, discharges). Geophysical investigation is often of great assistance. Unforeseeable conditions are thus very constrained.

In case of long tunnels at greater depths (in mountainous areas) the investigation possibilities are rather limited due to high cost. In such areas the investigation is mainly based on classical hydrogeological studies, procedures and techniques and covers a broader area for getting all necessary data and all geological boundary conditions. A study of this caliber must be based also on some kind of geological judgment.

This procedure must include:

- identification and classification of aquifer media (lithological and structural mapping)
- distinction of hydrogeologic units and water tables
- definition of hydrogeologic basins (underground catchment areas) and of the discharge areas
- delineation of water budgets
- study of springs: location, elevation, flow dynamics and discharge rates
- compilation of piezometric maps
- evaluation of hydraulic parameters both locally around the tunnel and in the broader area and basin (permeability, transmissivity, storativity)
- conclusions in the form of a report on the hydrogeological and geometrical boundary conditions for each aquifer and evaluation of heads and inflows relative to the underground construction. The report must provide also approaches for likely zones of sudden inrush hazard, such as fault zones.

Particularities in karstic rock masses

The particular or even unique hydrogeological features in a karstic environment demand special attention as there is an increased risk for water inflows and for environmental problems. Tunnelling in limestone terrane may thus be a challenge for both geologists and engineers owing to:

- high coefficient of infiltration from meteoric water.
- very high permeability; often non linear underground flow.
- preservation of high values of permeability at greater depths.
- potential of development of large hydrogeological basins, which may extend far beyond the boundaries of the corresponding geographic - hydrological basins of the considered area, involving, thus, greater quantities of groundwater.

- development of a non uniform, heterogeneous pattern of flow paths; depending on the post-tectonic and paleogeographic evolution of the area, preferential flow conduits and karstic tubes could be developed with a capacity to transmit water at large discharge rates; these conduits drain the surrounding jointed or finely fractured rock mass of low or medium permeability.
- groundwater flow in a flooding manner throughout the transfer ("unsaturated") zone.
- potential crossing of large underground cavities filled eventually with earth materials, with the possibility also to carry a column of perched ground water.

POTENTIAL HYDROGEOLOGICAL MODELS TO BE ENCOUNTERED

During the first stage of investigation in a limestone terrane it is crucial to understand the karstic pattern around the tunnel by means of a detailed hydrogeological study*. Such hydrogeological study should include a paleogeographic evaluation of vertical movements and changes of the geographic base level related to past locations of springs, in order to assess the depth of karstification inside the limestone mountain and the geometry of the karstic base level. This level is not necessarily restricted at the present elevation of the surficial springs. Thus, the geological reconnaissance in a broader area is a prerequisite for the investigation regarding tunnelling in karstic terrane.

Dye tracing testing and follow up of the route of major underground flow axes, i.e. between sinkholes (ponors) and springs, greatly assists the understanding of the delay of underground flow and is thus elucidating as to the presence of potential branching of the large karstic conduits or a general dispersion of flow to several directions. In this same rationale, the study of the distribution and the hydrographs of springs is always the most reliable tool for understanding the internal structure and geometry of a karstic aquifer, since it reflects the hydrodynamics of the interior of the karstic mass.

The question of whether concentrated or dispersed inflows are to be expected is of great concern since the former may threaten tunnelling operations. A detailed structural analysis of the hydrogeologic basin will define zones of possibly very high permeability (i.e. faults, or systematic bending zones).

Finally, the position of groundwater levels and fluctuations in the investigative boreholes, must be recorded at all times since they reflect the thansmissivity of the whole karstic mass. In the case of tunnelling in mountainous areas, pumping tests from wells, even if feasible, are not as helpful as for tunnels in low relief terrain. In those cases, packer tests restricted in the zone around the tunnel controlling the inflows, is a common practice.

Table 1 intends to provide the main hydrogeological models in a limestone environment. The answer on the most probable model to be crossed will facilitate the appropriate design of the tunnel and the provision of the methods and equipment necessary to face the hazards associated with the karstic conditions to be encountered.

Case1: Groundwater issues are considered as for a jointed or fractured rock mass. Permeability is generally low and decreases dramatically with depth. Exceptions may occur in fault zones.

- **Model A:** Tunnel will cross a completely dry limestone mass; no risk for floods
- **Model B:** Tunnel will encounter medium to insignificant flow, depending on the frequency and

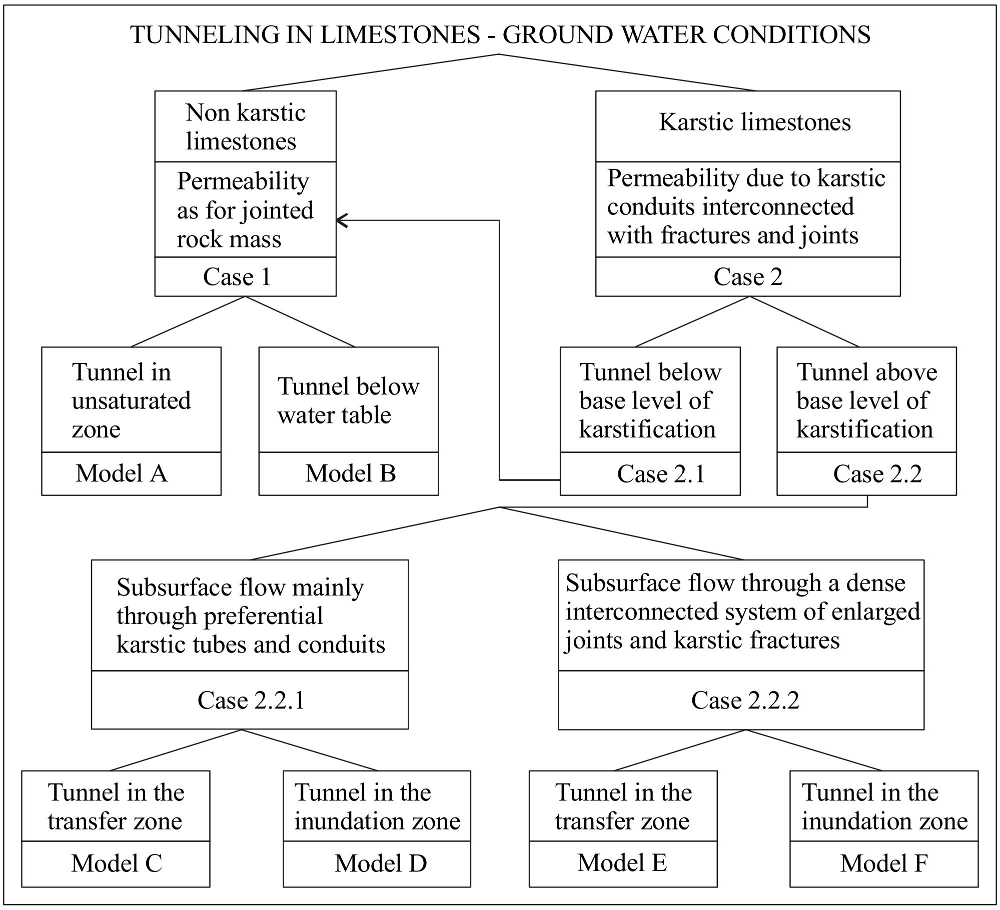


Table 1: Potential hydrogeological models in limestone environment. Note that in some cases (e.g. platform karst) the inundation zone may be insignificant or transient. Carbonate rocks with substantial primary porosity can be considered of the finely-jointed type presented in this table. Few climatic type of karstification may produce patterns different from those above.

* The reader can get insight on karstic processes in some excellent recent publications (Breznik, 1998, Milanović, 2000 and White, 1999).

aperture of joints or fractures.

Case 2: Dramatic difference in behavior compared with other aquiferous media; presence of high permeabilities, large discharges.

- *Case 2.1:* The rock mass surrounding the tunnel has never been exposed to underground erosion due to the paleogeographic evolution of the area or its isolation from infiltration and flow to outlets. In low relief morphology, the past geographic base level of the area to be crossed has never been lower than that of the tunnel. However in large mountainous masses the interior of the mountain could have escaped karstification and the base level of karst lies at much higher elevations than the present level of the springs. Tunnels with such conditions will comply with either model A or B.
- *Case 2.2:* The size of the problems and risks depend on the internal geometry of the karstic system. Two options are possible:
  - *Case 2.2.1* when the underground flow is mainly concentrated and governed by distinct preferential large karstic tubes and conduits or,
  - *Case 2.2.2:* when flow is guided by a more homogeneous interconnected system of karstic fractures and enlarged joints. The latter is usually the case of well-bedded limestone in areas characterized by a long lasting persistence of an extended flat geographic base level. The former is often the case where a continuous downward underground erosion persists as the geographic base level was progressing towards lower elevations or where the lowest geographic level was restricted to a confined zone.

Model C*:* The tunnel is in the transfer zone of a selectively highly karstified mass. It will cross dry limestones but if located at depth the hazard for personnel and equipment from sudden inrushes and flooding will be high when storms occur in the catchment area. The stability of the tunnel might also be endangered. Erosion of loose filling material may result to a mud flow into the tunnel. Probing ahead should be a common practice. Contamination of the underlying "water table" is a real risk.

Model D: The tunnel is in the inundation zone and will drain moderate quantities of ground water between karstic conduits. These quantities are fed by water stored in fractures between these conduits. Upon encounter of the conduits, considerable increase of inflow will be experienced and violent inrush or flooding of the tunnel cannot be excluded. Probing ahead during construction is an absolute need. Predrainage techniques with site specific character should be applied in order to assist the crossing of the conduit. A quasi-permanent drainage of the karstic aquifer will last almost all of the construction period. The water resources of the area will be affected. Ground water discharges from the limestone mass between karstic conduits can be approached by the graph of Fig. 1. This estimation does not apply for the discharges of the conduits themselves.

Model E: The tunnel is in the transfer zone of a dense interconnected system of slightly karstified joints and fractures of moderate aperture. It will cross a mass with dripping waters or small amounts of transient water during wet periods. There is no risk for floods as the infiltration is widely dispersed inside the karstic mass.

Model F*:* The tunnel, being in the inundation zone, will drain, almost permanently significant or very significant quantities of ground waters during the construction, imposing the need for appropriate draining equipment. Violent inrushes should be restricted. Special design arrangements are to be implemented (i.e. diversion of waters to the sides of the tunnel). A drainage umbrella in front of the face should_reduce the head and control inflows during the excavation (Fig. 6). Stability problems may occur only if the limestone is brecciated. Groundwater resources can be seriously affected.

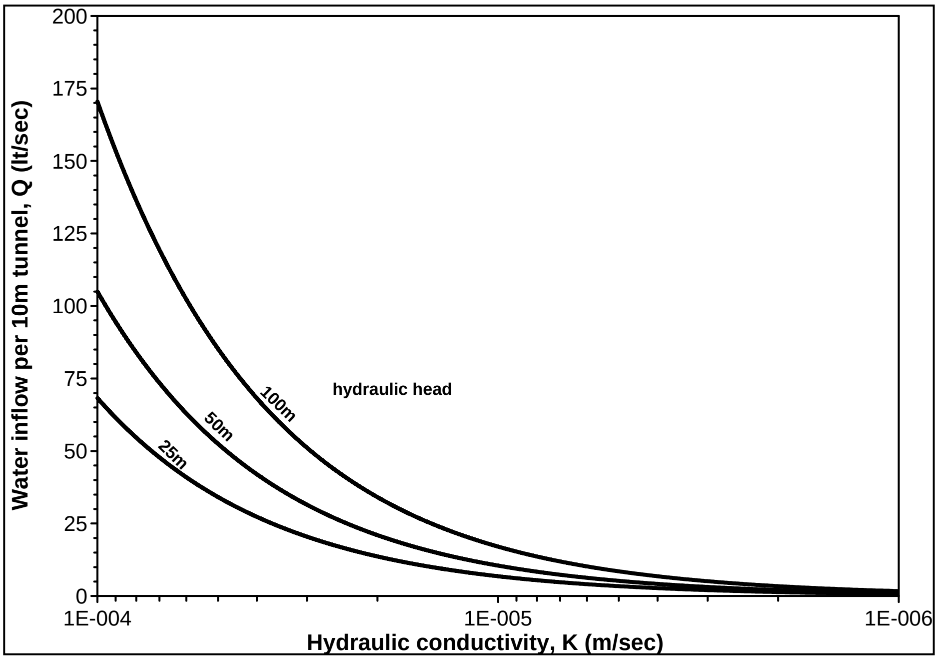


Figure 1: Estimation of water inflow in a 10 m diameter tunnel for steady flow condition. This graph can be applied in the inundation zone of a limestone aquifer for estimating maximum values before transient flow is established and in sections between two main karstic conduits. It does not apply to discharges through the conduits themselves. These conduits may recharge their fractured-jointed limestone environment simulating steady flow conditions.

Some Case Histories

Any tunnel in a karstic environment offers experiences or incidents regarding facing of groundwater problems. However little is published in scientific or technical literature. The karst commission of the International Association of Engineering Geology in a report published by L. Calambert, in 1975, succeeded at that time to collect a number of cases, some of which are presented followingly.

Several cases are reported in Spain, where large quantities of water flooded a number of tunnels (Yagüe, A. 1975, *in* Calembert, 1975). In the Talave tunnel, quantities in the order of 1000 l/sec owing to structural features, faults or synclinal zones, were diminished only after several months. Large quantities of water were also encountered in a tunnel in Asturia, where all works were ceased until a drawdown of the water table was achieved. An important amount of fill material was also eroded and filled partially both tunnels.

An interesting case is reported during the construction of the highway tunnels of Gran Sasso (Calembert, 1975). One of the tunnels came upon a thrust fault with a heavily sheared zone 25m of thickness. The roof of the fault was Cretaceous limestone with karstic conduits communicating with the surface where a high-yield aquifer was present. An inrush of 900 l/sec along with eroded material lasted for 5 days until the real discharge of the faulted zone occurred with 4-6000 l/sec and a peak of 20000 l/sec (!) filling, additionally, the tunnel with more than 30000 m^3 of debris such as sands and limestone blocks. The works were called off for many months.

In Turkey (Erguvanli, K., 1974, *in* Calembert, 1979) during the construction of a 7 km tunnel north of Tarsus, a localized discharge of karstic waters in the order of 250 l/sec caused a considerable delay of the works. A number of cases in Switzerland are briefly described in the same report with the countermeasures being mainly drainage; freezing techniques or cases of isolation are also reported (tunnels of Mont Dore and of Simplon).

TUNNELLING IN LARGE LIMESTONE MASSIF

The potential of coming across large cavities at great depths (more than several hundreds of meters) seems to be limited. However, small active conduits have been reported as was the case in France described by Petiteville, P. and Toulemont, M., (1974 *in* Calembert, 1975) where such conduits were found at a depth of about 1000 m.

The encounter of such karstic features partially filled or empty, under a thick cover, has a strongly accidental character that no method of investigation from the surface can trace. Recently, the application of geophysical methods from the face has been developed but with little success due to inherent limitations and because results are influenced by the tunnelling equipment and the steel support. Anyway in all cases where geophysics can be applied, corroboration must follow through exploratory boring from the face. Thus, often the best solution is to simply probe ahead immediately if there is suspicion for the presence of such karstic features.

An insight to the karstic conditions inside big mountains was gained while crossing a large karstic mountain at great depth in central Greece (Marinos, 1992). This experience can be easily utilized for other big massifs, as for instance, those occurring around the Mediterranean or other undergone the same geological evolution.

The question regarding the prevailing state in the interior of the karstified mountains at great depths and far beyond the areas where springs appear is often open. Such an issue arose during the construction of the Giona tunnel for the Mornos-Athens water-supplying aqueduct concerning both construction and operation. The tunnel now traverses this mountain for a length of 14.6 km, parallel to the coast, at an altitude of 377 m, beneath a cover of 1700 m, and with the central section 14-20 km from the coast of the Corinthian Gulf, where the groundwaters of the mountain are discharged through big coastal springs (Fig. 2).

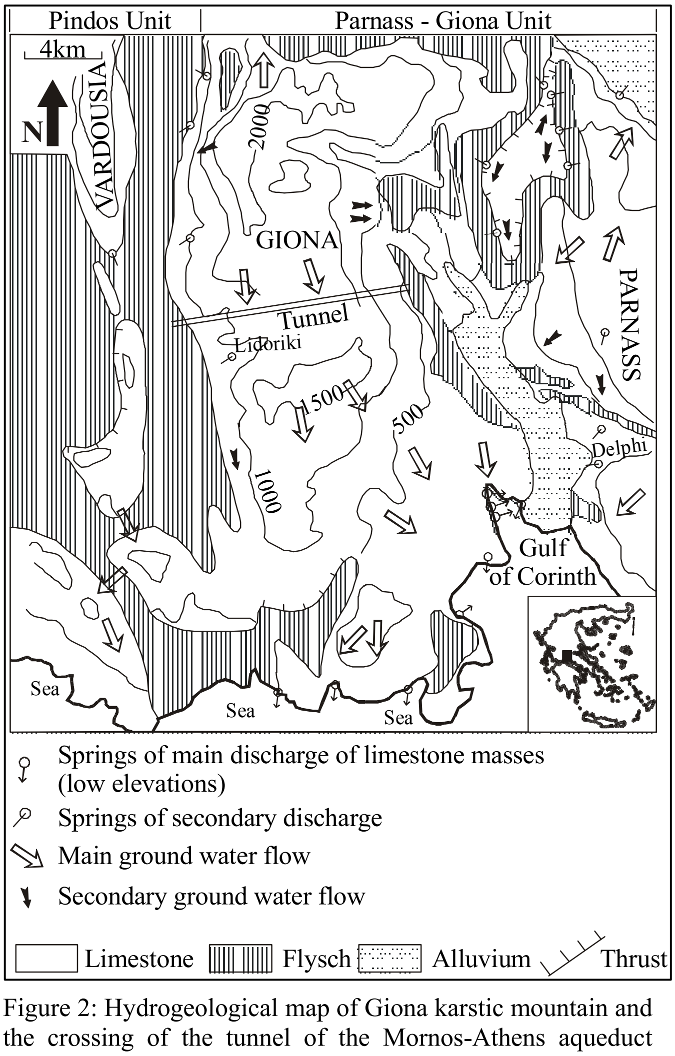


Figure 2: Hydrogeological map of Giona karstic mountain and the crossing of the tunnel of the Mornos-Athens aqueduct (Marinos, 1992)

At the beginning of the project, the intense karstification of the surface of the mountain and the drainage towards the low points of the coastline led the designers to a first hasty hypothesis that the tunnel would pass through karstic limestone, but more or less above the karstic water table due to the gentle hydraulic gradient expected for such a karstic environment (Fig. 3A). This is the case described by model C in Table 1. The tunnel was thus expected to be within the transfer (or conveyance) zone of groundwaters with high risk for sudden inflows during floods but with no permanent underground water. This water table would be considerably lower in areas of high permeability and of unobstructed discharge to the coast.

A few investigative drillings, although not deep enough, provided however some indications that the limestone could not be karstified at depth, but, merely, finely fissured. In such a case (model B in Table 1), the water table could lie considerably above the level of the tunnel and obviously with low-yield inflows in the tunnel (Fig. 3B).

Finally the karstic and hydrogeological conditions of the interior of the mountain appear more composite and to a certain extent, are a combination of models B and C (Fig. 4 and 5). The interior of the carbonate mountain does not appear karstified; karstification seems to stop at a depth of a few hundred meters and creates a karstic zone, which proceeds in stages parallel to the surface of the mountain. The paleogeographic development of the area, with gradual surface erosion and leveling due to successive faulting and changes of sea level, contributes to the formation of such an underground karstic geometry. Beneath the karstic zone, the limestones are not karstified but appear finely and tightly jointed, hence leading to low permeability.

The water table exhibits low gradients only at the karstic zone (in the lateral envelop of the mountain and behind the springs) but

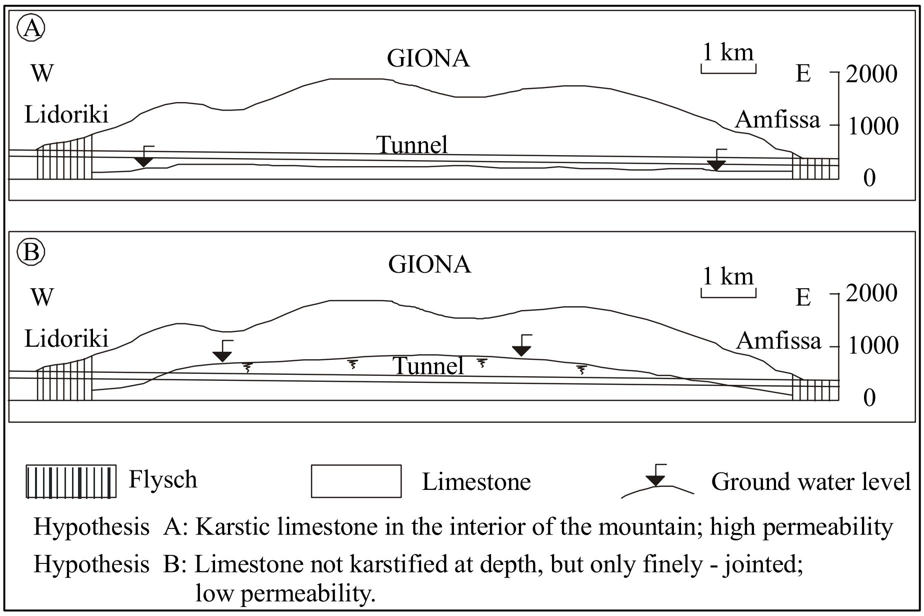


Figure 3: Schematic sections of two hypothesis of groundwater development in Giona Mountain, Central Greece.

the gradients become steeper towards the low permeability interior of the mountain. The classical concept of the karst base level does not apply except for the areas below these peripheral parts of the mountain and evidently the mountain remains non karstified in its central part.

The water table in the outer karstified parts of the mountain is below the position of the tunnel and rises above it in the non karstified central areas. In those areas, the limestone is of low to very low permeability and the flow can be thought of as that in a poor porous medium. Drainage in the tunnel is barely perceptible mainly in the form of "transpiration", wet sidewalls or drip flows.

However, a few deviations from the general regime of the mountain's interior were found in the form of very limited zones of high permeability. Throughout its length, for more than 11 km in the interior of the mountain, the tunnel crossed just two karstic conduits, which were developed most probably in fault zones (Fig. 4). These conduits constitute no more than an exception and do not change the general non karstic characteristics of the interior of the mountain. These barely wide conduits were crossed at 9.8 and 6.5 km from the western entrance of the tunnel. The voids were bridged by fill and concrete slabs to allow boring by the Tunnel Boring Machine in use. When the first conduit was crossed, water was released under pressure but then the discharge quickly declined to small amounts. The second conduit was partially filled with clay, sand and gravel without water, but with clear indications of underground flow. Following a heavy storm, a flood reached the tunnel with a delay of only 8 hours. The water drained away from the tunnel within about a week. The active hydrogeological role of these conduits as a zone of transfer of infiltrated waters to an underlying inundated section was verified when they could not drain the water discharged into them by the tunnel; given that it was the season of high rainfall, the water table was elevated close to the level of the tunnel in that area.

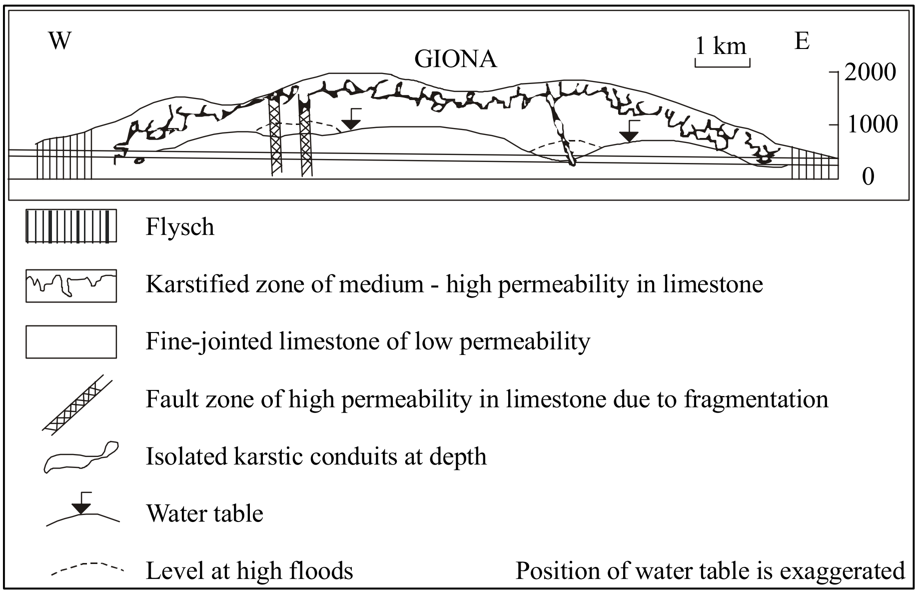


Figure 4: Underground hydraulic regime of Giona Mountain, Central Greece (Marinos, 1992).

These karstic tubes comprise axes of preferential isolated drainage according to a model similar of C or D (Table 1) but with a restricted extension inside the mountain. An additional result of their presence is a local significant lowering of the high, but low-yield water table prevailing inside the massif.

Given the information gathered during the tunnel construction, the hydrogeological description of the interior of the mountain is only complete if one takes also into account the presence and role of faults with or without a mylonitized zone. These fault zones do not bear karstic features or voids along their discontinuities, but few of them induced water problems, especially in the sections between 9.3 and 10 km from the East side (Fig. 4). In total, more than 400 l/sec of water entered the tunnel, 150 l/sec of which were contributed by a single fault through its fairly narrow mylonitic zone.

The water-bearing faults increase the underground hydraulic heterogeneity of the interior of the mountain. These faults are fed by the karstic and highly permeable portions of the surface of the mountain. Hence, according to the geometry of the faults and their discharge capacity, the resulting water column can maintain a significantly raised water table despite the high permeability of these zones. This column recharges the surrounding finely-jointed limestone during wet periods and drains it during the dry periods.

Leakage from this hydraulic tunnel (piezometric head of 80 m) was impossible for most part of the mountain and the water table applies a high hydrostatic load that the lining was designed to withstand. On the contrary, leakage from the tunnel was possible at the endmost parts the tunnel, where the karstic zone was crossed. In these parts, a tighter grouting program had to be applied, as the karstic water level lied lower than the tunnel.

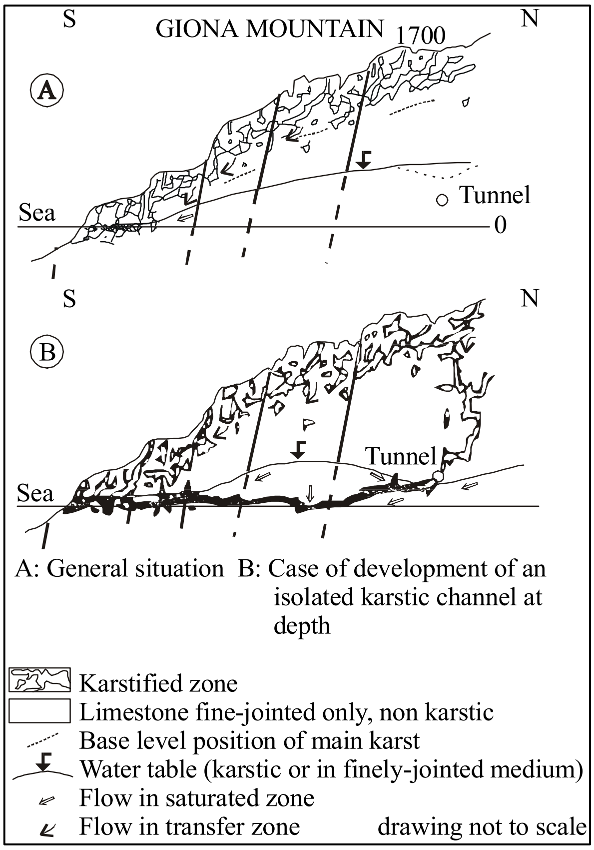


Figure 5: Underground hydraulic regime of two cross sections of Giona Mountain (Marinos, 1992). (A) Model B according to Table 1, (B) Model C according to Table 1.

CONFRONTING THE PROBLEM OF WATERS

Groundwater in tunnelling can be faced mainly with the following generally regarded operations (general information can be obtained from Anonymous, 1992 and Bauer, 1994):

- lowering groundwater level by controlled drainage or dewatering via pumping, thus reducing both head of water pressure and discharge into the tunnel
- grouting

Methods such as freezing, ground control by slurry, compressed air or earth pressure balance boring machines cannot be applied in highly permeable karstic limestone. Usually, drainage is more effective and often cheaper than any other operation. Predrainage prior to tunnel construction is probably the most commonly used water control method. The technique basically involves the lowering of the water table by drilling a series of wells or boreholes at either side of the projected tunnel. Drainage can be achieved from within the tunnel itself when dewatering from the surface is impossible. This can be done through drain holes from the face or from a long systematic drainage umbrella embracing the tunnel (Fig. 6), or even though the construction of small side pilot drainage galleries.

In the case of grouting in limestone, the primary goal is to reduce permeability. Modern practice is to drill a 360^{o} array of grouting holes forwarded subhorizontally, then blast out and seal a section of tunnel inside this completed grout curtain. This also largely deals with the hazard of catastrophic inrush, i.e. a flooded cavity should be first encountered by a narrow bore drill hole that can be sealed off quickly. Grouting anyhow is difficult in large openings or under high pressure of water.

Such an example is described by Ford and William (1992), "the cooling water intake tunnel for an atomic power station, in Ontario, was an 8m diameter tunnel extending 600 m from shore beneath Lake Huron. It followed a corallian limestone formation just below the lake bed. Grouting forward proceeded in 20 m sections and the tunnel was cut in 8 m sections i.e. there was 60% overlap of successive grout curtains. However a cavity was encountered that could not be grouted because it was too large. It was sealed off and the tunnel was then deflected around it without serious difficulty, but at substantial extra cost". It is obvious that this deflection is not always possible (e.g. for traffic tunnels). Thus most extended dewatering methods have to be applied.

Dewatering can have undesired side effects on adjacent properties, the tunnel itself and the environment, such as (see also Powers, 1985):

- ground settlement due to consolidation of compressible soils filling big karstic cavities up to the surface as an effect of increased effective stresses from water table lowering. Fortunately, such ground settlement cannot take place when limestones cover the ground as the rock is not compressible
- development of sinkholes
- depletion of adjacent groundwater and/or surface water supplies
- salt water intrusion
- expansion of contamination plumes
- release of contaminated water into the environment

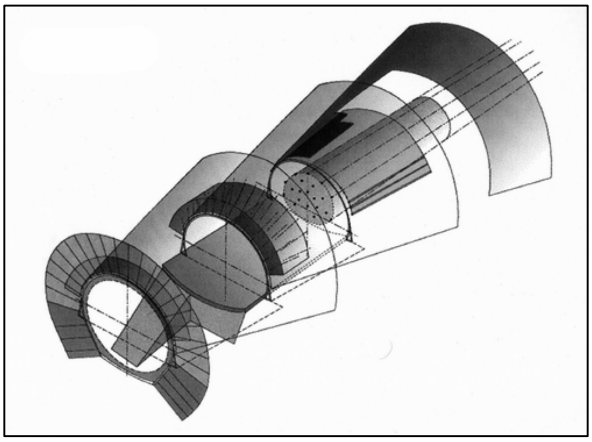


Figure 6: Driving a tunnel through an important water bearing zone with predrainage through embracing drainage umbrellas (sketch from "Geodata", Torino, personal communication).

THE CASE OF CONFRONTING KARSTIC GROUND WATER IN MINES

Grouting is not feasible in the extracting galleries of a mine or in an open cast exploitation. Here, a more elaborate strategy is to dewater the mine zone entirely, i.e. maintain a huge cone of depression around it for as long as the mine is worked.

Case from a mine in Poland

A good example of the method is provided by the development of lead/zinc mines at Olkusz, Poland (Wilk, *in* Ford and William, 1992). “The ores are contained in filled dolines and cavities in a dolomite paleokarst at a depth of 200 to 300 m below a plain of Quaternary sediments that is in hydrologic contact with the bedrocks. Potentially, this was a very hazardous situation. An area of 500 km^2 was surveyed about the potential mine. It contained 70 natural springs and 600 wells. A further 1700 exploration boreholes were drilled. Piezometers were installed in 300 wells and boreholes for carrying out pumping tests. From the latter it was estimated that 300 x 10^6 m^3 of groundwater would have to be pumped to establish the cone of depression for the mine. The cone was pumped via vertical wells plus drainage audits with high capacity pumps that were cut beneath each extraction level before ore extraction began. By these means, maximum local inrushes of water were held to 1.5 m^3/sec, i.e. within the capacity of the pumps”.

A case in Greece: prediction of groundwater table lowering for lignite mining and environmental implications

Extensive lignite deposits in the Ptolemais basin, in the Macedonia region of North Greece, are being exploited for electric power generation in thermal plants. The lignite-bearing horizons in the area are of about 50-70 meters thick and are covered by 100-200 meters of sterile overburden. Large-scale open-cast mining techniques are employed for the removal and disposal of the overburden and the excavation of the coal. Earth-removing operations are coupled with extensive groundwater table lowering in the alluvia of the basin using deep wells along the periphery of the mines.

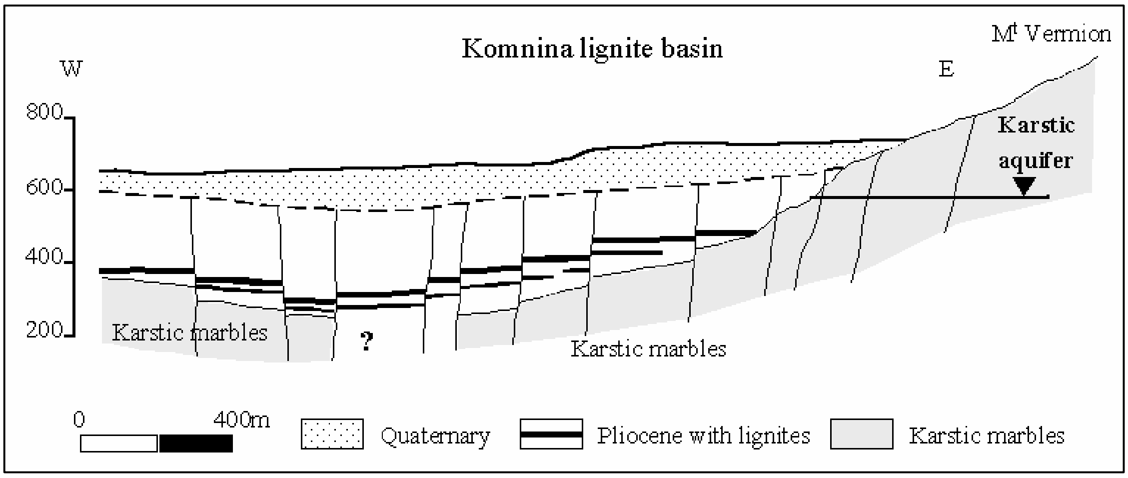


Figure 7: Typical East-West section of the eastern boundary of the Komnina basins, Ptolemais, Northern Greece (Kavvadas and Marinos, 1994).

The coal deposits are almost horizontally bedded, but a series of normal faults results in a progressive deepening of the beds and a corresponding increase of the thickness of the overburden towards the east-southeast rim of the basin (Fig. 7). Because of the continuously increasing demand for electric power in Greece, the Public Power Corporation, which owns and operates the lignite mines, plans to develop new mines in the Komnina Field, adjacent to the eastern boundary of the basin. Open cast coal exploitation in this field will require excavations down to 300 meters below the ground surface. The significant increase in the depth of the excavation is not the only challenge in planning the development of this new field. Mining operations will require lowering of the groundwater table by about 260 meters for the entire life of the field (estimated to be 10-15 years). In the planned Komnina Field, the increased depth of the lignite deposits and mainly the proximity of the field to the eastern boundary of the Ptolemais basin, which consists of highly karstified water-bearing marbles, make groundwater table lowering very costly. Furthermore, the effects of such an extensive operation will extend to a significant distance from the mines, altering the hydrological budget of a region, which is already stressed by over-exploitation for intensive use in irrigated agriculture and industry. More specifically, the long-term groundwater table lowering might also effect the hydrological budget of lake Vegoritis, some nine kilometers to the north of the Komnina Field, with possible consequences to its ecosystem (Fig. 8). It was thus necessary to study the nature of the karst aquifer along the eastern boundary of the Ptolemais basin, and assess the feasibility of dewatering and its effects on the hydrological budget of the region.

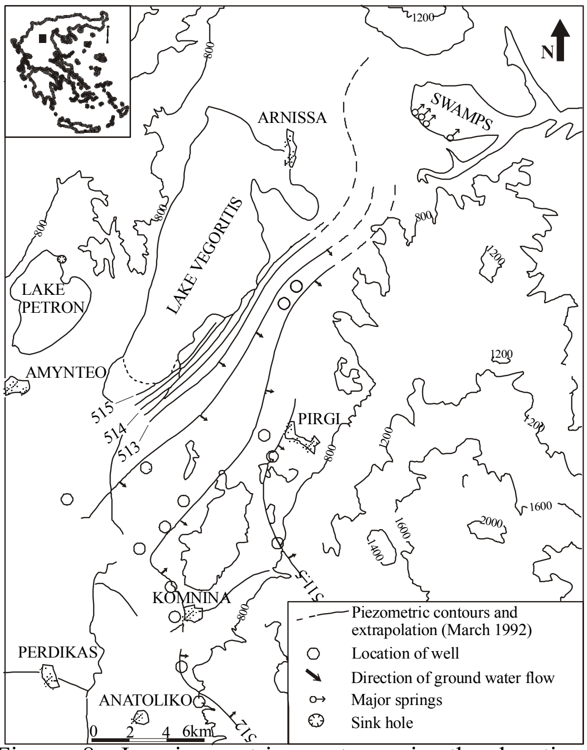


Figure 8: Iso-piezometric contours in the karstic aquifer to the east of the Komnina sub-basin in March, 1992 (Kavvadas and Marinos, 1994).

Hydrological and hydrogeological investigations in the well developed karstic marbles of western Mount Vermion revealed the existence of a single quite homogeneous unconfined aquifer having a very low hydraulic gradient (0.02-0.05%) towards the southeast and a high transimissivity (0.01-0.1 m^2/s) (Fig. 8). The high transmissivity of the aquifer poses severe problems to the feasibility of the required extensive and sustained groundwater table lowering.

A two-dimensional regional flow model based on the finite element method was developed and used to study the behavior of the aquifer (Kavvadas and Marinos, 1994). Model parameters were estimated from field measurements in the aquifer, experience with similar karstic aquifers elsewhere in Greece, as well as knowledge of the steady-state response of the aquifer. Darcy limitations were marginally accepted due to the homogeneity of the karstic net, the uniformity of the piezometric surface and the low hydraulic gradients. The predictions of the regional flow model show that for reasonable values of the aquifer transmissivity (0.01 to 0.1 m^2/s), the required groundwater table lowering cannot be achieved and maintained without extremely high cost (Fig. 9). It was also found that within a period of one to two years, the hydrological budget of lake Vegoritis will

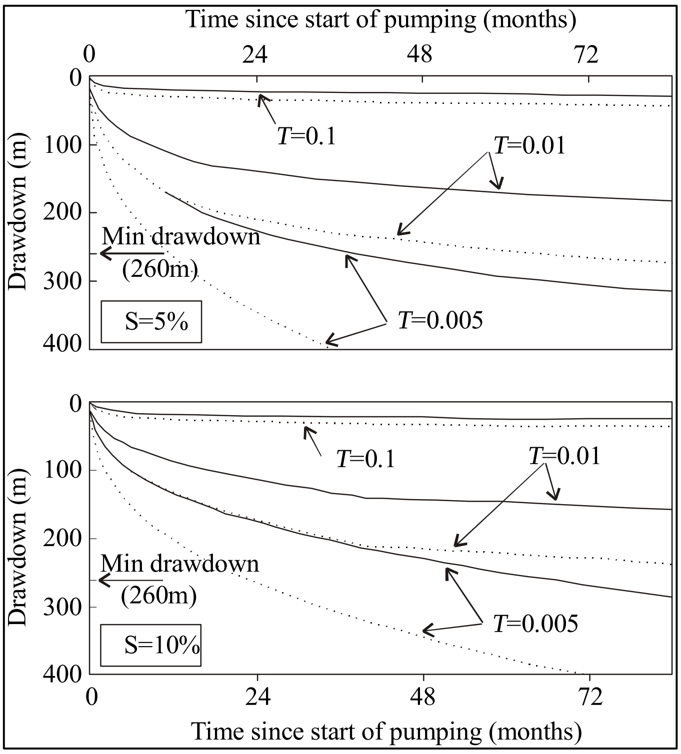


Figure 9: Predicted evolution with time of the average drawdown at the periphery of the mines, for various values of the transmissivity (T) of the aquifer. Pumping intensities: 10.000 m^3/h (continuous line), and 15.000 m^3/h (dotted line). Storativity: 5% (upper diagram), 10% (lower diagram).

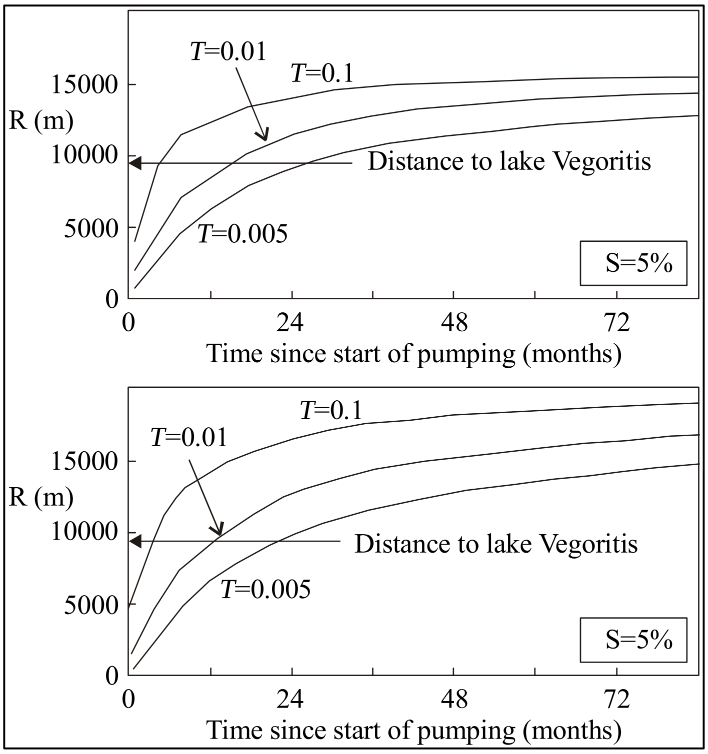
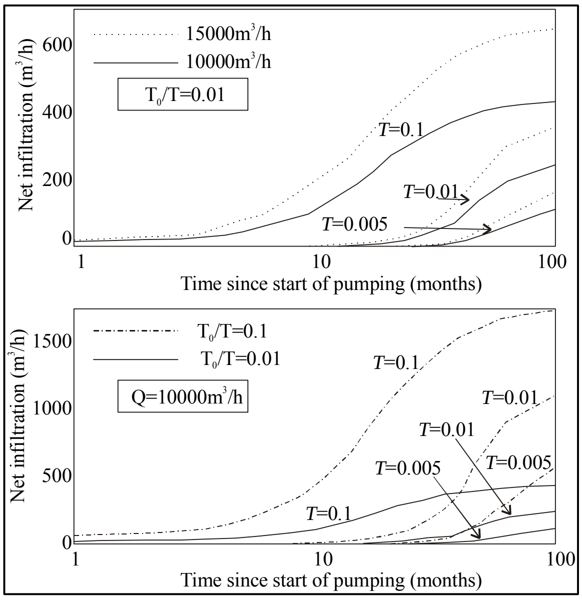


Figure 10: Predicted evolution with time of the average radius of of dewatering (defined as the distance at which drawdown is 0.5 m), for various values of the transmissivity (T). Storativity: 5%. Pumping intensities: 10.000 m^3/h (continuous line: upper diagram) and 15.000 m^3/h (dotted line: lower diagram).

Figure 11: Predicted infiltration rates from lake Vegoritis to the karstic aquifer as a function of the time since the beginning of pumping. Upper diagram: a zone of alluvial deposits having transmissivity $T_o = 0.01T$, exists at the interface between the lake and the aquifer. Pumping intensities at the Komnina field: 10.000 m^3/h (continuous line), and 15.000 m^3/h (dotted line). Lower diagram: a zone of alluvial deposits having transmissivity $T_o = 0.01T$ (continuous line) and $T_o = 0.1T$ (dashed – dotted line) exists at the interface between the lake and the aquifer. Pumping intensity at the Komnina field: 10.000 m^3/h.



be significantly affected by the dewatering, with induced losses from the lake being large, compared to their present estimated values (Fig. 10, 11). Thus large-scale dewatering schemes around the proposed Komnina field should be combined with artificial recharge of the lake with part of the water pumped at Komnina. It is thus concluded that the deep lignite-bearing horizons along the eastern boundary of the Ptolemais basin could be only partially exploited at present, and a zone of sufficient thickness has to be left intact along the margin of the basin to act as a seal between the karstic aquifer and the mine. However, the stability of the seal as well as the possibility of piping through sandy seams interbedded with lignite horizons would still require partial groundwater table lowering in the karstic aquifer to reduce the hydrostatic pressures acting on the seal.

GEOTECHNICAL ISSUES WHEN TUNNELLING IN LIMESTONE ROCK MASS

The rock mass itself

With the exception of the problem associated with the karstic characteristics, limestone and all other carbonate rocks in general, exhibit a good geotechnical behavior and a friendly tunnelling response. They exhibit reasonably good resistance to drilling or boring with reduced wear of excavation tools. The strength of a limestone rockmass can never reach low levels such as those of a squeezing ground, even when brecciated. Limestone breccias always exhibit good frictional values; however, support is sometime necessary with light steel sets or lattice girders, beyond rock bolts and shotcrete.

From another point of view, when the rock is at great depths or under high horizontal stresses, it cannot generate typical bursting instability, as is the case of hard rocks, since it is not a brittle material*. Any mild spalling problems in tunnels can be satisfactorily coped with rock bolting and reinforced shotcrete.

The case of voids and karstic caverns

The meeting of caverns and big karstic conduits may be associated with the following problems, often very difficult to overcome:

- bridging the void, if empty
- tunnelling through a geotechnically weak fill material
- confronting water inrush associated with mud flow if the void is water bearing and filled partially or totally with earth materials (as discussed earlier).

In the case of urban tunnels with a thin cover, the occurrence of these voids can effectively be investigated with a drilling program assisted by geophysical testing. In most shallow depths the georadar can give reliable information. In deeper levels cross-hole tomography could be the best choice. In tunnels close to the surface an associated risk is the collapse of an adjacent cavern after an earthquake; filling these voids prior to the completion of the tunnel is an additional task to be undertaken. In the case of deep tunnel through a mountain and given there are clear indications that such cavities are present, the only reliable method is probing ahead, as was previously mentioned.

Figure 12: Typical appearance of a small karstic void partially filled with clay and silt; Dodoni tunnel, northwestern Greece, 2000.

The Dodoni tunnel in northern Greece, with a length of 3.3 km and 12 m in diameter, is currently (fall – winter 2000) being driven in a limestone sequence with well developed bedding and possible local intercalations of siltstones or cherts a few cm or dm of thickness. The limestone encountered so far has behaved well and this behavior is expected to continue. However, significant overbreaks have occurred at some locations and these overbreaks were due to instability of the fill in karstic cavities (Fig. 12). Karstic solution features may indeed be observed in outcrops on the surface of the mountain ridge crossed by the tunnel under a cover of at least 100 m. These features indicate that karstic processes were active inside the limestone ridge.

Two major collapses occurred related to the presence of sinkholes at the surface with outcropping chimneys almost 100 m of height. The voids were filled with clayey material and pieces of broken rock and were prominently wet. The main collapse had a diameter of approximately 1.5 m in the tunnel and 3 m on the surface (Fig. 13), leading to 1200 m^3 of material falling into the tunnel.

In order to detect karstic cavities, pockets filled with soft and broken material, shear zones and gouge-filled faults, it was recommended that routine probe drilling ahead of the tunnel face should be carried out (Hoek and Marinos, 2000, experts' unpublished report to "Egnatia Highway S.A."). Typically, such probe holes are percussion drilled using the normal jumbo. Ideally, the probe hole should always be kept one tunnel diameter ahead of the advancing face and the most convenient way to achieve this is by drilling long holes (30 to 50 m) during maintainance shifts or at weekends. As in all karstic voids, because of the irregular and unpredictable shape and location of weak zones, it is recommended that at least three probe holes should be drilled from the face at 10, 12 and 2 o'clock positions. These holes are believed to have the highest probability of detecting the most dangerous zones. During

* In terms of mechanical properties typical bursting situation can usually be met in hard, strong and brittle rock, e.g. having an unconfined compressive strength higher than 100 MPa and a modulus of deformation greater than 4 GPa.

Figure 13: Collapse of the filling of a karstic chimney crossed by Dodoni Tunnel. The collapse outcropped on the surface about 100 m. over the tunnel.

drilling, a supervising geologist or engineer should be present and, together with the driller, should watch for rapid changes in drill penetration rates, nature of the chippings and color of the return drilling water. An experienced driller will usually be able to detect changes in the drill performance and to give a reliable prediction of the nature of the ground ahead of the face.

When a significant weak zone is detected additional probe holes should be drilled to define the extent and shape of the zone as accurately as possible. In exceptional cases, one or two cored holes may be required to determine the nature of the filling material.

As a general rule grouting of the filling material within the cavity is a primry consideration in order to improve its cohesive strength. However, it has to be realized that the effects of such grouting are highly unpredictable, depending on the nature of the filling materials.

The support measures to be used depend upon the nature and the extent of the weak zone. When a weak zone (e.g. a karstic cavity or a filled pocket of limited extent) is to be dealt with, the use of forepoles to bridge the cavity should be considered. These forepoles play an entirely different role from those used to pre-support the face in squeezing ground. Their function is to form a roof over the tunnel through which the weak filling material of the cavity cannot pass. Hence, depending on the volume of material to be supported, the forepoles should be reasonably light (say 75 mm diameter tubes) and they should be as closely spaced as possible. They should also be long enough to ensure that they are securely socketted in good limestone on either side of the cavity. The number of forepoles to be installed should be limited to the number required to form an effective barrier under the cavity. It is not necessary to implement a complete support system, with an extensive forepole umbrella and additional support measures, such as that used in squeezing ground.

When the probe drilling detects a continuous feature of significant size, the approach has to be quite different from that described above. In such a case, the rock mass on either side of the cavity will be most probably weaker than the surrounding limestone and the zone may be 10 m thick or more, depending on the orientation of the void. In such a case, it is prudent to implement the full forepoling solution, similar to that used in squeezing ground (Fig. 14).

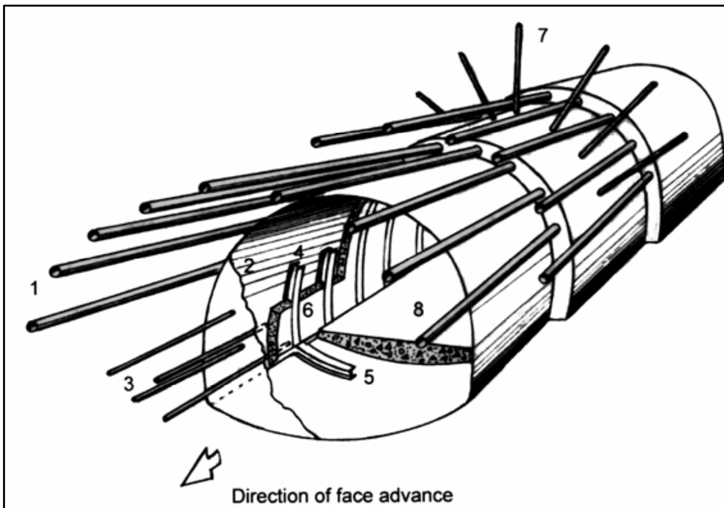


Figure 14: Full face excavation through weak ground under the protection of a forepole umbrella. The final concrete lining is not included in this figure (Hoek, 2000). The method can be applied in cases of large karstic caverns or large chimneys filled with cohesive soil under substantial load. Note that it is not always necessary to implement all the components shown in this figure.

1. Forepoles – typically 75 or 114 mm diameter pipes, 12 m long installed every 8 m to create a 4 m overlap between successive forepole umbrellas.
2. Shotcrete – applied immediately behind the face and to the face, in cases where face stability is a problem. Typically, this initial coat is 25 to 50 mm thick.
3. Grouted fiberglass dowels – Installed to reinforce the rock immediately ahead of the face. These dowels are usually 6 to 12 m long and are spaced on a 1 m x 1 m grid.
4. Steel sets – Installed as close to the face as possible and designed to support the forepole umbrella and the stresses acting on the tunnel.
5. Invert struts – Installed to control floor heave and to provide a footing for the steel sets.
6. Shotcrete – Typically steel fiber reinforced shotcrete applied as soon as possible to embed the steel sets to improve their lateral stability and also to create a structural lining.
7. Rockbolts as required. In very poor quality ground it may be necessary to use self-drilling rockbolts in which a disposable bit is used and is grouted into place with the bolt.
8. Invert lining – Either shotcrete or concrete can be used, depending upon the end use of the tunnel.

One further possibility needs to be considered and that is the case of a large empty karstic void. Such a void will generally require bridging and backfilling. The nature of the backfill will depend on the location of the void relative to perimeter of the tunnel. If water is associated with the void, drainage holes have to be foreseen as described earlier (see also Fig. 6)

Encountering a vertical karst channel, which is the most common case in the transfer zone of the aquifer (model C in Table 1), unpredictable concentrated water pressure may load the tunnel lining. In order to prevent possible damage, forced drainage of the channel towards lower elevations has to be secured (Fig. 15).

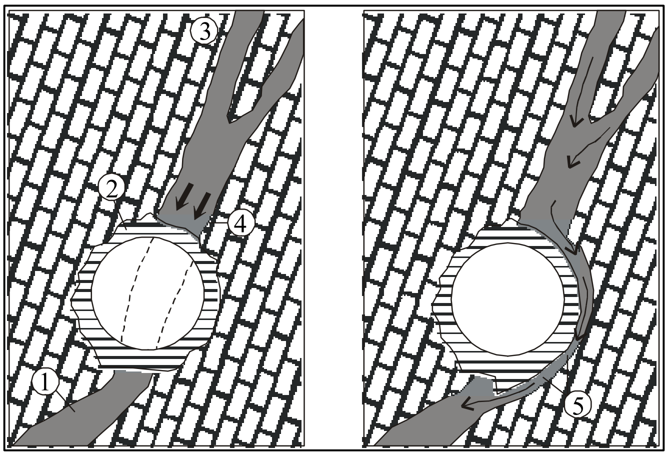


Figure. 15: Crossing a big karst conduit by tunnelling (Milanović, 2000). 1. Karst, 2. Tunnel, 3. Water level in the channel, 4. Lining under concentrated pressure, 5. Drainage around the tunnel pipe.

P. Milanović (2000) describes the way the tunnel Učka in Croatia faced the case of the approach of the tunnel to a large cavernous zone (Fig. 16).

"The total length of investigated karst channels is more than 1,300 m. The largest cave hole is 175 m long and 70 m wide while the cave hall close to the tunnel is 60 m long, 40 m wide and 55 m high. In the lower part of the cave system permanent groundwater flow is present. Summer flow rates vary from 10 to 30 l/s. After rainy storms the flow abruptly increases and exceed 1000 l/s (Hudec et al, 1980). Since the stability of the tunnel is endangered, artificial support is needed. Total plugging of the cave could not be carried out because of the existing permanent water flow through the cave. The use of reinforce concrete arch structures were rejected as complicated and very expensive. Since heavy mechanical equipment could not be used in the cave space, a simplified solution was applied to support the potentially unstable rock mass between the tunnel and the roof of the cave. The roof was supported by well compacted and stabilized fill (using water jet) which composed of fine-grained limestone aggregate, partially strengthened by addition of cement (150 kg of cement per 1 m^3 of fill). The strengthened zone forms the concrete "skin" around filled aggregate, and over the natural cone of limestone blocks. The average thickness of this concrete zone was 1 m. The space between strengthened aggregate and the cavern roof was filled by concrete. Along the entire contact zone between concrete and cave roof, 24 mm anchors have been installed. The entire supported structure was constructed without grouting treatment."

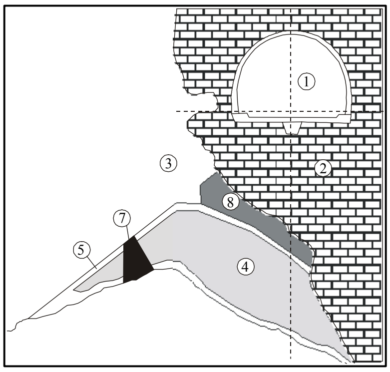


Figure 16: Driving the tunnel over a big karstic cavern. Longitudinal (a) and perpendicular (b) cross-sections (from Hudec et al, 1980). 1. Tunnel, 2. Limestone, 3. Cavern, 4. Fill – aggregate, 5. Fill – reinforced with cement, 6. Stone wall, 7. Retaining wall (reinforced fill), 8. Concrete, 9. Tunnell floor.

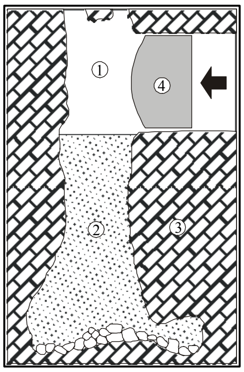


Figure 17: Cavern treatment by concrete filling for a TBM drive (Milanović, 2000). 1. Cavern; 2. Part of cavern filled by concrete; 3. Limestone; 4.TBM.

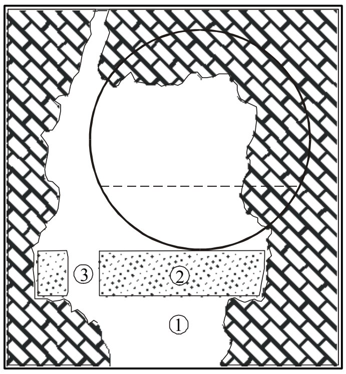


Figure 18: Cavern rehabilitation by bridging for a TBM drive (Milanović, 2000). 1. Cavern; 2. Concrete slab; 3. Aeration – drainage opening.

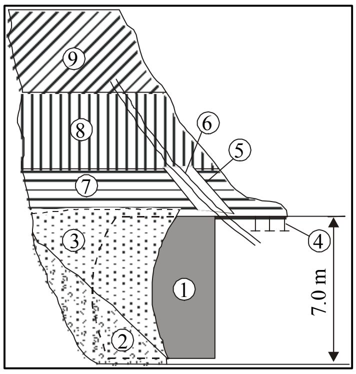


Figure 19: Filling of a cavern from inside the tunnel (Milanović, 2000). 1. TBM; 2. Cave clayey deposits; 3. Inert material – sand; 4. Tunnel support; 5. Shaft; 6. Pipe connected with concrete pump; 7, 8, 9. Stages of concreting.

When tunnel boring machines (TBM) are to be used, local realignment of the tunnel axis in order to avoid voids is not an option and usually a stoppage is imposed in order to backfill or bridge the void (Fig. 17, 18). If backfilling of the karstic cavern should be carried out from within the tunnel (Fig. 19) care should be given not to obstruct the cutter head with the concrete operations. When naturally filled, the voids have to be crossed by conventional tunnelling since the TBM, being usually of an open type, cannot bore the fill which could ravel through the cutter head of the machine.

SOME CONTRACTUAL CONSIDERATIONS

Successful groundwater control during underground construction is probably as dependent on the form of the contractual documents as it is on technical details of any particular method (Cuertin, 1989).

Indeed, groundwater control is always a high risk activity and the way that the risk is to be shared is regulated in the contractual documents. This sharing is obviously dependent on the results of a quality site investigation program and on the sound understanding of the most probable predominant karstic model.

Sharing risks associated with unpredictable events can substantially improve the success of a contract both in terms of cost as well as of schedule control. Where the overall financial and contractual arrangements permit, it may be possible for all parties to agree on some form of "Risk Sharing Package".

An example of a Risk Sharing Package for a tunnel in karstic limestone is given by Hoek and Palmieri (1989) and is presented in Table 2. In this particular project, the general geological conditions indicated a potential for sizeable karstic cavities. Site investigations had revealed some information, however the incremental investigation cost, which aimed at fully identifying them was not considered financially justifiable.

Hence, a set of limits was derived and agreed upon by both parties to the contract, based upon experience in the construction of similar tunnels. All of the indicators shown in Table 2 can be measured by simple quantitative site observations.

Tunnel length: 5020 m; finished diameter: 3.5 m; concrete lining to be provided. Geology: Miocene Limestone and Jurassic Dolomitic Limestone, cover 80 to 200 m.	
Description	*Extent (m)*
Massive to slightly jointed	2600
Closely jointed	1650
Weakly cemented	670
Fault zones, karstic cavities	100
Risk description	*Risk sharing*
Rock mass quality along the route	Since the main risk is associated with large karstic cavities and the average rock quality is fair to good, deviations from the assumed distribution can be included in the Contractor's risk.
Presence of groundwater	Inflows into the tunnel are within the Contractor's risk up to the following limits: a) 20 l/s at the tunnel face; b) 50 l/s at the tunnel portal; c) head of water not to exceed 50 m.
Karstic cavities	Limits to the Contractor's risk: a) cavity zone not exceeding the tunnel span, say 4 m; b) water inflows not exceeding 20 l/s and decreasing to less than 20 l/s at the unconfined state; c) delays caused by the occurrence of cavities do not exceed 30 days

Table 2: Example of a "Risk Sharing Package" for a tunnel in karstic limestone (Hoek and Palmieri, 1998, slightly simplified)

The provisions listed in Table 2 represent a reasonable risk package and it is probable that any international arbitrator would classify anything in excess of the limits defined in this package as Force Majeur conditions.

CONCLUSIONS

Tunnelling and mining in karst terrane require a thorough hydrogeological knowledge over a broader area. Lack of this knowledge may result to a design which will not be able to face problems or hazards that may occur during construction with probably dramatic consequences on the completion of the operation. Judgment and engineered solutions should always assist any decision at all stages during design and construction.

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