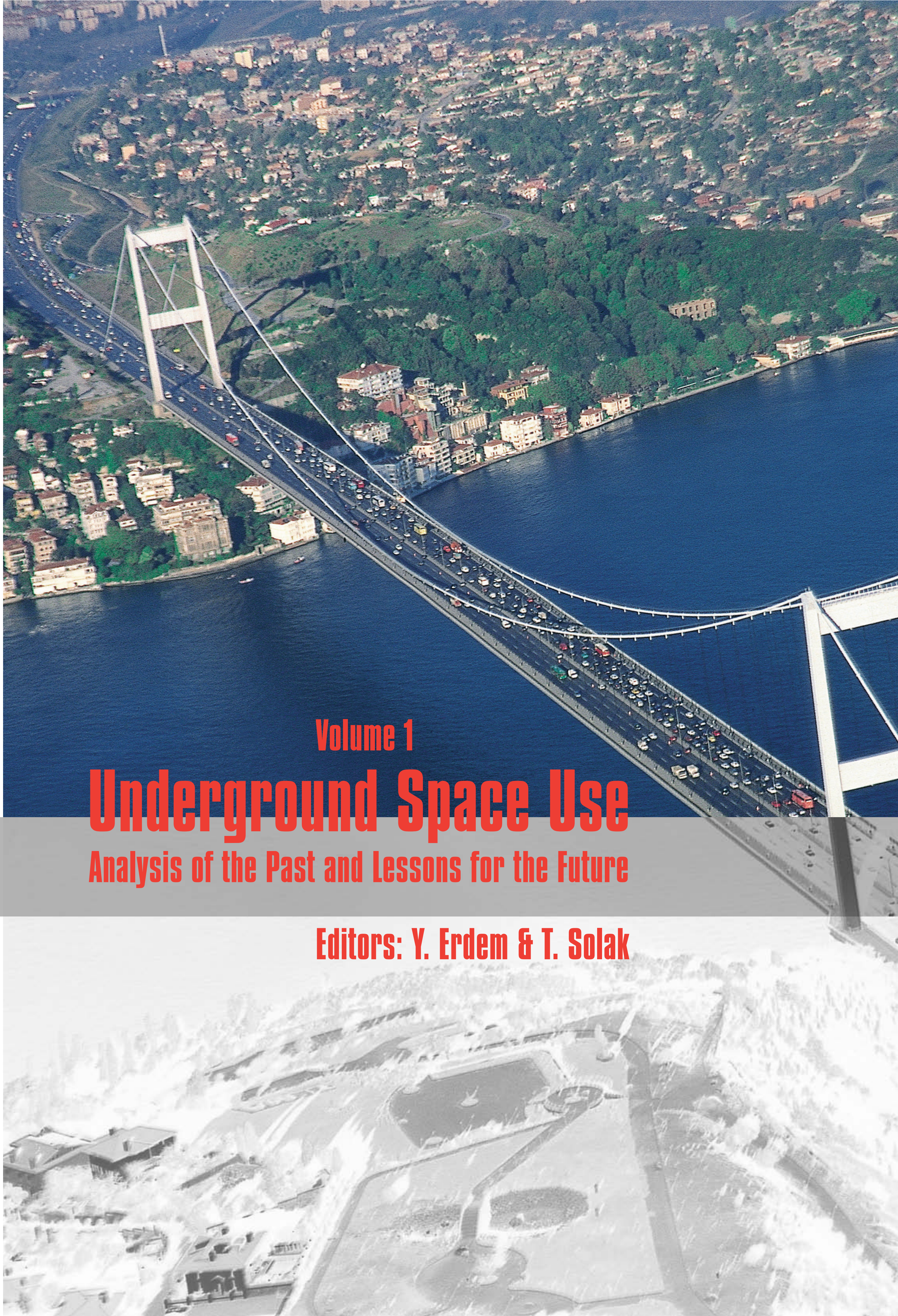

Volume 1

Underground Space Use

Analysis of the Past and Lessons for the Future

Editors: Y. Erdem & T. Solak

UNDERGROUND SPACE USE

ANALYSIS OF THE PAST AND LESSONS FOR THE FUTURE

PROCEEDINGS OF THE 31ST ITA-AITES WORLD TUNNEL CONGRESS, 7–12 MAY 2005, ISTANBUL, TURKEY

Underground Space Use

Analysis of the Past and Lessons for the Future

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Volume 1

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Preface

The ITA annual congresses and meetings have become premier events in the international tunnelling and underground construction calendar.

We are very proud and honoured to host the ITA-AITES 2005 WORLD TUNNEL CONGRESS & 31st GENERAL ASSEMBLY with the theme "Analysis of the Past and Lessons for the Future" in Istanbul, the eternal city, where for more than 2,500 years East and West have met to form a melting pot for a variety of races, cultures, religions and political systems.

The 2005 congress is organized by the Turkish Road Association and General Directorate of Highways, with support from the International Tunnelling Association (ITA).

Hundreds of potential delegates and interested parties from all over the world had been invited to submit their papers and contribute to the development of our field. The response to free paper submission was very impressive and many of the accepted papers contained new information and demonstrated significant developments that had taken place within our field. We would like to reaffirm our appreciation to all of our colleagues for their guidance, their willingness to share knowledge and for their continued support.

The abstract books and the CD-ROM present the full length papers accepted by the Scientific and Technical Committee of World Tunnel Congress 2005 – Istanbul. A total of 277 Abstracts were received and reviewed. The Scientific and Technical Committee and the editors selected 200 of these for publication. The published papers are divided into 13 groups.

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Tunnels, environment and public

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Positive impact of El Azhar road tunnels on environment, safety and tourism in Fatimid Cairo

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ABSTRACT: A two level steel bridge along El Azhar street was constructed to solve the problems of transport and traffic congestion in seventies of the last century. The bridge and also the very crowded street caused a bad appearance of the historic buildings in addition to air and noise pollutions which affect badly on the residents and the visitors. To avoid these negative impacts, the ministry of transport and National Authority for Tunnels constructed two road tunnels passing under El Azhar street and connecting Salah Salem road with the city center. It is concluded in this paper that these tunnels contribute in reducing the negative impacts of the traffic flow and resulted in positive impact on environment and tourism in Fatimid Cairo.

1 INTRODUCTION

The Fatimid Cairo was built one thousand years ago by El Moez Li-Dinellah to be the capital of Islamic Egypt. It has many historic islamic buildings and monuments and it is a very attractive area for tourists and traders as they can find the most famous antique markets in Khan El Khalily and many other places.The main route in the area is El Azhar street which passing through it.

The ministry of transport built a two level steel bridge along the street to solve the problems of transport and traffic congestion in seventies of the last century. The bridge and also the very crowded street caused a bad appearance of the historic buildings in addition to air and noise pollutions which affect badly on the residents and the visitors.

To avoid these negative impacts, the ministry of transport and National Authority for Tunnels constructed two road tunnels passing under El Azhar street and connecting Salah Salem road with the city center. It is concluded that these tunnels contribute in reducing the negative impacts of the traffic flow and resulted in positive impact on environment and tourism in Fatimid Cairo.

2 FATIMID CAIRO DESCRIPTION

2.1 *Natural environment*

2.1.1 *Climate*

It is 35.5°C of the average high temperature and 21.0°C of the average low temperature in summer at Cairo where locates at the south part of Nile delta. And it is 18.5°C and 7.1°C in winter respectively. Moreover, the precipitation in Egypt is little and also the rainfall is about 24.3 mm through the year of the Cairo City. Table 1 shows the weather in Cairo.

Table 1. Weather in Cairo.

Season	Temperature Max	Temperature Min	Humidity (%)	Rainfall (mm)
Winter	18.5	7.1	59	24.3
Summer	35.5	21.0	56	0.0

2.1.2 *Population*

The growth rate of population in last 13 years was 13% which shows average rate of 1.3% per year. Greater Cairo has a population of 17 millions, with area of 7000 ha.

2.2 *Establishment of Fatimid Cairo*

When the Fatimid region (969–1171) settled in Cairo under the leadership of El Moez Li-Dinellah it was called “Cairo of El Moez” because they decorated its four suburbs with luxurious buildings, delightful spots and gardens. This increased its delightfulness and beauty. It was the settlement of rulers and princes, where they had built about 360 historic islamic buildings.

The Al-Azhar Mosque (the most blooming), established in 972 (361 H) in a porticoed style shortly after

Figure 1. Photo of Fatimid Cairo.

the founding of Cairo itself, was originally designed by the Fatimid general Jawhar El-Sequili (Gawhara Qunqubay, Gawhar al-Sakkaly) and built on the orders of Caliph Muezz Li-Din Allah. Located in the center of an area teaming with the most beautiful Islamic monuments from the 10th century, it was called "Al-Azhar" after Fatama al-Zahraa, daughter of the Prophet Mohamed (Peace and Prayers Be Upon Him). Figure 1 shows a photo of Fatimid Cairo.

It imitated both the Amr Ibn El-As and Ibn Tulun Mosques. The first Fatimid monument in Egypt, the Azhar was both a meeting place for Shi'a students and through the centuries, it has remained a focal point of the famous university which has grown up around it. It was under Yaqoub Ibn Cals that the mosque became a teaching institute. This is the oldest university in the world, where the first lecture was delivered in 975 AD.

Today the university built around the Mosque is the most prestigious of Muslim schools, and its students are highly esteemed for their traditional training. While ten thousand students once studied here, today the university classes are conducted in adjacent buildings and the Mosque is reserved for prayer. In addition to the religious studies, modern schools of medicine, science and foreign languages have also been added.

Architecturally, the mosque is a palimpsest of all styles and influences that have passed through Egypt, with a large part of it having been renovated by Abdarrahman Khesheda. There are five very fine minarets with small balconies and intricately carved columns. It has six entrances, with the main entrance being the 18th Century Bab El Muzayini (barber's gate), where students were once shaved. This gate leads into a small courtyard and then into the Aqbaughawiya Medrasa to the left, which was built in 1340 and serves as a library. On the right is the Taybarsiya Medersa built in 1310 which has a very fine mihrab.

The Qaitbay Entrance was built in 1469 and has a minaret built atop. Inside is a large courtyard that is 275 by 112 feet which is surrounded with porticos supported by over three hundred marble columns of ancient origin. To the east is the prayer hall which is larger than the courtyard and has several rows of columns. The Kufic inscription on the interior of the mihrab is original, though the mihrab has been modified several times, and behind is a hall added in 1753 by Abd el-Rahman Katkhuda. At the northern end is the tomb medersa of Jawhar El-Sequili. Nowadays, the ministry of culture renewals all these ancient buildings which are lying in the area from north of Gamaleya to Bab El Wazir in the south and from Salah Salem road at the east to port said at the west.

Table 2. Number of tourist visited Egypt (Unit: thousand people, %).

	Year					
Nationality	1992	1993	1994	1995	1996	1997
All nationalities						
Number	3207	2508	2582	3133	3896	3961
Percent	100	100	100	100	100	100
Arabs						
Number	1103	922	932	823	897	967
Percent	34.4	36.8	36.1	26.3	23.0	24.4
Europeans						
Number	1555	1082	1030	1515	2021	2120
Percent	48.5	43.1	40.0	48.3	51.9	53.1
Americans						
Number	224	188	182	229	259	256
Percent	7.0	7.5	7.0	7.3	6.6	6.5
Others						
Number	325	316	438	566	719	636
Percent	10.1	12.6	16.9	18.1	18.5	16.0

As shown in Table 2, the number of tourist visited Egypt has high increase of 60% between 1993 and 1997.

3 URBAN TRANSPORT IN CAIRO

Cairo City is the capital of Egyptional country, and the biggest city in the African Continent which has 17 millions people. As a public transport system of Cairo city, it is composed of metros, buses mini buses, micro-buses, water buses, tram-metros, etc.

Due to economic development in Cairo, population has concentrated into the city, motor cars have rapidly increased and a choronic traffic jam has often occurred at major junctions in recent years. Such increase of traffic cars has not only brought the air and noise pollution, but also obstruction of economic activities and expansions.

On the other hand, the government agencies concerned started the construction of the underground metros as a drastical urban transport countermeasures in Cairo.

3.1 *Negative impact of transport modes*

In-movement licensed vehicles in Cairo in the end of year 1999 are shown in Table 3.

Table 3. Licensed vehicles.

Type of vehicle	No. of vehicles
Private cars	515402
Taxi	63445
Buses	
Public	5116
Private	5080
Tourism	4717
School	1387
Lorry & truck	104098
Motor-cycles	98327
Others	66163
Total	863735

Figure 2. A photo of El Azhar bridge.

Based on several studies of the transport and facilities, transport problems were identified in Fatimid Cairo and are summarized as follows:

1) Congestion Roads caused by large numbers of vehicles due to high demand for the transport service in peak hours.
2) Crowded modes of transport especially on public sector modes for some routes passing through crowded districts due to insufficient supply against high demand.
3) Low degree of efficiency and comfort on some public modes.
4) Unnecessary fuel consumption caused by low speed traveling.
5) Long duration of trip time especially on peak hours.
6) Air and noise pollutions.

The studies which had been carried out in seventies of the last century agreed that the main solution to meet the requirements of urban transport inside Fatimid Cairo is to build a steel bridge of two levels in order to connect El Darrassa in the East of El Azhar street with the city center of Cairo, in addition to keeping the on ground surface traffic. The bridge was built in the beginning of eighties passing through El Azhar street, and it transfers the traffic flow which is coming from Salah Salem road to El Opera Square and also in the opposite direction. Figure 2 shows a photo of El Azhar bridge.

The bridge causes many negative impacts on the historic Fatimid area which can be summarized in the following items:

- Air pollution with carbon monoxide and ozone.
- Noise pollution.
- Bad appearance and prohibiting sight seeing.

3.1.1 *Air pollution*

Carbon monoxide results from incomplete combustion of fuel and is emitted directly from vehicle tailpipes. Incomplete combustion is most likely to occur at low air-to-fuel ratios in the engine. These conditions are common during vehicle starting when air supply is restricted ("choked"), when cars are not tuned properly, and at altitude, where "thin" air effectively reduces the amount of oxygen available for combustion (except in cars that are designed or adjusted to compensate for altitude).

Nationwide, two-thirds of the carbon monoxide emissions come from transportation sources, with the largest contribution coming from highway motor vehicles. In urban areas, the motor vehicle contribution to carbon monoxide pollution can exceed 90.

Ozone is a form of molecular oxygen that consists of three oxygen atoms linked together. Ozone in the upper atmosphere (the "ozone layer") occurs naturally and protects life on earth by filtering out ultraviolet radiation from the sun. But ozone at ground level is a noxious pollutant.

Ozone is a severe irritant. It is responsible for the choking, coughing, and stinging eyes associated with smog. Ozone damages lung tissue, aggravates respiratory disease, and makes people more susceptible to respiratory infections. Children are especially vulnerable to ozone's harmful effects, as are adults with existing disease. But even otherwise healthy individuals may experience impaired health from breathing ozone-polluted air.

Ozone is not emitted directly but is formed in the atmosphere through a complex set of chemical reactions involving hydrocarbons, oxides of nitrogen, and sunlight. The rate at which the reactions proceed is related to both temperature and intensity of the sunlight. Because of this, problematic ozone levels occur most frequently on hot summer afternoons.

Hydrocarbons and nitrogen oxides come from a great variety of industrial and combustion processes. In typical urban areas, at least half of those pollutants come from cars, buses and trucks.

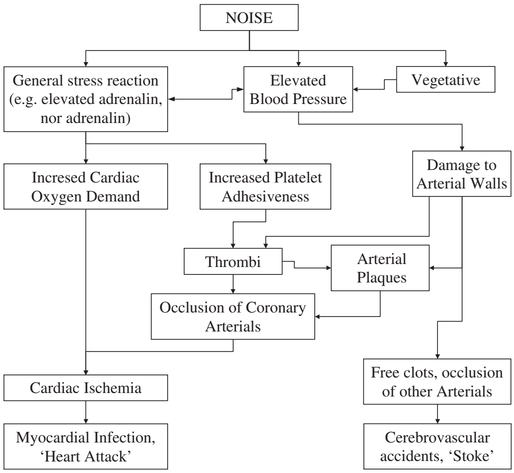

Figure 3. Effect of noise on human body.

3.1.2 *Noise pollution*

People who are exposed to high noise levels for short discontinuous periods may lose part of their hearing for some period, but those who are exposed to high noise levels for long continuous periods may lose part or all hearing abilities. Headache, stress and nervous instability may happen to the human body during the period of hearing loss.

The physiological effects are classified to short and long terms such as fatigue, headache, startle response, muscle tension response, respiratory reflexes, heart, circulation responses, and damage in the heart and brain as shown in Figure 3.

As El Azhar bridge is formed from steel sections, structure borne noise generated due to traffic flow is higher than standard regulations. It was found that air borne and structure borne noise levels are ranging 60 to 80 dB.

3.2 *Necessity of El Azhar road tunnels construction*

Within the framework of an overall revival and development plan for the glorious touristic area. The project of El Azhar road Tunnels has been proposed. The main objectives of this project are:

1) Providing a safe and efficient underground transport mean.
2) Creating a vehicular free surface for tourism and pedestrian purposes.
3) Reduction of air and noise pollution on the ground surface of El Azhar street.

To achieve the above mensioned objectives the ministry of transport and The National Authority for tunnels decided to construct two tunnels under

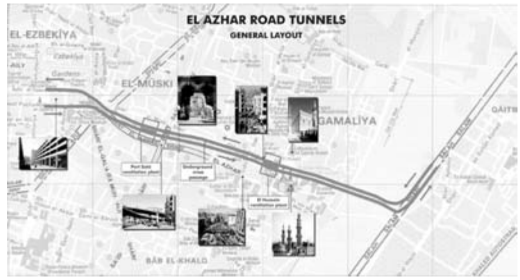

Figure 4. Outline of El Azhar tunnels.

Figure 5. Tunneling boring machine.

El Azhar street. The north tunnel transferes the traffic flow from Salah Salem road to El Opera square and the south tunnel for the other direction as shown in Figure 4. The length of each tunnel is about 2.65 km.

- *Alignment constraints and difficulties*

1) Passing underneath the existing main sewer tunnel, at a depth of 17 m below ground level in Port Said Street.
2) Avoiding any damage to the existing old buildings and interference with the piled foundations of an existing bridge in El Azhar Street.
3) Avoiding and/or diverting existing utilities.

- *Tunnel cross section*

The bored tunnel section was constructed using two bentonite slurry shield tunnel boring machines which shown in Figure 5. Each of these was 9.44 m in diameter and 66 m long, and articulated in order to enable them to follow the designed alignment of the tunnels where each tunnel should have 2 traffic lanes per one direction.

- *Ventilation system*

In addition to the tunnels, the project contains 4 ventilation stations, where they are working for the two tunnels. Every station contains fans for drawing and

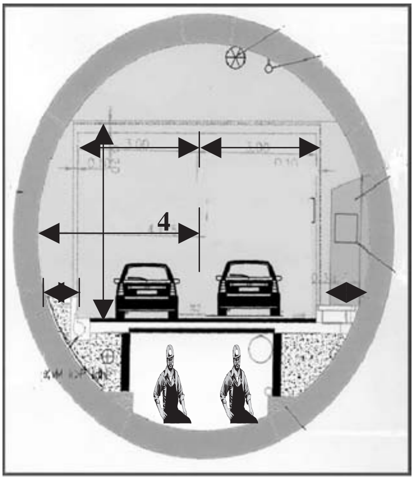

Figure 6. Tunnel cross section.

Figure 7. El Hossin ventilation station.

entering pure air and other fans for exhausting air. Figures 7 & 8 show El Hossin and Portsaid ventilation stations respectively.

- *Safety measures*

The choice of tunnel concept for traffic flow has been much attention. Two alternative tunnel concepts can be discussed in a safety perspective:

1) Single bored tunnel for two opposite traffic directions.
2) Two bored tunnels where every tunnel has a separate direction.

From a fire safety point of view it has been argued that twin bored tunnels are safer than single bored tunnels and it is preferable to distribute the emmisions of vehicles on large area. Also, construction of two tunnels reduced the rate of accedents in comparison with a single tunnel, especially that the tunnels are connected with 3 escape routes for cars and 5 escapes for transferring people in-between the tunnels and to the street level. The followings safety measures have been taken into consideration and implemented:

Figure 8. Portsaid ventilation station.

1) CCTV coverage, by remotely controlled color cameras, along the whole length of the tunnels and their approaches.
2) Controlled lighting to ensure that driver's sight is not affected by the varying levels of light from the surface to the interior of the tunnels.
3) Fire Detection instruments, along the tunnels, to detect smoke and excessive heat.
4) Fire Extinguishing Systems including pressurized fire mains, fire hydrants, hose reels and unit fire extinguishers containing powder and CO_2.
5) Air quality monitoring instruments, to check the degree of noxious fumes and alarms which will alert the operators to any problems with the ventilation equipment.
6) Emergency telephones.
7) Four dewatering pump stations, to prevent flooding of the tunnels.
8) Standby generators in case of loss of the main power supply.
9) Continuously manned Operations Control Rooms.
10) Emergency escapes every 100 m along the tunnels to transfer people from road level to the underpass where they can escape to the street level.

4 CONCLUSION

The urban traffic in Fatimid Cairo, which boasts its history of more than one thousand year, has always been congested on the ground of its ancient history. The streets in the city have changed to be in a torrent of automobiles owing to an overflow of vehicles by a runaway motorization in recent years. There is no doubt that building of El Azhar tunnels is contributing in solving many transport and traffic bad effects, thus it helps in protecting and keeping the historic monuments and buildings in the Fatimid Cairo. The positive impacts of building the tunnels can be concluded as follows:

1) It was necessary to build El Azhar road tunnels to reduce the trip time from half an hour to 4 minutes, and to avoid traffic jams.
2) The tunnels contribute in reducing air and noise pollutions by about 60% in case of removing the steel bridge.
3) The tunnels are the alternative of El Azhar bridge which should be removed to avoid the bad appearance in the historic Fatimid Cairo.
4) The tunnels have been constructed and equipped with very modern safety Aspects.

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Why did the hydropower industry go underground?

Einar Broch
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ABSTRACT: In Norway more than 99% of a total annual production of 125 TWh of electric energy is generated from hydropower. 4000 km of tunnels has been excavated for this purpose, and the country has 200 underground powerhouses. Special design concepts have over the years been developed related to this massive use of the underground. One such speciality is the unlined, high-pressure tunnels and shafts. Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. This paper will give a brief introduction to these and other solutions, and explain the advantages of utilizing the underground to its fullest possible extent for hydropower projects.

1 INTRODUCTION

Topographical and geological conditions in Norway are favourable for the development of hydroelectric energy. The rocks are of Precambrian and Paleozoic age, and although there is a wide variety of rock types, highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rocks.

More than 99% of a total annual production of 125 TWh of electric energy is generated from hydropower. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses are predominant. In fact, of the world's 500 underground powerhouses almost one-half, i.e. 200, are located in Norway. Another proof that the Norwegian electricity industry is an "underground industry" is that it today has more than 4000 km of tunnels. During the period 1960–90 an average of 100 km of tunnels was excavated every year.

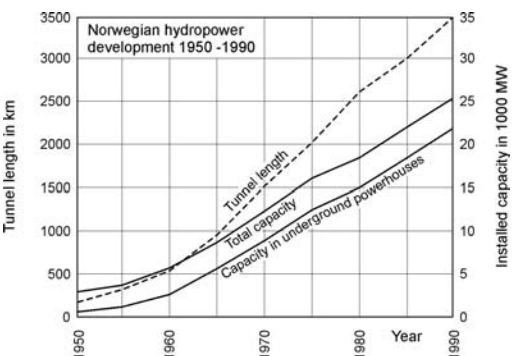

Figure 1. The development of Norwegian hydroelectric power capacity and accumulated length of tunnels excavated for the period 1950–90.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience has been gained. This experience has been of great importance for the general development of tunnelling technology, and not least for the use of the underground. The many underground powerhouses excavated in rock masses of varying quality are to a large extent the forerunners for the varied use of rock caverns which we find all around the world today.

Also, special techniques and design concepts have over the years been developed by the hydropower industry. One such Norwegian speciality is the unlined, high-pressure tunnels and shafts. Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. These specialities are described in further detail in Broch (2002).

Most of the Norwegian hydropower tunnels have only 2–4% concrete or shotcrete lining. Only in a few cases has it been necessary to increase this to 40–60%. The low percentage of lining is due not only to favourable tunnelling conditions. It is first and foremost the consequence of a support philosophy which accepts some falling rocks during the operation period of a water tunnel. A reasonable number of rock fragments spread out along the headrace or tailrace tunnel will not disturb the operation of the hydro power station as long as a rock trap is located at the downstream end of the headrace tunnel. Serious collapses or local blockages of the tunnels must, of course, be prevented by local use of heavy support or concrete lining when needed.

2 EARLY REASONS FOR GOING UNDERGROUND

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. At that time the traditional design was to bring the water down from the intake reservoir or the downstream end of the headrace tunnel to the powerhouse through a steel penstock. Both the penstock and the powerhouse were above ground structures as shown in Figure 2.

With the lack of steel for a penstock, the obvious alternative was to try to bring the water as close to the powerhouse as possible through a tunnel or a shaft. As a result, four Norwegian hydropower stations with unlined pressure shafts were put into operation during the years 1919–21. The water heads varied from 72 to 152 m. One (Skar) was a complete failure due to too low overburden of rock, only 22 m rock cover where the water head was 116 m. One (Toklev) has operated without any problems ever since it was first commissioned.

The Svelgen hydropower station, with a water head of 152 m, had some minor leakage during the first filling. A short section of the shaft was lined with concrete and grouted with cement. Since then the shaft has operated without problems. The fourth station, Herlandsfoss, had a 175 m long, horizontal high-pressure tunnel with water head of 136 m. Leakage occurred in an area of low overburden, 35–40 m, and the short penstock had to be extended through the whole tunnel to the foot of the inclined pressure shaft. The shaft itself had no leakage. Further details in Broch (1982).

Although three out of four pressure shafts constructed around 1920 were operating successfully after some initial problems had been solved, it took almost 40 years for the record of 152 m of water head in unlined rock at Svelgen to be beaten. Through 1958, nine more unlined pressure shafts were constructed, but all had water heads below 100 m. Until around 1950 the above-ground powerhouse with penstock was the conventional layout for hydropower plants as demonstrated in Figure 2.

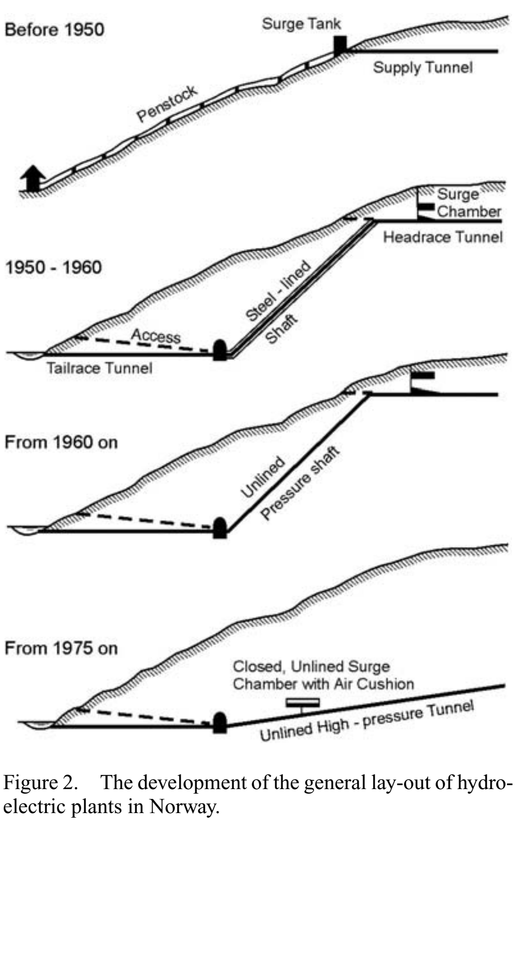

Figure 2. The development of the general lay-out of hydroelectric plants in Norway.

3 DEVELOPMENT AFTER 1945

3.1 *Underground powerhouses*

In a few early cases, underground location of a powerhouse was chosen as the only possible option (Bjørkåsen, 1921). During and after the Second World War, the underground was given preference out of considerations to wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent lowering of the costs, underground location came to be the most economic solution. This also tied in with the development of concrete lined, and later unlined pressure shafts and tunnels, to give the designer a freedom of layout quite independent of the surface topography.

Except for small and mini-hydropower stations, underground location of the powerhouse is now chosen whenever sufficient rock cover is available. Frequently the overall project layout requires the powerhouse to be placed under very deep rock cover where rock stresses may be substantial. This requires an investigation of the stress condition in advance for finding the most favourable orientation of the cavern and the optimum location, orientation and shape of ancillary tunnels and caverns.

In the early powerhouse caverns the rock support of the ceiling was limited to rock bolts. To safeguard against rockfalls, a 25–30 cm thick arch of in-situ concrete was placed some distance below the ceiling, see Figure 3.

In poor rock masses, the ceiling was often reinforced by an arch of concrete in contact with the rock. In the latter case a light arch ceiling would be suspended below the roof arch to improve appearance and to intercept any water leakage.

The present-day solution prescribes systematic bolting of the rock ceiling immediately after excavation of the top heading, followed by fibre-reinforced shotcrete from 70 to 150 mm in thickness, according to rock quality. It is also common practice to install deeply bolted girders for the powerhouse crane right after the excavation of the top heading, see Figure 4.

In this way the crane can be installed and be available early for concrete work and installation of spiral cases etc. without having to wait for concrete structures to be built up from the floor level. If needed, the crane girders may be provided additional support later on by columns, cast in place before handling the heaviest installation loads.

In the most common layout the transformer hall is located parallel to the main hall, at a sufficient distance for rock support, and a transport tunnel is utilised as tailwater surge chamber, but other solutions have also been used, see Figure 5.

3.2 *Unlined high pressure tunnels and shafts*

When the hydropower industry for safety reasons went underground in the early 1950s, they brought the steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. During the period 1950–65, a total of 36 steel-lined shafts with heads varying from 50 to 967 m (with an average of 310 m) were constructed.

The new record shaft of 286 m at Tafjord K3, which was put into operation successfully in 1958, gave the industry new confidence in unlined shafts. As Figure 6 shows, new unlined shafts were constructed in the early 1960s and since 1965 unlined pressure shafts have been the conventional solution. Today more than 80 unlined high-pressure shafts or tunnels with water heads above 150 m are successfully operating in Norway, the highest head being almost 1000 m. Figure 6 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

The confidence in the tightness of unlined rock masses increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant. This innovation in surge chamber design is described in detail by Rathe (1975). The bottom sketch in Figure 2 shows how the new design influences the general layout of a hydropower plant. The steeply inclined pressure shaft, normally at 45°, is replaced by a slightly inclined tunnel, 1:10–1:15.

Instead of the conventional vented surge chamber near the top of the pressure shaft, a closed chamber is excavated somewhere along the high-pressure tunnel,

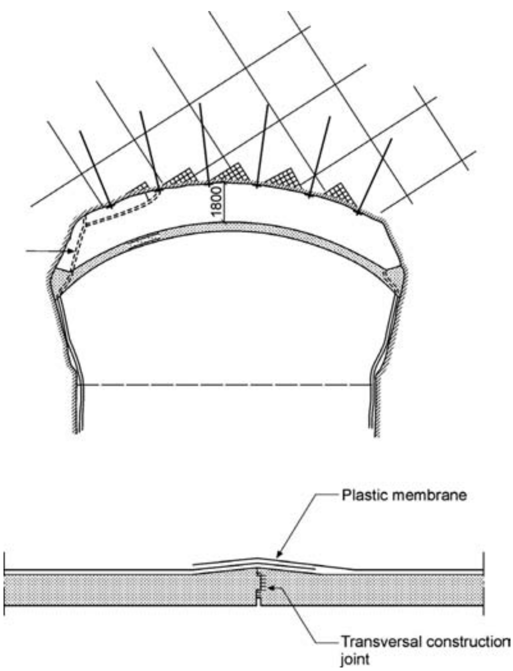

Figure 3. Typical design for a free span concrete arch.

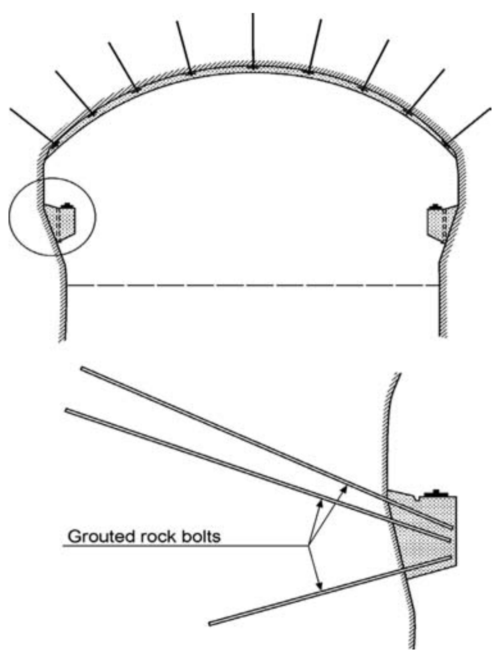

Figure 4. Steelfibre reinforced shotcrete arch and rock bolt supported crane beam.

preferably not too far from the powerhouse. After the tunnel system is filled with water, compressed air is pumped into the surge chamber. This compressed air acts as a cushion to reduce the water hammer effect on the hydraulic machinery and the waterways, and also ensures the stability of the hydraulic system.

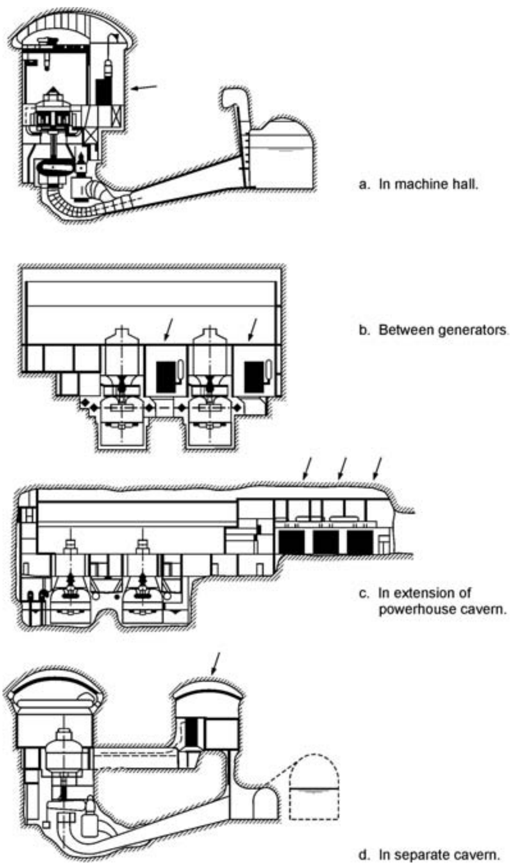

Figure 5. Common transformer locations.

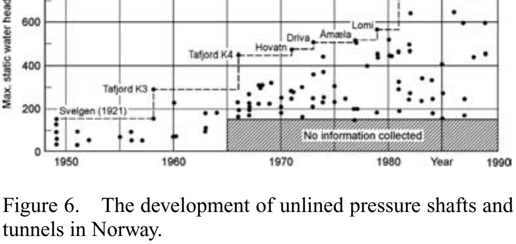

Figure 6. The development of unlined pressure shafts and tunnels in Norway.

In the years before 1970 different "rule of thumbs" were used for the planning and design of unlined pressure shafts in Norway. With new and stronger computers a new design tool was taken into use in 1971–72. This, as well as the "rule of thumbs", are described in detail in Broch (1982). It is based on the use of computerised Finite Element Models (FEM) and the concept that nowhere along an unlined pressure shaft or tunnel should the internal water pressure exceed the minor principal stress in the surrounding rock mass.

Very briefly, the FEM models are based on plain strain analysis. Horizontal stresses (tectonic plus gravitational) increasing linearly with depth, are applied. Bending forces in the model are avoided by making the valley small in relation to the whole model. If required, clay gouges (crushed zones containing clay) may be introduced.

Whichever method is chosen, a careful evaluation of the topography in the vicinity of the pressure tunnel or shaft is necessary. This is particularly important in non-glaciated, mountainous areas, where streams and creeks have eroded deep and irregular gullies and ravines in the valley sides. The remaining ridges, or so-called noses, between such deep ravines will, to a large extent, be stress relieved. They should therefore be neglected when the necessary overburden for unlined pressure shafts or tunnels is measured. This does not mean that pressure tunnels should not be running under ridges or noses – only that the extra overburden this may give should not be accounted for in the design, unless the stress field is verified through in-situ measurements, see Broch (1984) for further details.

As the permeability of the rock itself normally is negligible, it is the jointing and the faulting of the rock mass, and in particular the type and amount of joint infilling material, that is of importance when an area is being evaluated. Calcite is easily dissolved by cold, acid water, and gouge material like silt and swelling clay are easily eroded. Crossing crushed zones or faults containing these materials should preferably be avoided. If this is not possible, a careful sealing and grouting should be carried out. The grouting is the more important the closer leaking joints are to the powerhouse and access tunnels and the more their directions point towards these. The same is also valid for zones or layers of porous rock or rock that is heavily jointed or broken. A careful mapping of all types of discontinuities in the rock mass is therefore an important part of the planning and design of unlined pressure shafts and tunnels.

Hydraulic jacking tests are routinely carried out for unlined high-pressure shafts and tunnels. Such tests are particularly important in rock masses where the general knowledge of the stress situation is not well known or difficult to interpret based on the topographical conditions alone. The tests are normally carried out during the construction of the access tunnel to the powerhouse at the point just before the tunnel is planned to branch off to other parts of the plant, like for instance to the tailwater tunnel or to the tunnel to the bottom part of the pressure shaft.

To make sure that all possible joint sets are tested, holes are normally drilled in three different directions. By the use of Finite Element Models the rock stress situation in the testing area as well as at the bottom of the unlined shaft are estimated. At this stage the relative values of the stresses at the two points are more important than the actual values. During the testing the water pressure in the holes is raised to a level which is 20 to 30% higher than the water head just upstream of the steel-lining, accounting for the reduced stress level at the testing point. There is no need to carry out a complete hydraulic fracturing test. The crucial question is whether or not the water pressure in the unlined part of the shaft or tunnel is able to open or jack the already existing joints. Hence the importance for making sure that all possible joint sets are tested.

4 UNDERGROUND HYDROPOWER PLANTS WITH UNLINED WATERWAYS

To demonstrate the design approach an example of an underground hydropower plant will be shown and briefly described. Figure 7 shows the simplified plan and cross section of a small hydropower plant with only one turbine. No dimensions are given, as the intention is to show a system rather than give details. Similar layouts can be found for Norwegian plants with water heads in the range of 200–600 m.

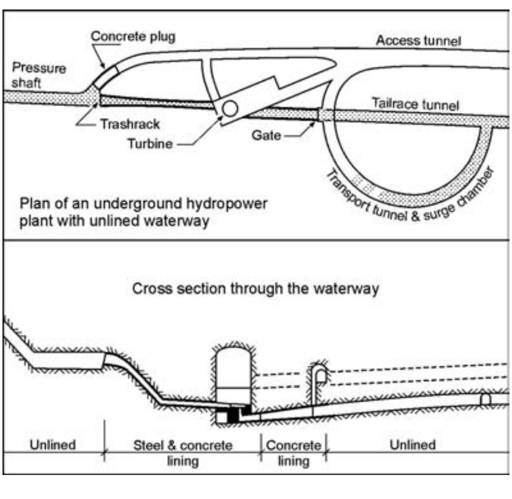

Figure 7. Plan and cross section of an underground hydropower plant with unlined waterways.

The figure is to some extent self-explanatory. A critical point for the location of the powerhouse will normally be where the unlined pressure shaft ends and the steel lining starts. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular the character and orientation of joints and fissures. Steel lengths in the range of 30–80 m are fairly common.

The access tunnel to the foot of the unlined pressure shaft is finally plugged with concrete and a steel tube with a hatch cover. The length of this plug is normally 10–40 m, depending on the water head and geological conditions. As a rough rule of thumb the length of the concrete plug is made 4% of the water head on the plug, which theoretically gives a maximum hydraulic gradient of 25. Around the concrete plug and the upper part of the steel-lined shaft a thorough high-pressure grouting is carried out. This avoids leakage into the powerhouse and the access tunnel. Further details about the design of high-pressure concrete plugs can be found in Dahlø et al. (1992) or Broch (1999).

5 OPERATIONAL EXPERIENCE FROM UNLINED PRESSURE SHAFTS AND TUNNELS

The oldest unlined pressure shafts have now been in operation for 80 years. None of the pressure shafts and tunnels, with water heads varying between 150 and 1000 m which have been constructed in Norway since 1970, has shown unacceptable leakage. It is thus fair to conclude that the design and construction of unlined high-pressure tunnels and shafts is a well proven technology.

It is normal procedure to fill a shaft in steps or intervals of 10–30 hours. During the intervals the water level in the shaft is continuously and accurately monitored by an extra-sensitive manometer. By deducting for the inflow of natural groundwater and the measured leakage through the concrete plug, it is possible to calculate the net leakage out from the unlined pressure tunnel or shaft to the surrounding rock masses. The loss of water from a tunnel is large during the first hours, but decreases rapidly and tend to reach a steady state after 12 to 24 hours, depending on the joint volume that has to be filled.

6 CONCLUDING REMARKS

Experience from a considerable number of pressure tunnels and shafts have been gathered over a long period of time. These show that, providing certain design rules are followed and certain geological and topographical conditions are avoided, unlined rock masses are able to contain water pressures up to at least 100 bars, equaling 1000 m water head. Air cushions have proven to be an economic alternative to the traditional open surge shaft for a number of hydropower plants. The geotechnical design of the air cushion cavern should follow the same basic rules as for other rock caverns.

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Workforce competence in the United Kingdom tunnelling industry

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ABSTRACT: For a number of reasons, there is an ongoing and increasing need for the UK tunnelling industry to ensure competence amongst its workforce at all levels. This paper reviews the background and describes a number of initiatives being taken by the industry to address this need for the workforce at all levels (operatives, engineers and management).

1 INTRODUCTION

There is an ongoing and increasing requirement for the United Kingdom (UK) tunnelling industry to ensure that its workforce at all levels (operatives, engineers and management) are competent to undertake their respective duties. This requirement for competence is being driven by high expectations of government, clients, insurers, as well as tunnelling contractors and consultants themselves, in terms of the quality, safety and environmental management of the work, and the confidence of delivery to time and budget.

This paper reviews the background to these high expectations. It then describes a number of initiatives which the industry is taking to address these expectations, and the roles of organisations such as the British Tunnelling Industry (BTS), the Pipe Jacking Association (PJA) and the United Kingdom Society for Trenchless Technology (UKSTT) in supporting such initiatives.

2 THE NEED TO DEMONSTRATE COMPETENCE

By its very nature, tunnel construction is inherently much more risky than most other forms of civil engineering. In the past there has been a reluctant acceptance of accidents and collapses in the mining and tunnelling industries being inevitable from time to time, but such events have now become totally unacceptable.

The UK tunnelling industry has had its fair share of challenges to respond to in recent years. In 1994 the industry's mood of confidence was dented by the collapse of tunnels being mined by New Austrian Tunnelling Method (NATM) techniques at Heathrow Airport's Central Terminal Area. There has since been a small number of ground subsidences associated with closed face tunnel boring machine (TBM) advances, which were unpredicted and difficult to explain. Inevitably, events like these do not help to give clients the confidence that the industry is competent.

At the same time, there is increasing recognition that the construction industry as a whole should and indeed can offer better value for money than it has been doing. Promoted by Sir John Egan and others, a number of high level initiatives, such as process driven approaches and partnering, have begun to instil into the construction better means of delivery, driving down overall project costs by up to 30%. Two of these initiatives, self-certification and design/construct contracting, have been adopted on a number of tunnelling projects. Self-certification is a good example of an initiative which brings with it a need for greater self responsibility and competence at all levels of the workforce. How can clients be confident that the whole tunnelling workforce is fit to deliver projects in these new ways?

Historically there has been a "grandfather rights" tradition in the workforce. Neither has it been obligatory for engineers and managers to have to demonstrate competence throughout their careers. This has now changed, brought about by a number of considerations, including:

- The changing legislative background, particularly in relation to workplace, designer and construction safety obligations.
- The expectations of the insurance industry. Insurers have been regarding tunnelling projects as high risk verging on being unacceptable. The Association of British Insurers (ABI) and the BTS have responded to this situation with the publication of a Joint Code of Practice (Ref. 1).
- Increased requirements from Clients for companies to demonstrate the competence of staff, to the occasional extreme of requiring tunnelling personnel to be formally certified as competent.
- The tunnelling industry itself, in responding to these increased expectations.

3 THE NATURE OF COMPETENCE

The authors have chosen to discuss the nature of competence because it is an ill defined term, although much used. Competence is widely accepted to mean that a person has any or all of the following – manual skills; an inherent ability; received training; a qualification; a practical and capable outlook; a sound mind.

However, the authors consider that competence is better defined as a combination of knowledge; experience; and inherent behavioural characteristics. They also believe that competence is specific to and should always be assessed against a particular set of circumstances or situation. This is because safe and productive tunnelling, more than almost any other form of construction, is a team operation with all team members needed to be confident through competence.

The process by which competence is achieved is broadly summarised in Figure 1.

The requirement to assess competence has come from a number of directions. ISO 9001 (Ref. 2) requires "*that we assess the competence of our staff*". The Joint Code of Practice (Ref. 1) requires for tunnel projects, training plans and a policy for the employment of skilled operatives. Requirements for a competent team (Managers, Engineers and Operatives) will also come from contract requirements, particularly for schemes involving self certification.

4 VERIFICATION OF TUNNELLING OPERATIVES

For many years Electricians, Mechanics (fitters), Carpenters and other building trades have had processes to train and approve their skills. Only in recent years has the UK tunnelling industry come to consider the need

Figure 1. Route to competence.

to train and verify the skills of many of its employees, for example Plant Operators, Crane Drivers, Concrete Constructors and, most notably, Tunnel Miners.

The UK Government introduced National Vocational Qualifications (NVQs) in 1986, including the declared object to recognise "*skills that were not otherwise recognised by existing available qualifications*".

An Occupational Working Group (OWG) has been established for each sector of the construction industry that wishes to have NVQs. The Tunnelling OWG was formed in 1993. The OWG is formed of representatives from the industry and the Construction Industry Training Board (CITB), and its function is to propose the content of the NVQs and submit them for accreditation.

NVQs are vocational qualifications and involve assessment by trained assessors who are themselves experienced in the particular discipline. They are not training based as such, and certainly are not training certificates, but are based on assessments of the candidates' performance and knowledge. There are five levels of NVQs:

- **Level One** is unskilled.
- **Level Two** is skilled.
- **Level Three** is for skilled supervisors.
- **Level Four** is for Senior Technicians and Engineers
- **Level Five** is for Managers.

The UK tunnelling industry's focus to date has been at the skilled tunnellers (Level 2) and is the only current fully accredited NVQ Level, although accreditation for Level 1 is currently being set up. First piloted in 1994, the initial tunnelling NVQ scheme suffered from over complexity and take up by the industry was low. More recently NVQs have been simplified, without reducing the requirements, and take up has much improved as a result.

The qualification is structured around units, as shown in Figure 2.

Units relate to operations, and defined in each unit are performance criteria together with the knowledge and understanding which underpin the performance. The assessment of a candidate can be carried out by a

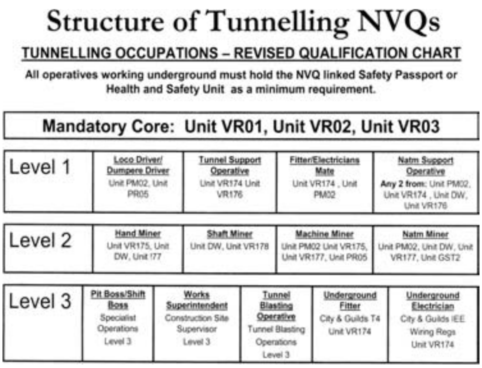

Figure 2. Structure of tunnelling NVQs.

combination of ways and if necessary over a considerable period, and is usually carried out on a site, where skills can be evidenced and signed off for their qualification. The assessors must work under an internally and externally verified accreditation centre, normally an employer who has to convince the Awarding Body that the proper checks and procedures are in place to ensure that the assessment is properly carried out.

A major challenge for the tunnelling industry is to resolve its short term, project to project outlook, which constrains long term career opportunities for skilled operatives; and instead create a real incentive to achieve qualifications.

Given the cyclical nature of the UK's tunnelling business activity, it is important too that the whole construction industry is encouraged to understand that the core competencies demonstrated by these NVQs permit the NVQ holders to be able to transfer readily into other forms of construction.

Recently, the substantial CTRL project with its large workforce provided the opportunity, with the help of the CITB, to establish a project based accreditation centre which registered some 300 operatives for NVQs, and achieved 40 full NVQs at Level 2. By August 2004 a total of 349 tunnelling NVQs had been registered and 76 NVQs awarded. The response from the tunnellers has been very positive. They have taken pride in achieving their qualifications. The authors also consider that recognising the skills of the tunnellers has encouraged a sense of responsibility which has spin offs in other areas, particularly in relation to health and safety.

5 UNIVERSITY STUDENT TRAINING INITIATIVES

It has long been recognised that many young tunnel engineers have had little or no exposure as undergraduate and postgraduate students to actual project related problem solving situations. Unless fortunate enough to have obtained some vacation work with tunnelling companies, they will continue to enter the industry with a lack of any practical experience.

An excellent example of an initiative taken to address this problem is provided by an arrangement between the University of Newcastle and Edmund Nuttall Limited. Together they have developed an integrated initiative to train students in the principles of underground construction using actual tender and construction information. This approach responds directly to the desire of the UK Institution of Civil Engineers (ICE) to see engineering taught in a holistic and multi-disciplinary way. Students are shown how their knowledge of geology, hydraulics, soil and rock mechanics, structures, etc. is put into practice to solve the technically demanding problems faced daily in the industry.

This unique course commenced in September 2001, and is delivered through a series of supporting lectures on shaft sinking and tunnelling using carefully chosen examples to highlight the importance of a comprehensive ground investigation, its interpretation in three dimensions, and the correct choice of construction method(s) to suit the ground conditions. One such project was a 7.8 km long, 2.85 m internal diameter wastewater tunnel constructed along the west side of Langstone Harbour, Portsmouth, UK, in two concurrent sections of 4.0 km and 3.8 km from an intermediate shaft, 7.5 m internal diameter by 28 m deep (Figure 3).

This project was chosen because it was technically challenging and demanded control of both ground water and ground during shaft and tunnel construction. In order to add realism to the design project students were not informed of the actual construction methods uses, or shown site documentation such as reports, temporary works drawings, photographs, etc., until the relevant project section detailed below had been satisfactorily completed. They were however presented with selected Tender Documentation and guided through a logical sequence from project conception to completion, typically producing:

- an interpretative geological report
- a conceptual design which they refined into the final design
- a number of design elements
- and the most advantageous construction method(s).

In achieving the project requirements students learned the importance of:

- a comprehensive well considered site investigation
- the accurate assessment of ground conditions and behaviour prediction
- the determination of the most appropriate construction methods, and

Figure 3. Portsmouth tunnel, UK.

- producing concise, well written and illustrated reports with logical thinking and clearly presented conclusions.

This course introduced students to the "real world" and showed them how to apply their underpinning knowledge to solve a technically challenging problem in a logical manner within an agreed timeframe. Clearly students who graduate from this course are better equipped and prepared for industry than their counterparts and will, in the future, fulfill a valuable role in geotechnical engineering.

6 CONTINUING PROFESSIONAL DEVELOPMENT ROLE

The increasing obligations on the whole UK tunnelling industry to deliver projects better, require the highest standards of effective leadership and management. It will no longer be enough for senior professional tunnellers to rely and trade upon their work experiences alone; they will be obliged to demonstrate Continuing Professional Development (CPD) throughout their careers.

In some professional institutions, for example the Institution of Structural Engineers (IStructE), CPD is already mandatory for members. ICE presently treats CPD as obligatory but not mandatory, and provides a CPD record form, which its members and fellows are encouraged to maintain up to date.

The authors consider that CPD records provide a valuable tool for professional tunnellers to help demonstrate their continuing competence. If really necessary, and other evidence is not readily available, CPD records could be used to prove to Clients that senior staff are appropriately trained in areas of critical knowledge, for example familiarisation with the latest CDM Regulations. Directors of tunnelling companies should be encouraged to ensure that their more senior staff maintain CPD, not just for the immediate benefit of the companies themselves but also to underpin confidence in the whole UK tunnelling industry.

7 THE ROLE OF PROFESSIONAL SOCIETIES AND INDUSTRY ASSOCIATIONS

Organisations with a primary interest in the wellbeing of the UK's tunnelling industry, such as the BTS, PJA and UKSTT, have recognised that the pursuit of demonstrable competence is of increasing importance for both individual and corporate members.

For its part the BTS has concluded that it has a duty to monitor competency levels and support all individual and corporate members in their quest to demonstrate competence. In recent years it has made major contributions to advancing the state of knowledge amongst members and the wider construction industry about tunnelling requirements. For example, it was proactive in establishing a dialogue with insurers and preparing the joint Code of Practice (Ref. 2) with insurers. It has led a Closed Face Tunnelling Working Group which has published guidance (Ref. 3), and has also produced a model Specification for Tunnelling (Ref. 4) and a Tunnel Lining Design Guide (Ref. 5). The BTS's active representation on the Tunnelling OWG and support of NVQs is testament to its commitment to operatives.

During 2003 the BTS considered the necessity and desirability to introduce a Registration scheme for tunnelling professionals. The Society concluded that to take no action in this regard carried with it a real risk that the BTS would fail to move forward with the changing times, but also recognised that it would be almost certainly impractical, probably unnecessary, and in any case premature, for the BTS to invoke a full scale Registration scheme. An interim measure being considered is to set up of a tier of key Tunnelling Professionals, who could fulfil a number of functions, including having an accrediting role if Registration is eventually introduced.

8 CONCLUSIONS

In responding to increasing demands from clients and others, ongoing training is required at all levels of the UK's tunnelling workforce. Competence, however, is about much more than obtaining qualifications on paper, being a combination of knowledge, experience and behaviour.

The UK tunnelling industry has taken note of these increasing expectations, and is doing much to provide a workforce which is competent, effective and well managed at all levels. Organisations including the BTS, PJA and UKSTT are playing an active role in supporting the industry's endeavours in this regard.

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Microtunnelling or trench excavation: a comparison of the two alternatives

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ABSTRACT: The technical and economic feasibility of an underground construction should involve not only geological, geotechnical and technological aspects, but also the evaluation of the factors that are related to the interferences of the construction procedures and techniques with the population and with the environmental components of the site.

The paper compares two different excavation alternatives, microtunnelling and trench excavation, for an urban sewer construction project using the Analytic Hierarchy Process.

1 INTRODUCTION

The need for new transport systems, new spaces for shops, offices, car parks, social and recreational activities and innovative service networks such as telephone cables, sewers and gas pipes is one of the reasons for the increase in underground works in urban areas. When a new service pipe has to be installed at a shallow depth below the surface it is normal practice either to excavate from the surface, to install pipes in trenches or to bore a small underground tunnel.

Open space works can cause problems for the surrounding environment and these aspects have to be considered for a correct technical-economic analysis of such a project; the microtunnelling technique, in fact, allows the surface area directly involved by construction to be reduced (Stein et al, 1989; Thomson, 1995).

Therefore, for a correct comparison of these two operational procedures, it is necessary to take into account the different environmental and social aspects.

The comparison can be performed with the Analytic Hierarchy Process (AHP) which is a multicriteria technique that provides useful support when a choice between several alternatives with different objectives and criteria has to be made (Fusco & Nijkamp, 1997; Nijkamp et al. 1990; Voogd, 1983).

2 THE ANALYTIC HIERARCHY PROCESS

The Analytic Hierarchy Process (AHP) is a procedure and computation approach that is suitable for dealing with complex systems related to making a choice from among several alternatives and which provides a comparison of the considered options.

The AHP helps organize the rational analysis of the problem by dividing it into its single parts, in a hierarchical form. The analysis then supplies an aid to the decision makers who, making several pairwise comparisons, can appreciate the influence of the considered elements in the hierarchical structure and give a preference list of the considered alternative solutions (Saaty, 1980; Saaty & Vargas, 1990; Mondini & Nati Poltri, 1996).

The AHP has been used to analyse different kinds of social, political-economic and technological problems. It uses both qualitative and quantitative variables. The fundamental principle of the analysis is the possibility of connecting information, based on knowledge, to make decisions or previsions; this knowledge can be taken from experience or derived from the application of other tools. Among the different contexts in which the AHP can be applied, mention can be made of the creation of a list of priorities, the choice of the best policy, the optimal allocation of resources, the prevision of results and temporal dependencies and the assessment of risks and planning (Saaty & Vargas, 1990).

The analysis is based on three fundamental principles:

1. breaking down the problem;
2. pairwise comparison of the various alternatives;
3. synthesis of the preferences.

The "*breaking down principle*" consists in subdividing the problem into simple clusters that are

Table 1. Saaty's fundamental scale.

Value	Definition	Explanation
1	Same importance	Two decision elements equally influence the parent decision element
3	Moderately more important	One decision element is moderately more influential than the other
5	Much more important	One decision element has a greater influence than the other
7	Very much more important	One decision element has significantly more influence over the other
9	Extremely more important	The difference between the influences of the two decision elements is extremely significant
2, 4, 6, 8	Intermediate judgment values	

represented by different levels in a hierarchal structure. The decomposition is carried out from the top to the bottom, starting from the objective going on to the criteria and sub-criteria and then to the final alternatives.

The "*pairwise comparison principle*" consists in giving a rate to each cluster to measure the importance of each level in the hierarchy. Each single element is evaluated using pairwise comparison. The comparisons are made on a 9-point scale, the so-called "*Saaty's fundamental scale*", which is represented in Table 1.

The numerical judgments established at each level of the hierarchy are made up using pair matrixes.

Let n be the number of criteria in a certain level of the hierarchy and m the number of the alternatives, there are therefore n matrixes with m lines and m columns in that level. An example of a pairwise comparison matrix is given in Figure 1.

All the pairwise comparison matrices have two fundamental properties:

- The principal diagonal is always composed of values that are equal to one (because each criterion *i* is compared to itself);
- The matrices are reciprocal (in assigning a value from 1 to 9 to the comparison between the element *i* and the element *j*, the reciprocal value corresponds to the comparison between *j* and *i*).

The last step of the procedure consists of a comparison of the various alternatives, referring to the given judgments and extracting the related principal autovector from each comparison matrix.

n	1	2	3	…	m
1	1			Reciprocal values	
2		1			
3			1		
…				1	
m	Judgements values				1

Figure 1. Pairwise comparison matrix.

The principal vector measures how the components, at each hierarchy level, contribute to reach the objective with reference to which comparison has been performed. Each autovector must be weighted with regards to the priority of the considered criteria. The final list, obtained by summing all the autovectors, is a vector that provides the part played by each alternative in reaching the initial goal.

3 TRENCH EXCAVATION AND MICROTUNNELLING COMPARISON USING AHP

The AHP methodology, as described in chapter 2, has been used to compare traditional trench excavation with microtunnelling for the installation of a pipe in an urban area based on the data of a real case in Turin (Italy) of a sewer installation using trench excavation under a local urban road.

The fundamental goal of the analysis was to define which excavation technique was most compatible, from the point of view of the public and private actors who are involved in the construction process, taking into account the environmental and social aspects.

The first step of the analysis is the definition of the hierarchical structure to represent the problem (Bottero & Peila, 2004). The general criteria considered for the comparison are:

– direct construction costs;
– indirect construction costs;
– social costs;
– environmental costs.

All these costs together give the "global cost" of the construction.

In this research the analyses were carried out taking into account the results of a questionnaire that had been distributed to the population in the surrounding area with the goal of focusing the most relevant social impact on the people who live around the work area and on the economic activities such as shops.

Table 2. Priority list of the subcriteria related to the "indirect construction costs" criteria.

Damage to private property	0.038
Damage to other infrastructures	0.066
Reduction in the life cycle of the road surface	0.242
Increase in road maintenance	0.242
Relocation of services that interfere with the works	0.412

Table 3. Priority list of the "traffic costs" subcriteria.

Measures for the diversion of traffic circulation	0.041
Time loss due to traffic increase and traffic jams	0.170
Choice of new routes for public transport	0.044
Installation of new road signs	0.040
Higher number of road accidents	0.115
Driving efficiency	0.299
Cost increase for public and private transport	0.291

3.1 *Analysis of the general criteria*

3.1.1 *Direct construction costs*

The direct construction costs involve the installation of pipes and consist of the total economic resources involved in the "physical" work and in the recovering of the site at the end of the works.

These costs, with reference to the considered example, are higher for microtunnelling than for trench excavation. Obviously different economic conditions are present when crossing under rivers, buildings or very sensitive infrastructures or when groundwater is present. These specific conditions are not considered and discussed in this work.

3.1.2 *Indirect construction costs*

The indirect costs are the expenses for materials, machinery and works not directly related to the construction phase.

The following costs are taken into account in the analysis:

1. costs due to damage to private property;
2. costs due to damage to other infrastructures;
3. costs due to the reduction in the life cycle of the road surface;
4. costs due to the increase in road maintenance;
5. costs due to the relocation of already existing infrastructures that interfere with the works.

In the considered example the highest cost value is given to the relocation of infrastructure (point 5).

The priority list that is obtained from the compilation of the pairwise comparison matrix and from the subsequent normalization is reported in Table 2.

If the two alternatives are compared with reference to the indirect construction costs, the surface excavation is the most critical.

Microtunnelling, in fact, involves the occupation of a limited area for the construction site and reduces damage to road surfaces, to private property and to the existing infrastructures. The use of microtunnelling also minimizes the relocation of the already existing underground infrastructures, which is limited to the shaft area instead of to the whole pipe length.

3.1.3 *Social costs*

The social costs are the costs borne by society during the operations. They are related to the externalities caused by the interferences between the construction site and the economic activities of the area.

The following costs are therefore taken into account:

1. problems of traffic circulation;
2. safety aspects of the work sites;
3. disturbance to commercial and recreational activities in the area;
4. landscape factors;
5. human factors.

Considering the complexity of the topics involved in these five aspects, it is necessary to further subdivide them.

The criterion related to traffic costs is therefore subdivided into the following main elements:

1(a) measures for the diversion of traffic circulation;
1(b) time loss due to traffic increase and traffic jams;
1(c) choice of new routes for public transport;
1(d) installation of new road signs;
1(e) higher number of road accidents;
1(f) driving efficiency;
1(g) cost increase in public and private transport.

When making the judgements, the increase in the costs for public and private transport, the driving efficiency and the increased number of road accidents are considered to be the most important. The priority list is reported in Table 3.

If these criteria are taken into account, trench excavation is worse than microtunnelling.

The criterion related to work safety (point 2) is linked to the costs borne by the contractors and by society for the safety of the workers in the construction site area.

In the analysis, these costs are considered, for this preliminary analysis, as being directly proportional to the surface of the construction site. The greater the area, the greater the exposure of the workers to risks (in fact, smaller shafts site areas are usually properly retained and safety measures are better enforced).

Therefore, as the construction site is larger for trench excavation than for microtunnelling the costs for safety are considered higher for the former solution.

The criterion related to the disturbance to commercial and recreational surface activities (point 3) is directly connected to the physical hindrances created by the construction site on the spaces for shops and facilities. In order to make a correct evaluation, it is necessary to have detailed information on the difficulties that are actually borne by the shopkeepers or on the temporary closing of shops or facility areas. This is obtained using the results of the questionnaire. It is, however, evident that the disturbance is higher for traditional excavation, especially when the roads are closed to both vehicular and pedestrian circulation.

The criterion related to landscape factors (point 4) has to be evaluated considering the negative impact generated by the construction site on the "urban and territorial values" of the area. In the analysis, this criterion was sub-divided into the following categories:

4(a) perceptive-visual aspects;
4(b) fundamental elements of the landscape.

In generic terms, the transformation of the urban landscape due to the construction site is reversible and when the works are over it is possible to restore the previous conditions or even improve them and the two options are therefore considered as equal.

The criterion related to human factors (point 5) is associated to the negative impact, caused by the construction site, as felt by the inhabitants of the area surrounding the job site. In the analysis, this criterion is sub-divided into:

5(a) aspects related to public safety;
5(b) aspects related to public health;
5(c) aspects related to the quality of houses.*

Even though the impact of a construction site for the installation of a sewer pipe system is transitory and stops when the works are over, it leads to the highest number of complaints from the population, as was clearly shown by the questionnaire.

In the analysis, public safety and health are considered to prevail over the quality of the houses, which, generally speaking, is not affected very much by this kind of excavation. The priority list is reported in Table 4.

It is intuitive, with reference to these factors, that trench excavation is more critical than microtunnelling because the former tends to create a higher impact in terms of dust emissions, noise and vibrations.

3.1.4 *Environmental costs*

Environmental costs are used to quantify the permanent negative effects on the environmental components of the area. In the analysis, this factor is divided into five criteria and the same weight is given to each of them: air, soil, water, physical agents and natural agents.

* These elements involve the possible disturbance for the inhabitants of the area due to the construction site, with reference to the impact on the building quality (noise, dust, vibrations, …).

Table 4. Priority list of the "human factors" subcriteria.

Public safety	0.455
Public health	0.455
Quality of houses	0.090

With reference to air, trench excavation has been considered to have more serious effects than microtunnelling because of the greater consumption of fuel for the excavation and transport machines and consequently the greater gas emissions in the atmosphere.

With reference to soil, trench excavation has a greater environmental impact than microtunnelling, but the latter requires more geological and geotechnical investigations for a better knowledge of the underground conditions.

With reference to water, the potential interferences between the operations and the quality of the surface and underground waters were taken into account: trench excavations can in fact interact with groundwater, generate pollution and affect surface water circulation (at least for the period of construction) while microtunnelling has a lower impact.

The physical agents are related to the impact of the construction site on the life of the inhabitants in the area due to noise and vibrations while natural agents refer to the impact of the operations on the vegetation conditions of in the area. In both cases, microtunnelling is considered preferable to trench excavation.

3.2 *Comparison of the two alternatives*

In order to develop a comparison between the alternatives, it is necessary to give specific weights to each cost.

These weights can be obtained starting from the pairwise comparison matrix for the general criteria. The results are summarized in Table 5.

If Table 5 is analysed, it is possible to observe that in this study some of the aspects that are usually underestimated by decision makers such as indirect construction costs and social costs are considered and high weights are given to them. This choice is arbitrary in the analysis and is closely linked to the decision making process. Different choices can obviously be made.

In the studied example, the evaluation was made while trying to highlight the point of view of the society and therefore the social and environmental costs take on a great importance.

The priority list obtained following the procedure described in paragraph 2 is reported in Table 6.

If the two alternatives are compared, on the basis of the chosen priorities (Table 5), it can be seen that trench excavation has a score of 0.23 while microtunnelling has a score of 0.77.

Table 5. Pairwise comparison matrix for the general criteria.

	Direct construction costs	Indirect construction costs	Social costs	Environmental costs
Direct construction costs	1	1/3	1/3	2
Indirect construction costs	3	1	1	5
Social costs	3	1	1	5
Environmental costs	1/2	1/5	1/5	1

Table 6. Priority list of the general criteria.

Direct construction costs	0.137
Indirect construction costs	0.394
Social costs	0.394
Environmental costs	0.075

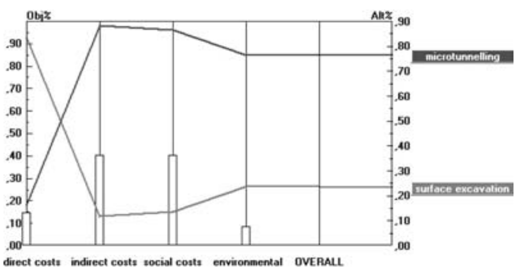

Figure 2. Performance graph.

Therefore, if the indirect construction costs, the social costs and the environmental costs are taken into account, microtunnelling appears to be preferable to trench excavation. Only if the analysis is performed maximizing the direct costs, does the trench excavation become preferable to microtunneling as it is possible to see in the graphs of Figures 2 and 3.

The Performance graph of Figure 2 highlights the priorities of the final alternatives, as regards the general criteria; the graph shows the course of the two alternatives (represented by two lines) with reference to each criterion (represented by a vertical bar proportional to the respective weight) and to the total priority list.

The graphs in Figure 3 show the relationships that exist between the two alternatives in function of the chosen weights of the general criterion (represented by the vertical line).

Figure 3(a) shows that if the weight of the direct construction costs is increased (moving the vertical bar to the right side of the graph from the chosen value of 14% to the value of 100%) and consequently the weights of the other criteria are decreased (in a proportional way to their initial weight) the trench method is preferable when a weight of 53% is given to the direct construction costs. In all the other cases, microtunnelling is always a better solution than trench excavation and these differences increase with a rise in the weight of the indirect, social and environmental costs.

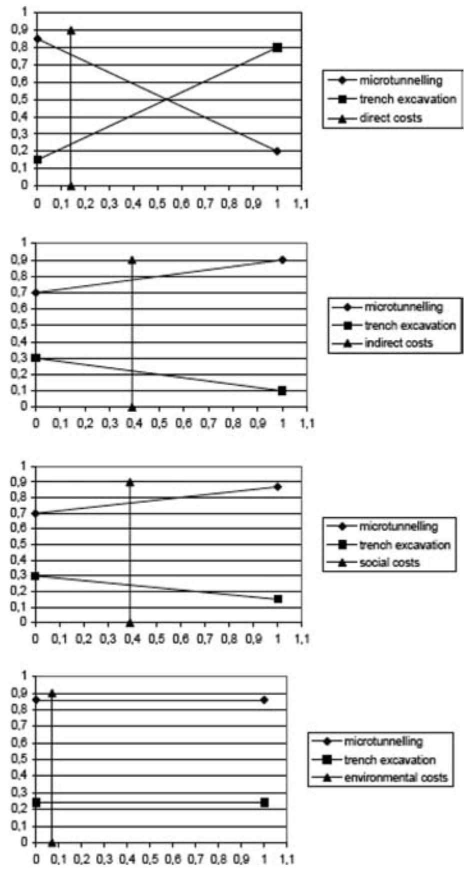

Figure 3. Gradient graphs with reference to the considered criteria.

With specific reference to direct construction costs, some analyses carried out in Germany highlighted the influence of depth of the excavation and of the presence of groundwater (Stein et al. 1989) on the comparison between microtunnelling and trench excavation: if there is groundwater or the pipe is deep, the microtunnelling alternative is the best solution ever when the direct costs are considered.

Furthermore, in some cases, microtunnelling is the only possible technique, for example when crossing under infrastructures that cannot be interrupted such as canals, railways or very important roads or when there are buildings or delicate surface structures close by.

4 CONCLUSIONS

The comparison of trench excavation and microtunnelling that was carried out in this analysis shows that if indirect construction costs, social and environmental costs are considered with appropriate weights, microtunnelling excavation for the installation of a sewer in urban area is a better solution than conventional trench excavation.

Analytical tools that are able to help decision makers to correctly evaluate all the factors which are involved are nowadays available. Tools such as the one used in this paper are suitable and should be used to make the correct decision when comparing these two possible underground construction alternatives.

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Tunnels related to Metro, Highway and Sewerage schemes in Istanbul

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ABSTRACT: Tunnels related to Light Rail Transit System (LRTS), Metro, Highway and Sewerage schemes which have been implemented in the last decade in Istanbul, have led to valuable data regarding tunneling methods to be implemented in Istanbul. This is particularly important for this major metropolitan city as minimum of 200 km of metro tunnels are planned to be implemented in the next decade with the capital expenditure of 10 Billion US Dollars. In this paper construction methods, theoretical analyses and geotechnical monitoring pertaining to Incirli tunnel on LRTS, Tantavi and Kisikli Tunnels on Umraniye connection of E5 and E6 Motorways, Taksim-4. Levent metro and Istanbul sewerage tunnels have been discussed. Good correlation was achieved between measured and calculated values and predictions have been made on probable advance rates and costs to occur in the future tunnels to be constructed in and around Istanbul with a possibility of results setting general global trends in similar rock masses.

1 INTRODUCTION

In this paper construction of Incirli Tunnel located on LRTS 2nd Phase, Kisikli and Tantavi tunnels located on Umraniye connection of E5 and E6 Highways, Taksim-4. Levent section of Istanbul Metro and Bebek-Baltalimani sewerage tunnel, which is part of major Istanbul Sewerage Project, have been discussed and based on the data obtained from geotechnical monitoring of these tunnels, valuable data regarding possible advance rates and costs of the tunnels to be constructed in Istanbul have been developed.

Figure 1. Location of infrastructure tunnels in Istanbul.

2 TUNNELS HISTORICALLY CONSTRUCTED IN ISTANBUL

History of infrastructure tunnels constructed in Istanbul extend beyond the century.

First transport tunnel in Istanbul is the 574 m long tunnel constructed between Beyoglu and Karakoy in 1874 (5). This was in fact commencement of 1st metro construction on BO Basis, 2nd in the world after London Metro. Since then not much has been achieved due to World Wars, economical difficulties etc. until 1980s.

Within the scope of infrastructure developments undertaken in Istanbul during the last 20 years, interceptor tunnels have been constructed since 1985 within the scope of Istanbul Sewerage Project. Within the project of Southern Golden Horn interceptors, approximately 7 km tunnel construction has been undertaken with internal diameters varying between 2.2–2.8 m. Within Northern Golden Horn interceptors project, approximate length of 5 km tunnel has been driven with internal diameters varying between 2.6–3.2 m and within the scope of Besiktas-Baltalimani interceptors, 8 km tunnel was constructed with internal diameters varying between 3.2–3.6 m (4).

Parallel to population growth mass transit systems gained priority at the city center in parallel to high density of passenger movement. In order to provide a solution to Istanbul's decaying urban transport problem, initially LRTS system construction had started in 1986 and Phase I incorporating Aksaray–Esenler–Zeytinburnu has been put into operation by 1993.

Later 2nd stage construction, between bus terminal and airport, had started in 1991 and was completed in 1997. This 9.6 km long section of LRTS includes 1545 m long Incirli Tunnel, between Bakirkoy and Atakoy.

Istanbul Metro Phase I construction, connecting the city's commercial and cultural centre Taksim and 4. Levent, had started in 1992 and completed in 2000. Phase II construction is currently ongoing.

Highway tunnels were built on peripheral motorways around Istanbul in order to assist problems of traffic congestion and to connect both E5 and E6 motorways where necessary. Among these highway tunnels Kisikli and Tantavi tunnels were driven on the Anatolian side for the connection of E5 and E6 Highways at Umraniye between 1995 and 1997.

Accordingly when infrastructure works gained acceleration throughout the last 20 years, approximately 30 km tunnels were driven in Istanbul at separate locations due to transport and environmental goals and valuable data gathering was achieved regarding construction and cost aspects of these infrastructure tunnels.

3 INFRASTRUCTURE TUNNELS IN ISTANBUL

3.1 *Construction method*

Properties of tunnels driven in compliance with the New Austrian Tunneling Method (NATM) (7) are given in Table 1. Only Baltalimani tunnel in the table is constructed with shielded roadheader tunnel mechanism and without implementing the NATM.

Main principle of NATM is to convert the rock mass surrounding the tunnel to a load carrying rock mass. Achievement of this main principle requires utilization of supporting elements such as shotcrete, wire mesh, rock bolt, steel supports and fore poling when necessary. Support and excavation details showing the typical sections implemented in stages for Incirli Tunnel, for Kisikli – Tantavi tunnels and for Istanbul Metro tunnels are presented in the author's previous publication (5).

For Incirli, Kisikli, Tantavi and Metro Tunnels, following geotechnical investigations prior to design, three types of excavation and support systems have been determined and certain alterations have been implemented in these excavation systems as required following in-situ geotechnical monitoring carried out during tunnel excavation (1, 6).

Main reasons for selection of NATM method were as follows:

1. Rapid adaptation to varying geotechnical conditions.
2. Easy adaptation to varying sections.
3. A proven track record in weak and fractured rock masses.
4. Avoiding disturbance of soil and rock with deformation measurements and minimizing settlements on the surface.

3.2 *Geological–geotechnical conditions*

3.2.1 *Incirli tunnel*

Incirli Tunnel has been constructed in formations constituting of Upper Miocene old clay-marn and limestone (1). In the region these formations exist occasionally as carboniferous or as discordance on Eocene and on the tunnel alignment soil mainly consists of Gungoren Formation, Bakirkoy Formation and clays.

Table 1. Characteristics of Istanbul infrastructure tunnels.

Tunnel properties	Incirli Tunnel	Kisikli Tunnel	Tantavi Tunnel	Metro Tunnel	Baltalimani Tunnel
Location	Bakirkoy-Atakoy	Camlica	Umraniye	Taksim-4. Levent	Bebek-Baltalimani
Type	Single tube-double track	Twin Tunnel	Twin Tunnel	Twin Tunnel	Single Tube
Length (m)	1,337	634	210	4,987	2,342
Cut & cover (m)	68	68	60	700	–
Station (m)	140	–	–	1,080	–
Total (m)	1,545	720	270	6,677	2,342
Excavation section (m^2)	75	145	145	36	16
Concrete cover thickness (cm)	40	20	20	40	45
Concrete type	BS30	BS30	BS30	BS25	BS30
Tunnel excavation (m^2)	100,000	105,000	39,500	250,000	40,000
Shotcrete (m^2)	17,000	5,500	2,000	50,000	7,000
Internal lining concrete (m^2)	19,000	6,150	2,300	57,000	15,000

Gungoren Formation consists of sand, silt-clay, clay and marn. Any sand layer has not been encountered at the tunnel level. Green-grey colored silty-clay is quite hard with claystone lithology at certain levels and limestone consists of marn and sand intrusions.

Geotechnical parameters of Gungoren Formation are listed below:

Soil Grade	: CH
Liquid Limit (LL)	: 67%
Plastic Limit (PL)	: 27%
Plasticity Index (PI)	: 40%
Natural Water Content (Wn)	: 38%
Unit Weight (yn)	: 1.81 t/m^3
Density (Gs)	: 2.73
Saturation Level (S)	: 97%
Swelling	: 0.78%
Compression Coefficient (Cc)	: 0.24
Consolidation Coefficient (Cv)	: 0.031 cm^2/min
Cohesion (c)	: $>$1.5 kg/cm^2
SPT N/30	: 63
Elasticity Modulus (Em)	: 579 kg/cm^2

Bakirkoy Formation: Dominant lithology is limestone in Bakirkoy Formation. Cream-white colored limestone include occasional thin clay bands and display transversal and vertical transactions with marn layers. Limestone comprising thin-thick layers, which are occasionally massive, include mactra fossils in abundance at certain levels. Along the tunnel alignment approximate thickness of limestone varies between 10–20 m.

Average physical and mechanical parameters of Bakirkoy Formation are as follows:

Unit Weight (yn)	: 2.29 t/m^3
Natural Water Content (Wn)	: 0.92%
Uniaxial Compressive Strength (σ_c)	: 222 kg/cm^2
Cohesion (c)	: 40 kg/cm^2
Internal Friction Angle (Ø)	: 48
Elasticity Modulus (Em)	: 25.000 kg/cm^2
Poisson Ratio (v)	: 0.25
Total Core Ratio (TCR)	: 60%
Rock Quality Designation (RQD)	: 16%

3.2.2 *Kisikli tunnel*

Kisikli Tunnel located between B.Camlica and K. Camlica is constructed in an excessively sophisticated geological structure and major rock formations are quartzite, arcose, clay stone-siltstone intrusions and intersecting volcanic dykes.

Geotechnical parameters of claystone-siltstone intrusions and volcanic formations encountered on the alignment of the tunnel are given in Table 2 enclosed.

3.2.3 *Tantavi tunnel*

Tantavi Tunnel is constructed in over weathered clay-siltstone, silt-clay stone and volcanic dykes intersecting along joints and fault zones.

Geotechnical parameters of volcanic formations are listed below:

Unit Weight (yn)	: 2.53 t/m^3
Natural Water Content (Wn)	: 0.71%
Uniaxial Compressive Strength (σ_c)	: 545 kg/cm^2
Cohesion (c)	: 13 kg/cm^2
Internal Friction Angle (Ø)	: 30
Elasticity Modulus (Em)	: 59.000 kg/cm^2
Poisson Ratio (v)	: 0.30
Total Core Ratio (TCR)	: 50%
Rock Quality Designation (RQD)	: 10%

3.2.4 *Metro tunnels (Taksim-4.Levent)*

Metro tunnels are generally constructed in Paleozoic aged Thracian Formation.

Table 2. Typical geotechnical parameters of formations encountered along Kisikli Tunnel.

Parameters	Sandstone	Claystone-siltstone	Diabase-Andesite
Unit Weight (yn)-t/m^3	2.29	2.57	2.31
Natural water content (Wn)-%	8.95	1.0	15.8
Uniaxial compressive strength σ_c-kg/cm^2	360	1,080	620
Cohesion (c) kg/cm^2	18	46	14
Internal friction angle (Ø)	34	38	31
Elasticity modulus (Em)-kg/cm^2	80,000	79,000	40,000
Poisson Rate (v)	0.27	0.35	0.28
Total core ratio (TCR)-%	35	65	30
Rock quality (RQD)-%	15	34	20

Table 3. Typical geotechnical parameters of formations encountered along Metro Tunnel (Taksim–Sisli).

Parameters	Claystone-siltstone	Quartzite	Volcanics
Unit Weight (yn)-t/m^3	2.60	2.30	23.2
Natural water content (Wn)-%	1.5	7.5	14.0
Uniaxial compressive strength σ_c-kg/cm^2	540	250	720
Cohesion (c) kg/cm^2	30	30	35
Internal friction angle (Ø)	35	30	50
Elasticity modulus (Em)-kg/cm^2	20,000	60,000	58,000
Poisson Rate (v)	0.35	0.29	0.30
Total core ratio (TCR)-%	70	60	50
Rock quality (RQD)-%	25–50	50–75	50–75

Table 4. Typical geotechnical parameters of formations encountered along Baltalimani Tunnel (Bebek–Baltalimani).

Parameters	Sandstone	Claystone-siltstone	Limestone	Shale
Unit Weight (yn)-t/m^3	2.32	2.30	2.29	2.38
Natural water content (Wn)-%	0.85	9.0	0.95	1.0
Uniaxial compressive strength σ_c-kg/cm^2	555	300	874	520
Cohesion (c) kg/cm^2	35	25	50	18
Internal friction angle (Ø)	37	32	41	33
Elasticity modulus (Em)-kg/cm^2	27,000	71,000	70,000	43,000
Poisson Rate (v)	0.23	0.26	0.25	0.30
Total core ratio (TCR)-%	70	55	60	50
Rock quality (RQD)-%	25–50	25–50	0–25	0–25

Thracian Formation consists of grey-brown-green colored, weak-medium strength sandstone-siltstone and clay stone, with fine-mid size grained thin-thick layers, weathered and occasionally over weathered, altered, curly, with faults and very frequent fractures, and volcanic Andesite and Diabase dykes intersecting these along faults and weak zones. Related geotechnical parameters are enclosed in Table 3.

3.2.5 *Baltalimani tunnel (Bebek-Baltalimani)*

Paleozoic rock formations representing a continuous sedimentation from lower Devonian to Middle Carboniferous are found on the tunnel alignment. These are locally covered by 5 to 15 m thick Neogen Deposits of clay, silt, sand and boulders of 2–30 m. Devoninan and Carboniferous Buyukada and Thracian Formations, which form the bedrock, are cut by andesite and diabase dykes of 1–30 m thickness.

Buyukada Formation of Upper Devonian Age consists of micritic and nodular limestone and carbonate rich shale. It is strongly folded, little jointed and has massive appearance. The joints are generally vertical to bedding and have vertical to subvertical dip.

Thracian Formation as indicated above consists of mudstone, shale-greywacke and conglomerate units. It is closely jointed and strongly folded. RQD in Thracian Formation is comparatively low. The geotechnical parameters for the rock masses on the alignment of Bebek-Baltalimani tunnel have been provided in Table 4 enclosed.

3.3 *Geotechnical measurements*

All the tunnels constructed in Istanbul are naturally located under highly populated areas.

Hence, geotechnical measurements have been carried out at critical sections along the tunnel alignments during construction, in order to minimize expected settlements following the excavation and to be able to implement possible alterations in the support system without losing time. During the monitoring of these measurements below equipment/systems have been used as necessary:

1. Approach bolts installed within the tunnel
2. Settlement bolts installed outside the tunnel
3. Rod/Tape Extensometers
4. Inclinometers
5. Pressure Cells

Most critical sections subject to geotechnical measurements are shown in Figure 2 for Incirli Tunnel and in Figure 3 for Kisikli Tunnel. Metro tunnels extend totally under Cumhuriyet and Buyukdere avenues, and Baltalimani Tunnel follows Bebek-Baltalimani coastal

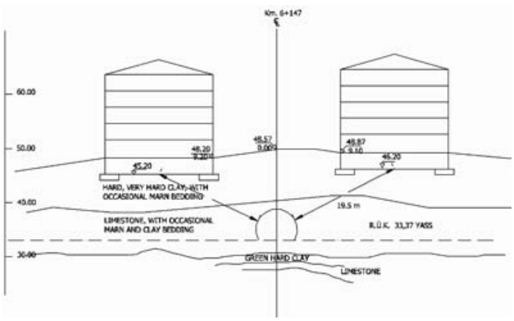

Figure 2. Typical crossing under buildings along Incirli Tunnel.

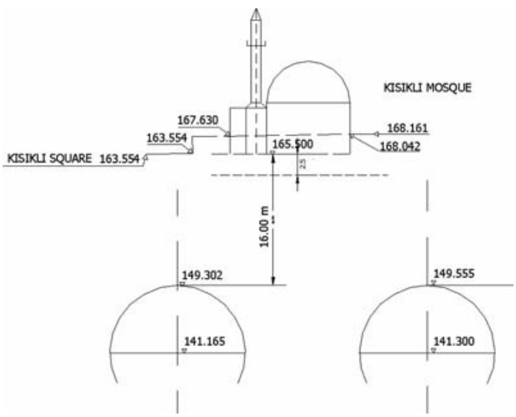

Figure 3. Kisikli Tunnel crossing under historical Kisikli Mosque.

road alignment and therefore these tunnels are less critical from subsidence point of view.

Maximum deformation values measured within the tunnel for Incirli Tunnel, Kisikli Tunnel, Tantavi Tunnel, Metro and Baltalimani Tunnels are presented in Figure 4 in a comparative manner.

For all tunnels finite element analysis was performed and in these analysis geotechnical parameters given in Section 3.2 obtained from laboratory results were used. In the finite element analysis NATM excavation steps and shotcrete was simulated and elasto-plastic material model was attributed to the rock and lining concrete (3). In the calculations permissible maximum deformation for Incirli, Kisikli, Tantavi, Metro and Baltalimani tunnels were assumed to be 100 mm. In finite element analyses, as rock masses remained within the elastic zone it was possible to compare the calculated values with some empirical methods developed based on the elastic theory. Finally, it was observed that good correlation has

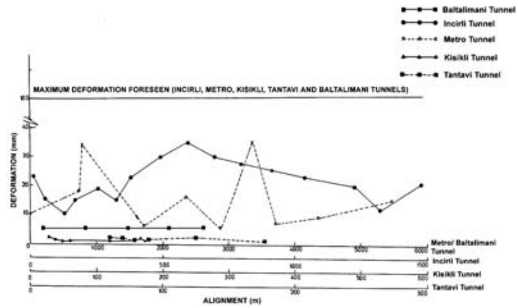

Figure 4. Comparison of convergence (deformation within the tunnel) values obtained with measurements at Incirli, Kisikli, Tantavi, Metro and Baltalimani tunnels.

been achieved between calculated, numerically checked and measured convergence values (5).

4 SITE OBSERVATIONS

1. For tunnels constructed under urban settlement zones, precise geotechnical measurements have been undertaken to avoid damage to existing buildings and to secure safety of excavation. Consequently with the utilized methods of geotechnical monitoring any serious damage to important historic and commercial buildings have been avoided.
2. Apart from Incirli Tunnel other transport tunnels constructed in Istanbul have been generally driven in Paleozoic Formations. Geotechnical monitoring of these tunnels usually display similar behaviour in spite of major local variations. On the other hand Incirli Tunnel is constructed in comparatively younger Upper Miocene formations. Whilst geotechnical parameters of these formations are still suitable for tunnel construction, they are more problematic compared to Paleozoic formations.
3. During Incirli Tunnel construction which was driven in weak to very weak rock conditions and under settlement areas with shallow cover, subsidences prospected in design and actual monitoring of excavation have been found in conformity to a great extent.

   In Incirli Tunnel, parallel to the lithology and structural situation, at the tunnel crest centerline with thin cover and thick fill, maximum 83 mm subsidence and 35 mm convergence values were observed. During Incirli Tunnel excavation shotcrete was applied continuously at the forepoling and face, apart from limestone dominant sections.
4. In Kisikli and Tantavi Tunnels excavation, different excavation techniques were implemented due to highly variable rock and soil conditions and this had negative effect on tunnel advance rate.

During the excavations of Kisikli and Tantavi tunnels most critical factor was highly variable rock mass conditions. At Kisikli Tunnel very hard quartzite and softer but excessively weathered and altered volcanic dykes have been encountered. Excavation in clay stone and siltstone encountered apart from quartzite and volcanic dykes was relatively easier and it has been possible to work with shovel. Stand-up period of these formations was sufficient enough for shotcrete implementation. However, collapses have been encountered at jointed and faulted sections.

During the excavation of Kisikli and Tantavi tunnels geotechnical measurements were made with "tape extensometer" within the tunnels and tacheometer was used outside the tunnel along the axis. These monitoring studies have revealed very small values. (Maximum 6–7 mm deformation). However, it was observed that sudden settlements occurring until installation of measurement bolts for extensometers could not be accounted for successfully at site.

5. Significant surface settlements were not encountered at the surface along the alignment of Istanbul Metro Tunnel. However, within the tunnel, deformation up to maximum 35 mm was measured (6). In general maximum deformation values were obtained from horizontal readings close to the tunnel invert.
6. A choice of full-face type tunnel boring machine for Baltalimani tunnel was proven to be wrong under severe ground conditions and pulled back after a drivage of 1230 m and replaced by a shielded roadheader. This was due to insufficient geological investigations during the bidding stage and the subject is still on the legal dispute involving claims both from the contractor and the client. After utilization of shielded road headers no significant settlements have taken place on the surface and only negligible convergence displacements up to 5 mm have been encountered on the tunnel periphery. No serious collapses have also taken place. Subsequently, the tunnel has been successfully driven to completion with advance rates of 5 m/day on average.

5 CONCLUSIONS

Since 1985, parallel to the development of infrastructure schemes in Istanbul, approximately 30 km tunnels with varying diameters between 2.2–13.6 m were constructed in Istanbul at separate locations for transport and environmental purposes and a major data has been accumulated regarding construction problems, advance rates and cost aspects of these tunnels. This data base could provide valuable insight in particular for the metro construction prospects, which will be expanded all over Istanbul with minimum 200 km length during this decade. The accumulated data also presents valuable insight to underground constructions to be constructed at various parts of the world under similar conditions.

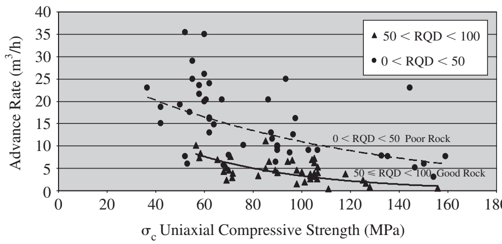

Figure 5. Advance rate of Tunnels with respect to RQD and uniaxial strength of rock masses.

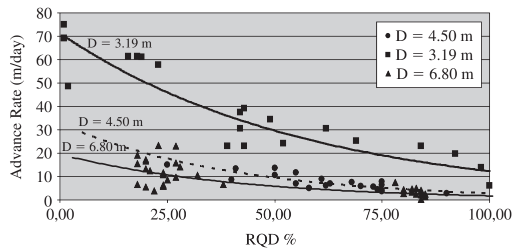

Figure 6. Advance rate of Tunnels with respect to RQD and diameter.

Relevant construction data could be summarized below related to infrastructure tunnels constructed in Istanbul:

1. The most appropriate and economic solution for small/medium diameter sewerage and metro tunnels in Istanbul is to utilize mechanized methods and by using "shielded roadheader". In highway/railway tunnels incorporating variable and big excavation diameters and in cases of variable soil conditions, implementation of the New Austrian Tunnelling Method has revealed good results.
2. Advance rates and costs of metro and sewerage tunnels are given in Figures 5, 6 and 7 with respect to rock strength and quality and diameter of tunnels. As can be seen in Figures 5 and 6, tunnel advance rate in general is affected by the rock mass quality (2). For good rocks with RQD $\geq$ 50, the advance rate is observed to be around 3–5 m/day and in medium to low quality rock with $0 < \text{RQD} < 50$ the advance rate is found to be 13–15 m/day in the rock masses frequently encountered in Istanbul.
3. Problematic sections in tunnel excavations are generally very hard diabase dykes or highly fractured sections incorporating large blocks. Water

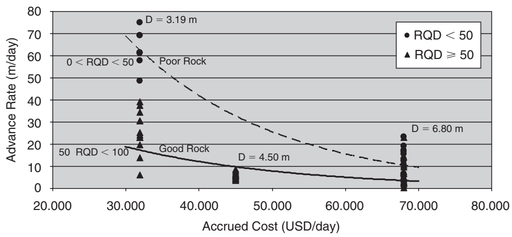

Figure 7. Accrued costs of tunnels with respect to advance rates.

ingress is generally at very low or controllable level.

4. Cost data summarized in Figure 7 have been derived from general construction statistics obtained from the various tunnel excavations discussed above including the support systems implemented. Based on the data in Figure 7 and taking into account length and advance rate of excavation the tunneling costs can be calculated as a rule of thumb (Diameter $\times$1000 \$/m), as an average and empirical approach. Considering that rock masses encountered in Istanbul tunnels are in general of medium strength and quality, above general and empirical approach may even be adopted in calculating the approximate costs for other tunnels elsewhere in similar ground conditions, as well as being very beneficial in forecasting approximate budget for future infrastructure tunnels in Istanbul.

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Underground Space Use: Analysis of the Past and Lessons for the Future – Erdem & Solak (eds)
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Underground space use in Ancient Anatolia: the Cappadocia example

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ABSTRACT: Globalization enabled people to learn about the world in which they live in than ever before in the past. Today's global culture is determined by a rapidly growing communication network and advancing technologies. As a consequence, sustainability of local natural environments and cultural heritages becomes a global issue, while local sites become learning grounds for global communities. As one of the best example of man's symbiotic relationship with nature Cappadocia, located in Central Anatolia, offers many possibilities and lessons for underground habitation. This paper aims to discuss the ancient settlement of Cappadocia unique for its creative subterranean space use. The focal points of discussion are the physical geography and dominating forms of underground space use.

1 INTRODUCTION

Globalization enabled people to learn about the world in which they live in than ever before in the past. Today's global culture is determined by a rapidly growing communication network, expanding extend of knowledge and information and advancing technologies. As a consequence, sustainability of local natural environments and cultural heritages becomes a global issue, while local sites become learning grounds for global communities.

Due to the recent debates on energy consumption, cost efficiency and sustainability, new architectural strategies and trends emerged in the last decades. A greater awareness of energy efficient techniques in building design has developed and have been applied. Many alternative strategies have been adopted that reflect an evolution towards a set of redefined values in construction, maintenance and utilization. This new architecture is usually determined by prefixes such as sustainable, ecological, environmentally conscious or symbiotic in the related discourse. Among these is a design approach most commonly referred to as earth sheltered or subterranean architecture.

Although underground space use has historical roots in many cultures, the reemergence of the phenomena in the western world has occurred for many diverse and interrelated reasons. Usually the reasons for going underground is more than one: to conserve energy, to overcome tight zoning regulations, to minimize the entity's impact on historic buildings in the vicinity, to enlarge open space capacity,… Rafael Moneo's recent expansion to the Prado Museum, Sverre Fehn's Ivar Aasen Center in Orstad, Renzo Piano's underground extension to the Morgan Library in New York, Herzog and de Meuron's media art facility in California could be listed as prominent examples of this approach. Other examples may include a number of works by David J. Bennett, Philip Johnson's underground art gallery in Connecticut, Kevin Roche's Winthrop Rockefeller Archaelogy Museum and I.M. Pei's underground extension to Louvre.

1.1 *Historical perspective*

Through the course of time, one may find many examples of the situation where underground spaces provided potentials to meet shelter needs of man. Protection from predators and climatic conditions, lack of building materials were among the reasons for going underground. In 4000 B.C. inhabitants of the Banpo site in China lived in semiunderground pit dwellings with A-frame roof supporting a thin layer of soil. The thermal advantages of the earth's temperature moderation in a harsh climate, together with an ability to create shelter with minimum materials, energy and mechanization consequented in the mass construction of these caves to our day (Carmody & Sterling, 1985).

Between second and fifth centuries, Romans adopted a similar approach in Tunisia to overcome the disadvantages of the extremely hot and arid climatic conditions. The outcome was the below-ground atrium dwellings constructed at Bulla Regia. Other examples of the earth-integrated vernacular architecture in the Mediterranean region exist in the Gaudix region of Spain, in Sicily, and on the Isle of Santorini, Greece. Indian cliff dwellings of Mesa Verde, Colorado, subterranean houses of the central Australian desert and mined houses of South Australia set other examples. Among these examples the Cappadocia region in Anatolia is a valley dominated by cave buildings (Carmody & Sterling, 1985).

2 SUBTERRANEAN ARCHITECTURE OF CAPPADOCIA

As one of the best example of man's symbiotic relationship with nature Cappadocia, located in Central Anatolia, offers many possibilities and lessons for underground habitation. It is very interesting to observe a variety of answers to the question "why go underground?" dating back to the days of Byzantine Cappadocia, 400 A.D. The terrain was transformed within the hands of its inhabitants who hollowed out the soft rock into churches, houses and monasteries, evolving an entire architectural tradition from its unique material. The outcome is fascinating to the eye; it is as if nature and man competed to create a special terrain, as Charles Texier stated in 1882, "… *before descending from the plateau, which dominates this territory, I paused for a while, amazed at the spectacle before me. I know of no other corner of the world where there exists so striking and remarkable a phenomenon*." (Rifat, 2000).

2.1 *Geographical conditions and topography of the region*

Cappadocia, characterized by the natural features and subterranean architecture has been proclaimed a world heritage site by UNESCO in 1985. The story of this unique region covering almost 10000 hectares goes back far in time to Neocene, around 25 million years ago (Tuncel, 1998). The forces inside the earth started to form the cracks in the crust, through which molten magma came out, thus creating the volcanic mountains in the vicinity. They continued to erupt, producing the layer of volcanic tuff which gradually erupted into the unique landscape of our time. Volcanic emissions of lava and solid material buried the entire region under horizontal layers of widely diverse hardness, color and depth. Wind and rain then set to work to sculpt the rock, creating the strange pinnacles with their caps of more durable rock. This is how "fairy chimneys" were formed (Tuncel, 1998).

There exists a variety of shapes and sizes, but it is not only the diverse shapes of these formations that are unique to the region. The diversity of color is no less remarkable, ranging from white, yellow and pink to grey and black, depending on the varieties of rock in each area. Where white rhytolitic tuff is abundant, and in the absence of a surface covering hard rock, the result is badlands topography, determined by sharp or rounded ridges forming white waves over large areas. Yellowish white columns occur in areas of rhytolitic tuff where red and pink columns in areas of red rhytolitic tuff. When man started to settle in the area, they took their cue from the wind and water and began sculpting the soft tuff to create shelter. The rock cones formed practical and versatile houses when the inside was carved.

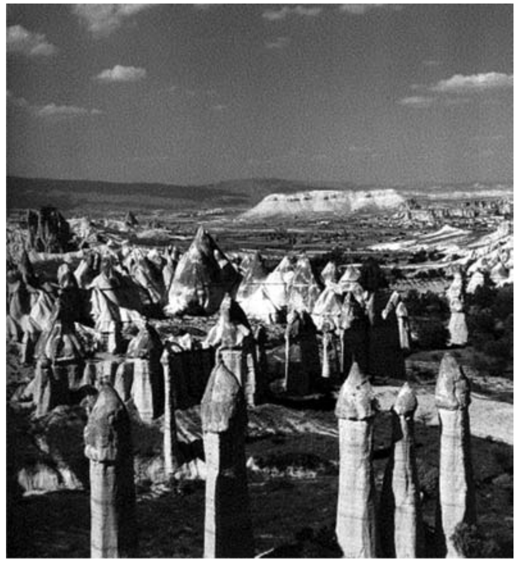

Figure 1. Pinnacles of Cappadocia.

2.2 *The Byzantine times*

The Byzantine period of Cappadocia starts in 395 A.D. when the region was inherited by the Eastern Roman Empire. The fame of Cappadocia today rests to a large extend on its Byzantine period rock churches and chapels. The volcanic terrain enabled a unique subterranean architecture carved from the soft tuff cliffs and pinnacles of the region. But although the carver masters adopted a different method of construction, they created spaces simulating the interiors of buildings constructed with stone and mortar. They modeled columns, vaults, arches and other architectural elements of their time. These elements did not have any structural function but were mere symbolic elements.

Instead of the classical definition of architecture as the creation of spaces for determined functions by

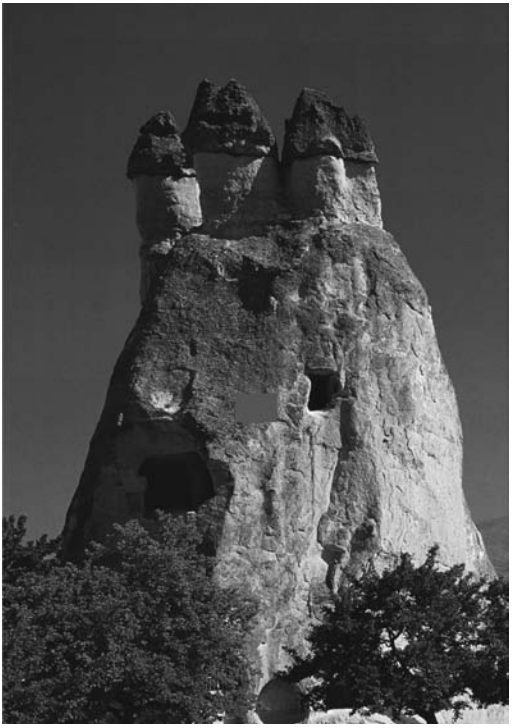

Figure 2. Monk Symeon's cell, Zelve (Akyürek, 1998).

Figure 3. Monk Symeon's Cell, Zelve. The small cell connects to an upper room. There is also a chapel on the lowest level (Akyürek, 1998).

enclosure with structural elements, here the interior space is created within solid matter by hollowing it out (Akyürek, 1998).

A large space could be completed in a couple of months, while the carver masters or architects did not require the complex knowledge required by their counterparts. The only limitation for the carver architect was the size of the rock mass that he was carving to form the spaces (Kostof, 1972).

2.3 *Dominating typologies*

In Cappadocia, one may find a variety of typological entities that differ in size and complexity: hermits' cells, dwellings, churches, monasteries and connected underground settlements. Inner rooms and spaces also vary in shape. The main idea is to carve out spaces within the limits of the pinnacles and stones, thus modifying the plans to fit the shape of the stones to be carved were a common practice among the carver masters. Consequently, each of the rock building was unique and none conformed to strict formal standards of Byzantine architecture.

The carving methods enabled Cappadocian masters to imitate the decorative elements of structural architecture by carving them along with the interior space. Although most of the buildings display no architectural features from the exterior, the interior spaces are decorated by frescoes or sculptured ornaments.

2.3.1 *Hermits' cells*

The simplest architectural form in the region is that of cells where hermits lived a reclusive existence. These carved cells or chambers contained niches to serve a variety of functions. They are usually devoid of decorations. The rocks which formed the landscape became dwellings for the monks when hollowed out.

2.3.2 *Churches*

Although there is a number of surviving subterranean architecture examples in the region, the highest percentage consists of religious buildings. In Cappadocia it is possible to find examples of religious buildings from early Christian to late mediaeval periods. However the architectural tradition flourished through the course of time and most of the existing churches highly decorated with frescoes date from the late tenth and early eleventh century. There are five main types of church in Cappadocia, single nave churches, double nave churches, basilicas, cruciform churches, and cross-in-square plan churches (Akyürek, 1998).

Single nave churches consist of a nave and a large apse. The main space is rectangular in plan and roofed by a barrel vault. Access to the churches is through a single door in the shorter west wall. The apses are usually higher than the nave, and delineated by a screen carved from the rock. There exists a rectangular rock altar in the apse.

Double nave church plan was widely implemented during the Byzantine Empire. The plan type is common for both rock and structural churches of the era. The naves are separated by a series of small openings or arcades of pillars and arches.

Basilicas have rectangular plans that are divided into three or more aisles by a series of columns. The middle aisle is usually the widest and highest, ending with a semicircular apse. The basilica plan was used in early rock architecture in Cappadocia for cathedrals and important monastery churches. Carving the particular plan scheme was relatively difficult and so was reserved for religious entities of higher importance.

Cross-in-square plan type is typical of medieval Byzantine architecture. Following the tradition of the Great Palace in Constantinople, this plan spread all over the empire. The dome over the square central area is supported by four columns, and vaults on four sides of the dome form a cross plan roof.

There are also churches with irregular or amorphous plans which were probably dictated by the rock structure of the site.

2.3.3 *Monasteries*

From seventh century on, vast number of monasteries and chapels were formed in the Cappadocia terrain. Monasteries of Cappadocia are complex structures including refectory, dormitory and church all carved into the stone. The monasteries were comparatively small edifices, largest having a capacity for fifteen to twenty monks at a time. The monasteries were unique in their organizations as well: they combined the concept of a collective life under the lavra system unique to the East, and the more orthodox idea of individuality and isolation from others. This principle was reflected in the plan schemes as well. There are allocations both for private cells and spaces for communal functions.

2.3.4 *Tombs*

The sacral buildings of Cappadocia were frequently used for burials, a custom that has its roots in medieval Byzantine times. The dead were buried in either church floor or the wall niches in the narthex.

2.3.5 *Houses*

The housing tradition of Cappadocia goes beyond the Byzantine era. It is not easy to estimate the start of the subterranean architecture. The inhabitants of the terrain discovered the means of dealing with the rock before the Christian era. For them, carving out houses was preferable to ordinary construction methods. The houses created provided better inner climatic conditions and easy to maintain. They were also easy to defend in times of war or civil unrest.

2.3.6 *Multistory settlements*

Besides churches and monasteries constructed in the vicinity, there are also multistory settlements created by the Byzantine carver masters. These settlements are entirely underground and formed by uniting or linking a series of underground rooms or chambers. The plan schemes resemble complex labyrinths. Although there are a number of such settlements, the most impressive ones are at Derinkuyu and Kaymakli (Ousterhout, 1995). Like their above ground counterparts these settlements also contain a series of functional spaces allocated for residential, religious, administrative and public functions.

3 CONCLUSIONS

Many societies exist with their codes and customs, just as are transitory sorts of places. Architecture does not only build to shelter communities, it also builds powerful means of expression and communication. In Cappadocia, one does not only experience a powerful architectonic existence but also the cultural message conveyed through the transformation of a natural setting to a sound habitat. Remarkable identity of the setting is the consequence of a very strong symbiotic relation. Not only the formation of architectonic enclosures by working with the physical and material potentials of the terrain but also the realization of a dichotomy between private and public/religious life possess lessons for future developments.

Cappadocia sets a unique example of subterranean settlement with its dominating architecture and spectacular landscape. The carver architects or masters of the region adopted Byzantine forms and features but redefined them while creating the unique spaces of the dominant architecture. One may find architectural spaces in a variety of scales ranging from single rooms to totally integrated subterranean cities. The spaces that could survive till our day have attributes that provide lessons to contemporary practitioners as well: the spaces provide sound insulation against excessive weather conditions, the inner climatic conditions of the caves and their low levels of humidity provide perfect storage potentials for agricultural products and wine, the caves are also possess higher potentials to withstand earthquakes. Moreover, the underground architecture of Cappadocia sets a unique example for man’s symbiotic relation with nature, while constituting a prominent example of highlighting local values and creating environmentally conscious settlements and spaces. In a way, analysis of the precedent provides many lessons for the future.

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Explosion tests in drainage pipes of long tunnels

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ABSTRACT: Accidents due to petrol leaks have caused serious accidents in the drainage pipes of urban sewer systems in the past. Ten years ago, a petrol train ran off the rails and petrol escaped into the Zurich sewer system. Several hundred metres of the urban sewer system were destroyed due to an explosion of the petrol gases.

To avoid similar accidents in the new Swiss AlpTransit base tunnels, which are 35 km and 57 km long, a protection system has been developed and tested using real explosions.

In the base tunnels through the Swiss Alps, all fluids and water leaking out of the trains will be collected in waste water drainage systems. These drainage systems include maintenance openings measuring 100 m, which will be a weak point in case of an explosion in the drainage system.

The probability of an explosion of this sort in the drainage system is quite low, but should such an accident occur, subsequent trains could be affected and fire fighters could be trapped.

A constant water flow and the use of siphons should prevent propagation throughout the entire drainage system. The efficiency of methods of this kind has been proven in real-scale explosion tests over a length equivalent to one section of the drainage system. The aim of these tests was to prove that an explosion could not cross a maintenance shaft so as to affect a subsequent drainage section.

The tests used methane gases which were ignited in a simulated shaft. Different concentrations of methane were used to simulate different levels of explosion and detonation. The pressure of detonation and the propagation velocity of the flames were measured (frequency: 5000 Hz).

With a water flow of 5 l/sec in the drainage system, the explosion was cooled down and suppressed after about 50 m. Rapid increases in pressure and flame velocity were documented in each test. After reaching a certain value, the explosion was suppressed and the flames were extinguished. It emerged that serious explosions would be suppressed earlier than those of lower intensity.

Siphons are not necessary in order to halt the development of an explosion.

1 INTRODUCTION

The release of liquid and combustible hazardous substances such as petrol inside a tunnel can also result in explosive atmospheres in the tunnel drainage pipes. Accidents involving escaping petrol that reaches large-volume inner-city sewage systems indicate that an explosive event in the pipes can have catastrophic consequences.

In long rail tunnels, an event of this sort may cause an explosive incident to spread below the subsequent train or among the rescue teams arriving at the site.

To prove that the drainage system equipment selected for the AlpTransit rail tunnels is adequate for the safety requirements, explosion tests for this purpose were carried out on pipe sections which correspond to the dimensions of the AlpTransit installations.

The aim of the tests was to furnish experimental proof that an explosion, which is triggered in a pipe section between two maintenance shafts, is unable to spread beyond the shafts and the immersion tubes located in them across longer sections of the drainage system in the tunnel installation.

A closely related aspect is proof that the shaft structures, including the immersion tubes, do not suffer any damage, which would reduce the required outflow capacities for the system at the dimensioned pressures of 10 bar (radial) and 20 bar (axial).

2 EXPLOSION MECHANISM

If petrol flows into the drainage pipes after an accident, the shafts and pipes of the drainage system will fill with petrol as a consequence. The petrol will flow out of the pipes into the shafts, where residues that vaporize are left behind.

If, due to any mechanism whatsoever, an ignition event reaches the pipe system that is filled with combustible/explosive gases, an explosive event will occur.

The term ‘explosion’ is used to group together all the processes involved in a sudden conversion of substances, regardless of whether a deflagration or a detonation is involved.

The explosive event itself consists of a pressure wave and a flame front. The explosion is only deemed to have been extinguished or interrupted if both the elements of such an event are halted.

The explosion process depends on various factors in this case:

- the composition of the explosive mixture
- the type of ignition
- the pipe length
- the pipe geometry
- the closure at the end of the pipe
- the design of the shaft (open, closed)

The potential effects of an explosion depend, in particular, on the concentration of combustible substances and the geometry of the system under consideration.

In long and extended systems, such as the drainage channels in this case, the combustion gases spread along the longitudinal axis and push the as yet uncombusted mixture in front of them. This causes an increase in the turbulence in the uncombusted mixture, and hence an acceleration of the progression of the reaction with an increasing combustion reaction. The pressure effects increase in direct proportion to the velocity of propagation of the explosion and in extensive systems; they may attain values of 20 bar to 30 bar.

The larger the diameter of the system, the longer will be the initial distance for an accelerated explosion and hence the transition to a detonation. This distance is also extended in the case of concentrations which deviate from the stoichiometric petrol vapour/air mixture.

3 TEST SET-UP

3.1 *General test concept*

The explosive event was triggered in a run of pipes leading into the shaft. In the first phase, the run of pipes carrying the explosion was selected with a length of approx. 30 m, and this figure was extended to approx. 72 m as the tests proceeded further. A real-scale simulation of the shaft is connected to this pipe. A short run of pipes is positioned on the side of the shaft facing away from the explosion. The explosion was deemed to have been interrupted if no flame was detected in this pipe run.

The explosive event was recorded along the pipe carrying the explosion, in the shaft and in the pipe leading away from the shaft, by means of pressure sensors in the radial and axial directions, and by flame detectors.

The tests were carried out in three phases:

- pre-tests
- tests on 30 m-long pipe sections, with and without constant flow
- tests on 70 m-long pipe sections, with constant flow

The pre-tests were used to determine the gas mixture concentrations which led to the application of the required pressures at the open end of the pipe, taking account of the gas mixture, the pipe length and the pipe geometry. In this way, the dimensioning event was reproduced on the unprotected pipe system.

Following this, tests were carried out on 30 m-long pipe sections and the corresponding shaft structures. Tests were initially carried out on dry pipes, and then in a further testing phase; a constant flow of 5 l of water per second was simulated in the pipes.

On the basis of the results obtained, it was decided to add a further testing phase in which a longer pipe section (approx. 72 m) was examined in front of the shaft.

4 RESULTS

On the basis of experience, above-stoichiometric methane–air mixtures produce more stable results, so the pre-tests were carried out with methane proportions of 10 and 10.5%. In this case, a methane content of 10% leads to pressure values which are significantly higher than the target values stated. Methane contents of 10.5 lead to pressure values which are at the lower limit of the target range of 10 bar (radial) and 20 bar (axial). For these reasons, a methane concentration of 10.3% was selected as the initial situation for the tests.

The first tests were carried out on dry pipes and water-filled shafts. As expected, the pressure in the pipe rose as the length increased. However, the expected pressures, which had also been proven in the pre-tests, were not attained. In the shaft itself, the massive extension of the cross-section combined with the shaft cover leads to a significant reduction of pressure. Only minimal overpressure is measured in the secondary pipe.

Various measures were implemented in attempts to attain the required pressures in the area where the drainage pipe opens into the shaft. Firstly, the methane concentration was gradually shifted towards the stoichiometric optimum. Even with 10% methane, axial pressures of only 7 bar were attained as compared to the pre-tests, where axial pressures of up to 40 bar had been achieved.

When the constant flow was introduced into the test set-ups, the attainable explosion pressures fell even further, to maximum values of approx. 1 bar.

However, a flame breakthrough occurred in the area of the shaft in all of the tests, meaning that the desired interruption of the explosive event was not achieved.

Since the pressure loss in the 30 m-long pipe section was significant, further tests were carried out with a longer pipe section (approx. 72 m).

In these tests on a long pipe section, conditions were ascertained in the initial stage of the explosion that were analogous to those in the preceding tests. However, in the further course of the pipe, there is complete extinguishing of the flame front. This effect is apparent with different gas concentrations, down to the stoichiometric optimum of 9% methane. In the shaft itself, no flames and hence no potential ignition events for the continuation of the explosion are detected. The tests suggest that as the ideal stoichiometric value is approached, and hence in the case of more violent explosions, there is faster extinguishing of the flame front.

The effect whereby the progression of the explosion can be braked and even halted by the constant flow in the pipes can only be explained as follows: the incipient explosion whirls up the water in the constant flow and disperses it over the cross-section of the pipe, thus cooling the explosion and preventing its development; in addition, it may be assumed that the water vapour may exert an inerting influence in certain circumstances. This principles has been successfully used by the mining industry for a long time in order to prevent the propagation of explosions in coal pits. Accordingly, decisive importance attaches to constant flow for the purpose of limiting explosive events in the drainage systems of the AlpTransit.

5 CONCLUSION

The tests that were carried out showed that explosive pressures of the order of significantly more than 20 bar may occur in dry pipes, even in short pipe sections of approx. 30 m. When a constant flow is introduced into the experimental set-up, the explosive pressures fall below 1 bar. However, a breakthrough of the flames through the immersion tubes is registered in the shafts. The low pressure level is not exceeded, even with gas compositions which are optimal in terms of explosion technology. In a second phase of testing, the explosion behaviour in longer pipe sections (approx. 72 m) was examined. In all the tests carried out, it emerged that the explosion flame – which is critical for the continuation of the explosion – is extinguished within a pipe length of 60–70 m. Accordingly, critical importance is attached to constant flow for the control of explosive events in the drainage pipes of the AlpTransit.

The tests carried out were performed under the most optimal conditions for an explosion process. In an actual incident, such conditions will only be present for short periods of time, which are probably in the range of seconds. Likewise, no consideration is given to the consumption of oxygen due to the burn-off of hydrocarbons in the pipes, which will make it even less probable that optimal conditions for an explosion would be present.

Underground Space Use: Analysis of the Past and Lessons for the Future – Erdem & Solak (eds)

A multi-level tunnel for Amsterdam Zuidas

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ABSTRACT: The Zuidas (South-Axis) will develop into the second city centre of Amsterdam. The station will become the fifth station of the Netherlands. Several alternatives for the development of both infrastructure and urban habitat have been studied. Till recently, only two alternatives remained: the 'Dike' model, developed from the point of view of the infrastructure, but frustrating urban development, and the 'Dock' model, developed from the point of view of city planning, but neglecting the infrastructural and also the real estate requirements and therefore leading to a long building time. This paper will discuss a new design principle for the 'Dock' alternative: Stapeldok (Multistorey Dock) [1]. It is based on an underground longitudinal stacking of heavy-rail and light-rail, forming a multi-level tunnel. Real estate can be built on top of it, up to the desired height of one hundred meters. This design principle is being worked out presently as a third alternative. It has several benefits for both the infrastructure and the urban habitat that will be developed around and above it, including improved public quality [2], [3], more efficient phasing and lower overall costs.

1 INTRODUCTION

The Amsterdam Zuidas (South-Axis) lies between two southern districts of Amsterdam. It consists of a bundle of infrastructure for trains, metros and cars (Figure 1). There is a small station at present: Amsterdam Zuid/WTC. The present railway tracks will be augmented with new ones for the high-speed lines to both France and Germany. The station will become the fifth station of the Netherlands [3].

Planning for the development of the Zuidas began in 1994 and two alternative designs have remained for tracing out this bundle of infrastructure through the city. In the most ambitious alternative, the 'Dock' model, the infrastructure is built over with one million square meters of real estate (Figure 2). This paper

Figure 1. Zuidas at present.

Figure 2. The Zuidas in future with infrastructure built over.

will present a new design principle based on a multi-level tunnel. This multi-level tunnel model optimises the phasing, overcomes important technical difficulties of existing alternatives and has positive financial consequences. Beside these advantages, the public quality and the flexibility are improved.

2 EXISTING ALTERNATIVES

At present there are two alternatives. The first alternative is the 'Dike' model that plans to extend the infrastructure on the existing embankment and to develop building sites only on both sides of the infrastructure. The second alternative is the 'Dock' model that plans to bring the entire infrastructure for heavy-rail (six tracks), light-rail (four tracks) and the motorway (four lanes on both sides) underground at level minus one to enable overhead construction of one million square meters of real estate extra. The latter alternative of building over railway tracks is the largest in its sort in Europe.

2.1 *'Dike' model*

In the 'Dike' model (Figure 3), the extension of the infrastructure takes place on the existing embankment. From the point of view of the infrastructure this is obviously the simplest model, but the infrastructural bundle is a wide, lasting obstacle in the city, frustrating the development of a new city centre. New real estate can only be developed on both sides of the infrastructure and the development of housing is limited by regulations for noise and air quality. This will lead to a mono-functional office development that lacks urban quality.

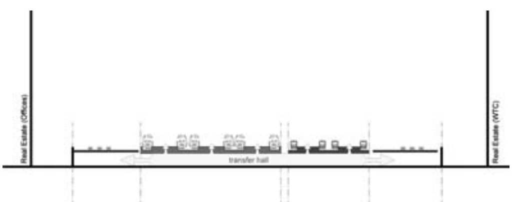

Figure 3. Infrastructure extended at plus-one level, the railways on the left and the metro on the right.

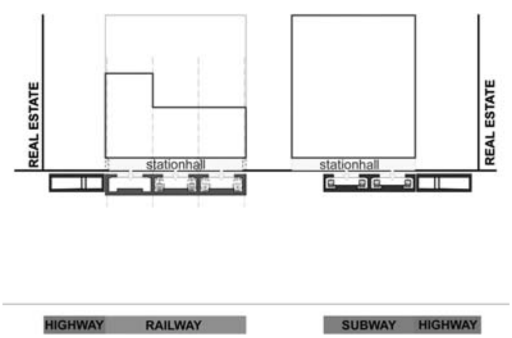

Figure 4. Infrastructure located at minus-one level, the railways on the left side and the metro on the right side.

2.2 *'Dock' model*

The 'Dock' model (Figure 4), with the infrastructure underground, at minus-one level, has obvious urban advantages over the 'Dike' model [4]. Up to one million square meter of real estate can be built over the infrastructure, thus connecting the city districts on both sides of the infrastructure bundle on existing street level and forming a new city centre. Compared with the 'Dike' model, this model is much more expensive. The extra costs must be earned back by real estate developments over the tunnels. Apart from the elevated price, this model has some important disadvantages.

First of all, this model contains two stations, one for the heavy-rail, one for the light-rail, approximately a hundred meters apart, with an urban axis in between. This leads to streams of passengers, pulling their trolleys through the future city centre. Besides, two stations take up a lot of commercial space at ground level. Another serious problem is the lack of possibilities to realise car parks for the real estate developments, as the underground space is already occupied by the infrastructure.

A major problem is the phasing of the building activities. The difficulty is that the present infrastructure should stay in use during construction of the underground docks, leaving little space for the construction of new tunnels. The entire surface of the bundle is part of the building site. This leads to a building time of not less than eighteen years and several temporary infrastructural measures, including temporary stations for trains and metros, separated from each other by a building site. This is not only expensive, it also leads to inconvenience to the public. In addition, the building of the real estate above the infrastructure only starts after eighteen years in the centre, leaving a gap in the city heart and in the financing of the project, during that time.

3 'MULTI-LEVEL TUNNEL' MODEL

On the IABSE Conference '*Structural Engineering for Meeting Urban Transportation Challenges*' in Lucerne in 2000 an alternative was suggested for the Zuidas project, that is to build the metro tracks underground and the railway tracks at plus-one level, thus leaving a large area available where, without any restrictions, building of real estate could take place [2]. In order to bring the entire infrastructure underground and to overcome the disadvantages of the existing 'Dock' model, this alternative has recently been improved and worked out as a multi-level tunnel, the so-called 'Stapeldok' (Multistorey Dock) [1]. It leads to several benefits for both the infrastructure and the urban habitat that will be developed around and above. It is compared to the 'Dock' model in Figures 4 and 5.

3.1 *Infrastructure*

In the multi-level tunnel alternative, the motorway and heavy-rail are positioned at minus-one level, while light-rail is placed at minus-three level under the heavy-rail. This is possible, because light-rail can cope with much steeper slopes than heavy-rail and has shorter platforms. At minus-two level a transfer hall between heavy-rail and light-rail finds its place, the station hall being situated at ground level.

The infrastructure of the metro has to be weaved in and weaved out underground under the railway infrastructure on the sides. Above it all, real estate developments are constructed. The weaving of the infrastructure under the real estate developments is represented in Figure 6.

3.1.1 *Construction*

The excavation for the multi-level tunnel will be built with slurry walls and an underwater concrete slab with tension poles. The tunnel being approximately 26 meters deep, it will rest on the stable sand soil layers, which start at the same depth at that location. The slurry walls bear at the same depth of 40 meters to act as an excavation wall, as well. The tension poles will be brought in under water up to minus 60 meters, after the ground between the slurry walls will be removed and the slurry walls will be propped.

3.1.2 *Phasing*

An important advantage of the multi-level tunnel is the phasing of the building activities (Figure 7). It is possible to build the tunnel, while leaving the rail infrastructure intact during construction. This prevents the use of expensive temporary infrastructural structures. The rail infrastructure can be migrated to the new positions without an intermediate stage. Also, there will be no building site between the different means of transport during most of the construction process, so changing of transport modality will be convenient even during the building stage. The building time of the infrastructure can be reduced from eighteen years of the 'Dock' model to thirteen years only. The roads will be brought underground both sides of the area to make space for the construction of the rail infrastructure.

Because the phasing is much faster, the development of real estate can set off sooner, starting from the new city centre, which will be available much earlier. This has important positive effects on the finance of this project because benefits can be realised much earlier.

3.1.3 *Flexibility*

Future infrastructural needs can be met in an easy way. This applies both in longitudinal direction, where a third light-rail platform can be added, apart from infrastructure possibilities under the urban axis, as for the transverse direction, where it will be possible to add a crossing light-rail line in the future at minus-two level, next to the transfer hall at that level.

3.2 *Urban surroundings*

The multi-level tunnel has important positive effects on the urban surroundings. Real estate, up to a height of one hundred meters, can be built at any desired place on top of the slurry walls of the longitudinally stacked tunnels, as these walls are present over the entire length of the tunnels.

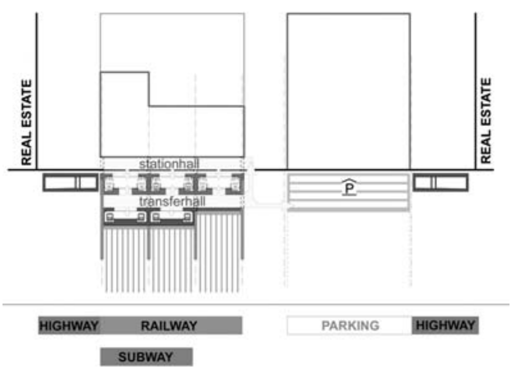

Figure 5. 'Stapeldok': a multi-level tunnel alternative.

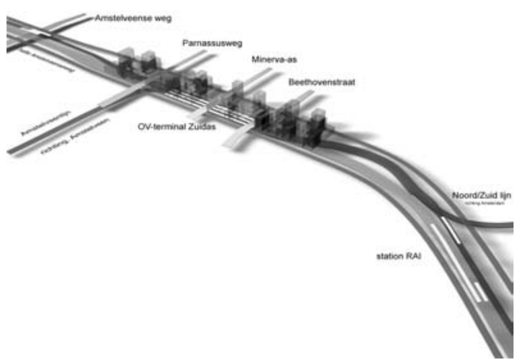

Figure 6. Impression of the longitudinal stacking in the 'Multi-level tunnel'.

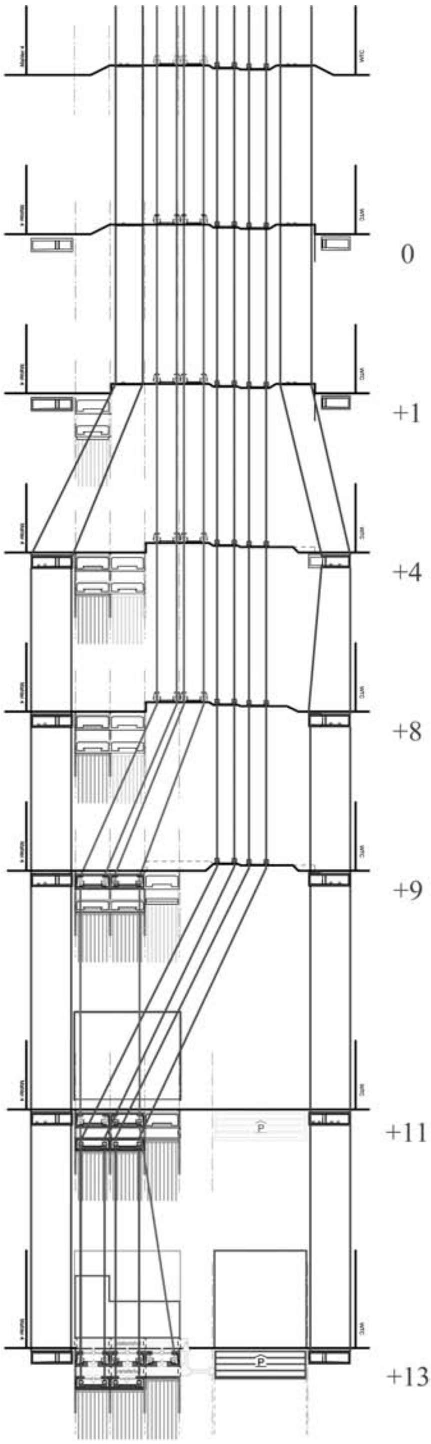

Figure 7. Phasing of the multi-level tunnel.

On the parcels beside the tunnels, building of real estate is not restricted anymore and car parks can be freely built under the real estate.

3.2.1 *Construction*

The longitudinal walls between the tunnel partitions at 21.60 meters centre to centre will spread the weight of the real estate on top of it, so it is possible to build on top of these walls without additional foundation. Of course a 'transfer' structure will be needed between the office grid of 3.60, 5.40 or 7.20 meter, commonly applied in the Netherlands and the 21.60 meter grid of the infrastructure [2],[5].

Next to the multi-level tunnel, no 'transfer' structure is required, but a standard foundation has to be made instead.

3.2.2 *Quality*

The quality of the real estate increases, as underground car parks will be available. There will also be more shopping space at ground level in the center of the project next to the station, as the two stations are integrated. The construction of the new district can start from the centre and gradually grow, thus creating high urban quality from the start on.

A secondary effect is created for existing real estate beside this building site. A lot of buildings have already been developed and some are under development. These all profit from a shorter building time of the infrastructure.

3.2.3 *Flexibility*

Half of the real estate can be freely developed. As the centre can quickly be built, it is possible to delay the rest of the real estate building activities till the moment when it suits best, without leaving an open city heart. Building on top of the multi-level tunnel hardly requires pre-investments, so there is no need to accomplish all the real estate within a short period of time.

3.3 *Integral public transport terminal*

The quality of the station will improve enormously compared to the existing 'Dock' alternative, combining all modalities within one station and thus making the interchange between transport modalities possible without leaving the station. The transfer hall that is integrated at minus-two level enables an efficient interchange in which transferring passengers don't have to gain ground level.

Creating an efficient integral public transport terminal means short transfer connections for a passenger between heavy rail, light rail and other modalities likes buses and taxis. By placing the infrastructure of heavy rail and light rail on top of each other, we are able to create a convenient arrangement. These modalities are linked with each other within the compact terminal. Passengers no longer need to cross the central axis road with their trolleys when transferring from one modality to the other, like in the 'dock' model. Transfer becomes quicker and more comfortable.

By introducing a transfer level on minus-two level, between the heavy rail tunnels and the metro tunnels, the passengers can overview the station within moments. They can see all levels and vertical transport equipment, like stairs, escalators and elevators and find their way to another level or exit easily. The tunnel walls between the tracks can be perforated to create more view and visual relation between the different platforms. The station will be visually larger; it will have a better atmosphere and will provide more social safety for the passengers.

The new integral terminal will be the first building on the Zuidas-site and will be the future city centre of this new area. The terminal will be a 'connector' between all future developments and between the two existing urban areas on the north and south side. Therefore it is important that all temporary stages in phasing this project will have a high urban quality. Because of the efficient phasing, the temporary situations during the construction of the tunnels and the use of the existing infrastructure will be of higher quality for the passengers, as the current situation will not change until the migration to the final position of the platforms.

3.4 *Financial aspects*

The 'Multi-level Tunnel' model has some lower investments compared to the 'Dock' alternative and some higher returns.

3.4.1 *Lower investments*

Although the tunnel infrastructure itself will require higher investments due to the depth of the excavation and the longer slopes to gain the depth of the light-rail station, overall investments will be lower. This is because on the one hand the building cost of the real estate on top of the multi-level tunnel will be lower, as no additional foundation will be required. On the other hand, the building cost of the real estate next to the multi-level tunnel will be lower, as no 'transfer' structure will be required between the grid of the real estate and the one of the infrastructure. The reduction of construction time reduces the interest expenses and financial risks substantially, as the time gap between profits and expenses diminishes.

3.4.2 *Higher returns*

As the building time and inconvenience will be reduced, the surrounding real estate will have higher rents. New real estate will also have higher rents, thanks to the increased quality of the station and its surroundings, the extra shopping space at ground level next to the station, the car park possibilities and the fact that the development will start at the station in the centre of the new city. Also, the benefits from the issue of the land will become available earlier. In addition, the return on investments of the real estate begins substantially earlier at the multi-level tunnel side, as no foundation has to be built there.

4 SAFETY

The fore mentioned 'Multi-level Tunnel' model has some points of attention, when compared with the 'Dock' model: Safety will be more an issue, because of the stacking of functions and as the depth of the station will increase. This applies mainly for the internal safety for the rail infrastructure. This is the safety of the passengers due to for instance fire in a train or metro, due to derailment or explosions. Escape routes play an important role, here, as well as the possibilities for the fire brigade and the first aid team to access the place of the accident. A quick study on these challenges indicates, that they can be easily overcome.

For external safety, that is the safety of the real estate due to accidents within the infrastructure underneath or vice versa, the situation is better than with the 'Dock' model, because only half of the real estate will be built over the rail infrastructure, when compared with that model.

In co-operation with the central government, the Rail Management Company, the city of Amsterdam and private investors, the possibilities of this 'Multi-level Tunnel' will be worked out in the next phase.

5 CONCLUSION

In the South of Amsterdam a second city centre is planned: Amsterdam Zuidas. In this project railway infrastructure is extended, also for the high-speed train, and up to 2.2 million square meters of real estate is planned, making it the largest project in its sort in Western-Europe. For this development, two models are under consideration, the 'Dike' model and the 'Dock' model. The first frustrates real estate developments and the latter not only neglects the infrastructural requirements, but frustrates real estate developments as well. For this reason a multi-level tunnel has been developed. This multi-level tunnel offers a substantial increase of quality and flexibility for both infrastructure and real estate. It also enables an extreme reduction of building time from eighteen to only thirteen years. This reduction of building time is possible due to improved phasing. It also reduces the interest costs and financial risks.

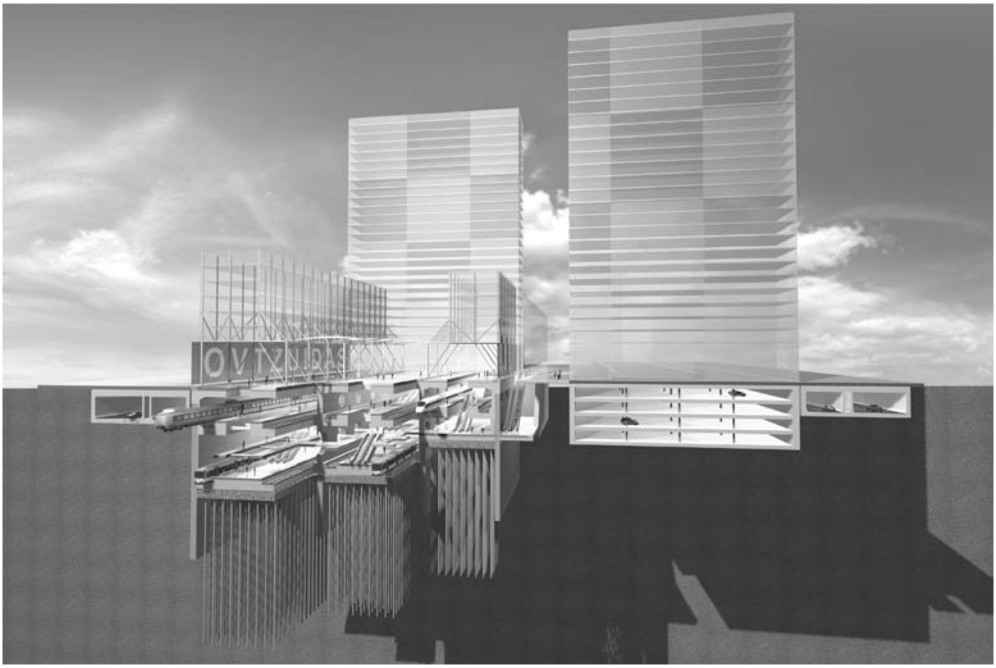

Figure 8. Impression of the 'Multi-level tunnel'.

The 'Multi-level Tunnel' (Figure 8) is a design principle in which knowledge of both infrastructure and real estate are combined enabling faster construction at lower overall costs and higher overall returns. Governmental parties, railway related parties, and private investors reacted positively to this alternative. That is why this new design principle is presently being worked out for the Amsterdam Zuidas.

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Planning research, development and design aspects on underground structures

Underground Space Use: Analysis of the Past and Lessons for the Future – Erdem & Solak (eds)
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Design of pressure tunnels of Ermenek hydro power plant

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ABSTRACT: The Ermenek hydro power plant, currently under construction, with numerous underground facilities represents a big challenge the rock mechanics and tunneling works, especially for the pressure tunnel situated in the very heterogeneous rock mass with exchanging of weak and hard rock units. The tunnel is excavated by TBM, lined with concrete lining and loaded with internal pressure of up to 13 bar (130 m of water). Design requirements and design considerations with selected solutions are presented.

1 INTRODUCTION

The Ermenek hydro power plant is located in southern Turkey and collects the water of the Ermenek river and its tributary, the Erik river. The double-curved arch dam is located in a narrow gorge and has a height of 210 m with a crest length of only 123 m. Most of the hydro power plant facilities, except the dam and the main powerhouse, are placed below ground. Grouting galleries with a total length of approximately 10 km, covering a grouting area of 1.5 km^2, are designed in 4 levels in a vertical distance of approximately 70 m in order to make a grout curtain in karstified limestone (Figure 1).

The waterway comprises a 8.1 km long headrace tunnel with 5.60 m internal diameter and 1.1 km inclined shaft. With a gross head of 327 metres, two vertical shaft Francis turbines with a capacity of $2 \cdot 150 = 300$ MW are situated in a surface powerhouse. The project also includes the power plant scheme Erik with the 4.2 km long headrace tunnel, vertical pressure shaft with 170 m height, cavern powerhouse with $2 \cdot 3.4$ MW horizontal Francis turbines and tailrace tunnel in the Ermenek headrace tunnel. Both power plants with an annual production of 1 014 GWh are foreseen to be completed in 2007.

Together with the grouting galleries, spillway tunnels, gate, surge and elevator shafts and access tunnels, the project forms a tunnel net of more than 30 km. The excavation of most facilities, except headrace tunnels is carried out by drilling and blasting according to the NATM. The open TBMs: The Ermenek machine with a diameter of 6.70 m and the Erik machine with a diameter of 3.93 m excavate both headrace tunnels.

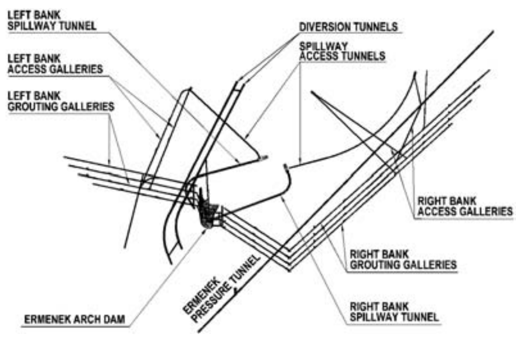

Figure 1. Underground structures in the vicinity of the Ermenek dam.

2 ERMENEK PRESSURE TUNNEL

2.1 *General*

The Ermenek pressure tunnel has length of 8.1 km with an excavation diameter of 6.70 m, is excavated by open hard rock TBM. The tunnel starts with an open surface intake structure, passing the gate shaft the surge shaft and ending in the valve chamber. TMB excavation starts from the valve chamber situated approximately one kilometre from the entrance portal. The gate shaft with excavation diameter of 8.0 m and height of 55 m will be excavated by NATM. Because of the tight time schedule the excavation of the surge tank with its 11 m excavation diameter and a height of 150 m is done independently of the tunnel excavation from the level above tunnel. After completing the

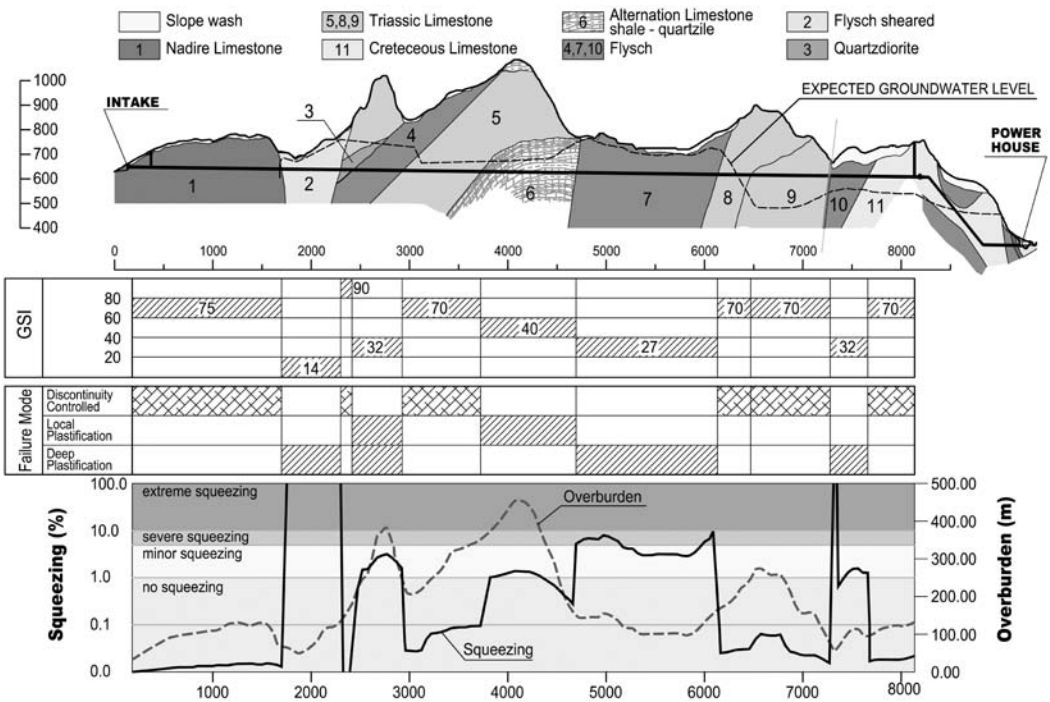

Figure 2. Longitudinal section of the pressure tunnel with rock behavior and squeezing potential.

TBM excavation connection shaft between the tunnel and surge shaft will be excavated.

Difficult geological conditions, especially several changings between the relatively hard and strong limestone with weak layers of flysch rock is a big challenge for the machine, but also for design of the primary and final lining.

2.2 *Geotechnical design of excavation and primary support*

The underground excavation and application of the support system is an interaction between ground properties and the selected excavation method in a defined period of time. Definition of the ground properties with its scatter represents a difficult task for the design of each structure in the ground. During the design period – independent of the design stage – type, characteristics and behavior of the ground are only known in points, along a line and/or only on the surface and together with other unknown factors, such as primary state of stress and groundwater, and can only be estimated by a characteristic value or by a range of values. Frequently, during the project phases with additional exploration works, the estimated values have to be changed or adopted. The design method is used that is flexible enough to allow these changes (Marence, 2003).

The design process started with definition of the regions with similar geological characteristics, so-called homogeneous regions. For these regions the geomechanical significant parameters (key parameters) and their characteristics are defined. The definition of these parameters is mostly done by geologists, however in cooperation with geotechnical engineers and is based on all obtainable information collected from the geology "in situ" and laboratory tests. Such information defines the ground type of the ground unit. For the Ermenek pressure tunnel geological conditions along the tunnel route have been separated in eleven rock mass types ("units") as shown in Figure 2. This un-oriented geology is put into relation with the orientation and geometry of the excavation and the information about groundwater, primary state of stress and other specific parameters are included. Also, qualitative geological and geotechnical information is transformed into quantitative parameters defined as fixed values or with their scatter. Then the key parameter and the RMR (Bieniawski, 1989) and GSI (Hoek, 2002) value for each rock mass type have been defined, All this information leads to the definition of the ground behavior type, which defines ground behavior including all important factors and excavation methods without considering the primary support and excavation phases. The categories of the

Table 1. Input data and rock mass parameters for representative rock mass types (flysch and limestone).

Rock mass type	Flysch: claystone, siltstone and sandstone	Limestone
Comp. strength* [MN/m^2]	18/1–130	80/80–140
GSI [–]	32	70
m_i [–]	7	10
Rock mass friction angle [°]	31	52
Rock mass cohesion [MN/m^2]	250	2200
Rock mass E-modulus [MN/m^2]	1400	27 000
Rock mass comp. strength [MN/m^2]	0.3	14

* Intact rock mass.

ground behavior types are defined based on recommendations of the Austrian society for geomechanics (ÖGG, 2001).

The next design step was the calculation of the strength-, frictional- and elastic parameters of the rock mass as demonstrated for the units 10 – flysch and 11 – limestone which are representative for the project (Table 1).

In order to do the calculation for heterogeneous rocks like flysch rocks, which are built up of alternating layers of sandstones, siltstones and claystones the uniaxial compressive strength and the constant m_i of the intact rock has to be defined at first. For the heterogeneous rock mass with strength values of components from 1–130 MN/m^2 it was done based upon the relative percentages of their occurrence. The "weighted average" rock strength (18 MPa) and m_i (7) have been calculated according to Hoek and Marinos (2000). In case of the limestone units (like unit 11) the laboratory values of the compressive strength ranged from 80 to 140 MN/m^2 and for safety reasons the lower value was selected as input parameter for the calculations.

The failure mechanism strongly depends upon the stress conditions. A rough estimation of the failure mechanism is based upon the ratio between rock mass strength and in situ stress Hoek (1999) or by defining a squeezing potential (Hoek & Marinos, 2000 and Barla 2001). The squeezing potential is defined as a ratio between radial deformation of unsupported tunnel and tunnel radius defined as a percentage. The simple closed form solution for circular tunnel in a hydrostatic stress field is seldom met in the field, but the method gives useful indication about the tunnel support requirements and possible problems (Marence, 2003). The squeezing potential is used as a first estimation of the failure mechanism, defining regions where discontinuity conditional failures or plastification and squeezing rock conditions can be expected (Figure 2).

Considering all of these data the possible failure mechanisms (ground behaviour types) can be defined. The Austrian society for geomechanics (ÖGG, 2001) distinguishes eleven different ground behaviour types. A unique calculation method covering all ground behaviour types does not exist. The calculation method, analytical and numerical, must be appropriate for the assumed behaviour type. Three frequently encountered failure mechanisms are: block failure, local plastification and squeezing rock conditions. A suitable model gives answer on suitability of selected excavation process and support measures and gives possibility for risk assessment (Marence, 1998). In case of structurally controlled instabilities kinematic models are used for surface and underground excavations. The condition of joints and their orientation in relation to the designed structure are decisive for stability and several software products are available for such calculations. In case of homogenous and isotropic rock mass conditions continuous models like finite element or finite difference models are used.

Based on the calculations the primary support system is defined. Support is defined taking into consideration requests of the TBM:

- installation of the support in the region L1 immediately after the head and in L2 behind the machine
- application of shotcrete in the L1 region is restricted to the support classes where immediate support and rock sealing is necessary
- support with steel rings as an immediate support in weaker rock mass

The support systems are grouped in 6 support classes giving smooth transition from class to class. The first three classes, M1–M3, define the support in the blocky rock mass such as limestone and the last three, M4–M6, for the much weaker flysch rock. The support class M5 (weak rock) is shown on Figure 3. The support consists of steel arches TH 36 on the stroke distance of 1.70 m, systematic bolting in L1 and additionally in L2 region with 3.60 m long grouted bolts and 5 cm shotcrete in L1 and additional 10 cm shotcrete in L2 region reinforced with two layers of wire mesh.

On site, after each excavation cycle, the geological conditions on face are compared with design values, the suitability of the designed system is checked and a decision about support class application is done. In case of over- or under-estimation of the designed support system, the system can be easily adopted to the actual circumstances.

Geotechnical design gives possibility for understandable and comprehensive design with the option of intervention and adaptation in each project stage, based on the new knowledge, and during the construction works according to the as found conditions.

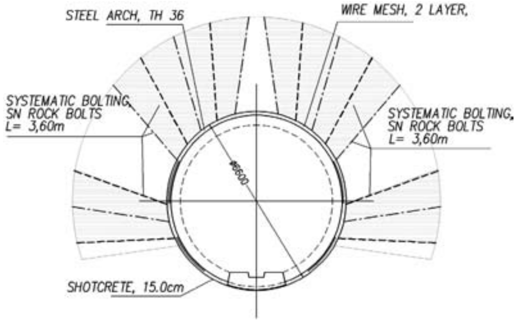

Figure 3. Primary support classes M5 for weak rock (flysch).

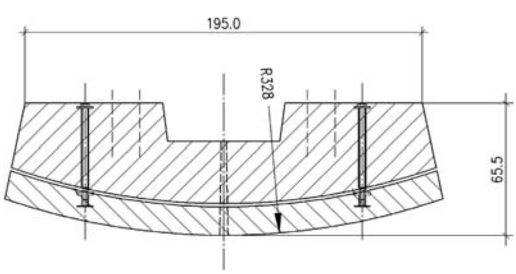

Figure 4. Two-part pre-cast concrete segment.

2.3 *Pre-cast invert segment*

The pre-cast concrete segment is used as a TBM sleeper. The segment is designed as a reinforced concrete element. Based on the selected final lining technology the segment is made of two parts – bottom and top part that are screwed together. The reinforcement design is made for separate parts (transport) and for the connected system (operation).

Five different segment types have been designed, distinguishing the technology and additional geometric conditions from the support requirements – gaps for the still rings in bottom part. A simple sketch of one invert segment type is shown in Figure 4.

2.4 *Final lining design*

2.4.1 *Reinforced or un-reinforced lining*

Main loading for the pressure tunnels is internal water pressure – static and dynamic pressure caused by maintenance of turbines and valves. The internal water pressure causes tensile forces in the lining. This, because of low concrete tensile strength, is an unfavourable loading and needs special treatment. Different methods to decrease, eliminate or overtake tensile forces are developed. In the Ermenek pressure tunnel two systems are selected: un-reinforced and reinforced concrete lining.

The plane, un-reinforced, concrete lining, as the most economic final lining, is adapted only for tunnels where only small tensile stresses in lining occurs during operation. In pressure tunnels subject to high internal pressures the plane concrete lining has to be pre-stressed in order to keep it free from fissures. The pre-stressing is made in form of grout injection in the gap between the final lining and the surrounding rock mass. In case that prestressing is not possible, tightening of the tunnel can be done by thin foils (plastic or thin steel lining), reinforced concrete and by high internal water pressure by classical thick steel penstock. By the Ermenek project sections where full prestressing was not possible the final lining is reinforced. A tight reinforced lining is achieved by special design requirements on lining and consolidation injections around the tunnel.

2.4.2 *Design based on Seeber theory*

Seeber (1984) defines the design method for unreinforced final linings of the pressure tunnels. The grout is pumped under high pressure into the contact joint between primary and final lining, a circumferential gap is opened and filled with cement grout. With the grout pressure exerted in the circumferential gap, the concrete ring is pre-stressed against the rock and the deformations of both sides are fixed by the hardened cement grout. To reduce adhesion between shotcrete and concrete and to achieve the circumferential gap opening the shotcrete separation layer is applied on the contact surface. In other to maintain the pre-stressing effect permanently, the rock mass has to be sufficiently firm and the rock stresses around the tunnel must be high enough to take the pre-stressing pressure. The final lining and the surrounding rock mass take the internal water pressure dependent on their stiffness. In the case that the pre-stressing by the injection is high enough, the lining will remain in compression during the operation and can be left unreinforced.

The selection of the injection pressure and optimal lining thickness is based on the simple graphic load-line method (Figure 5). The method is based on the assumption of the circular tunnel in homogeneous rock mass with full around radial pre-stressing. The injection pressure is limited to the concrete compressive strength and a minimal primary stress in the rock mass during the injection phases and by leaving the lining in compression during operation. An additional design condition is the groundwater pressure in case of an empty tunnel.

The same design philosophy as for the unreinforced concrete lining is used for the design of the reinforced lining. The reinforced lining is used where full grouting is not possible, because of too low overburden or too weak rock mass. The grouting with maximal allowed pressure – minimal initial stress – is applied. Grouting with pressure of 10 bars guarantee

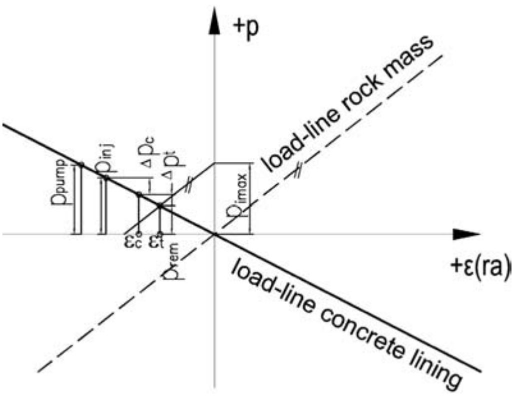

Figure 5. Load-line design method.

a contact between final lining and the rock mass after cooling the lining. The contact is necessary to achieve smooth and continuous force distribution between lining and rock and also to include the surrounding rock in the bearing of the internal pressure. Higher grouting pressures, applied if possible, additionally reduce the reinforcement amount in the lining.

To achieve the continuous reinforcement in the lining, the pre-cast segment must be removed and replaced by reinforced concrete. The two-part precast segment allows removing the upper part without demolishing the primary tunnel lining and endangering the tunnel stability.

The preliminary calculations by spreadsheet program are suitable for parameter sensitivity analyses in pre-design. The calculations are performed for each geological unit with necessary division; dependent on overburden, groundwater and other important parameters. The comparison of results shows good agreement between this spreadsheet program and finite element calculations (Marence, 1996).

2.4.3 *Numerical model*

The spreadsheet calculations are based on the analytical solution with idealized assumptions that are not fully satisfying in practice. The most influenced parameters rock mass anisotropy and primary state of stress can be easily included in the finite element calculations. The calculations follow on the numerical modeling of the excavation and primary support (Swoboda, et al. 1993) including the stresses around the tunnel after excavation: The model includes all important influences such as self weight, shrinkage and creep of lining, contact and consolidation grouting, cooling of the lining by filling and internal water pressure.

The numerical model cannot portray all nature phenomenas, but should include the most important parameters of influence. Detailed description of the finite element modeling the pressure tunnels is described in Marence (1996). The most important model part is modeling of the gap between the linings and its behavior before and after grouting. The calculations are performed by the finite element program FINAL (Swoboda, 2004) simulating possible loading cases and modelling phenomena that occurs.

In case of unreinforced final lining, the lining must stay in compression and the stresses in lining are checked. For reinforced lining the reinforcement design is performed on the results of the finite element calculations. The reinforcement amount must satisfy following criteria:

- higher than minimal reinforcement in compression and tension
- designed for actual loadings with full safety
- crack width less than 0.30 mm

The crack with width of 0.30 mm means not absolutely tight lining, but technically tight system. In case of crack width less than 0.30 mm no erosion by wash out of fine particles in the rock joints is expected and self-cure process by filling with sediments will occur (Schleiss, 1997). In case of water sensitive regions – tunnel situated near the surface or in a vicinity of potentially unstable slope the crack width criterion is tightened up.

2.4.4 *Grouting philosophy*

The grouting is performed in two steps: contact grouting and consolidation grouting.

The contact grouting achieves the contact between the final lining and surrounding rock in the tunnel crown. A gap in the crown is a result of self weight deformations and shrinkage of the concrete. Also, in the crown during concreting void (highest point) can occur. Because of all this effects for all tunnels, independent of function (traffic or conducts), contact grouting is performed. The contact grouting is performed in the tunnel crown with low pressure 2–3 bar in the crown.

The consolidation grouting has a function to achieve continuous contact between the final lining and surrounding rock mass after shrinkage of lining by drying of concrete (draught in tunnel) and cooling of the lining by water (occurs by first filling of the system). The consolidation grouting by radial hole method is selected.

By the radial hole method the grouting is performed through mostly radially arranged boreholes. During the grouting procedure the grout penetrates into the surrounding rock and the gap between the final lining and the rock mass. A separation layer – lime paint or thin plastic foil – installed in the gap will easily activate the gap. The grouting injected in the gap between lining and the rock mass results in the so-called pre-stressing effect on the final lining. To achieve the nearly constant pressure in the gap and nearly continuous loading on the lining the distance between grouting holes, in radial and longitudinal direction, should not be larger than 2–3 metres measured on the final lining extrados. In the rock mass around the tunnel open joints or plastified zones occur and must be filled with the consolidation grouting to achieve a continuous support for the final lining.

In addition, the consolidation grouting decreases the rock mass permeability and increases the rock mass stiffness and strength. Reduction of the permeability up to 20 times can be achieved. The strength and stiffness of the rock mass will not be changed with such effort, but in case of anisotropic rock the minimal values of strength and stiffness will be increased, which will cause a more uniform system around the tunnel. The reduction of the permeability is important to decrease the water loses by both solutions where technically tight lining is necessary. Technically tight means that the lining is not impermeable, but the water losses can be neglected. Additionally, the leaked water in tight surrounding will increase the water pressure outside resulting in less loading on the lining.

The length of the gap grouting holes is defined based on the following criteria:

- Achieving the continuous support around the tunnel and continuous pre-stressing pressure on lining, the length should be in order of the tunnel radius
- In case of weak rock mass the plastified zone should be fully grouted
- In highly karstified regions with two and three dimensional karst phenomena (open joints and caves) the pattern of longer boreholes (up to tunnel diameter) for exploration is foreseen
- The open joints, especially semi-parallel to the tunnel axis, and caves that are encountered during the excavation must be filled with concrete before installation of the final lining.

Generally, the system of relatively short boreholes set with tight pattern is more efficient as long boreholes set in wide pattern.

3 CONCLUSION

The design of pressure tunnel with the geotechnical design, developed as a design of excavation and primary support of underground works, is extended on the design of the final tunnel lining. The geotechnical design represents a flexible procedure that can be adopted to any project stage. During the excavation the geotechnical design of the excavation and primary support is adopted to the encountered conditions, not only to achieve a stable, safe and economical solution, but also to collect additional data necessary for the final lining design. In the pressure tunnels the final concrete lining and surrounding rock mass make an integrated bearing system. The better the rock mass behavior is known, more the final lining and the consolidation grouting can be optimized.

In the project, generally, two types of final lining are selected; unreinforced and reinforced concrete lining. The unreinforced concrete lining is used in rock mass that can be grouted by sufficient pressure. The main criterion is no tensile stress during the operation in the lining. In case of reinforced lining the consolidation grouting is used to achieve continuous contact with the surrounding rock mass and reducing the reinforcement amount.

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Underground Space Use: Analysis of the Past and Lessons for the Future – Erdem & Solak (eds)

Three-dimensional rock mass documentation in conventional tunnelling using JointMetriX3D

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ABSTRACT: JointMetriX3D is a novel system that generates reproducible records of tunnel faces based on metric 3D images. The photogrammetric approach allows measurements from the 3D images, such as positions, lengths, or areas, as well as orientations either by discontinuity surfaces or traces. From those data discontinuity spacing and frequency as well as stereograms are generated. The 3D images itself represent a reproducible record of the tunnel face preserving the information on the actual rock mass conditions allowing analyses also at a later time. Measurements can be taken at any required number and extent, even in regions that are not accessible. The improved data serve as proof of the actual conditions but also as substantial information for the decision making on site.

1 INTRODUCTION

Conventional tunnelling requires continuous adaptation of the excavation and support method to the actual ground conditions in order to get an economical and safe construction (Schubert et al., 2003). This observational approach requires among others the continuous collection of information on rock mass type, structure, and quality, as well as the system behaviour. Very important in this context are geometric properties of the rock mass and especially the discontinuity network as observed at every tunnel face and subsequently descriptive parameters which altogether supports to establish a plausible rock mass model.

Traditional methods of geological data acquisition are prone to errors (Fasching, 2001). First of all, there are sampling difficulties, human bias, and instrument errors. Secondly time restrictions lead to incomplete data and finally getting access to relevant locations is often difficult or hazardous. The resulting mapping usually do not allow to objectively reproduce the actual rock mass conditions. Data not recorded immediately are lost as excavation proceeds or support is applied.

The presented JointMetriX3D system overcomes those problems and opens new possibilities for optimisations on the tunnel site. JointMetriX3D bases on the generation of a high resolution metric 3D image of every tunnel face which is then analysed and assessed on a computer. This way the visible rock mass structures are completely recorded and geometric data on the rock surface and the discontinuity network are easily measured at an arbitrary number. There are no access restrictions and results are instantly available for further processing. The 3D image is an objective documentation of the rock mass conditions which makes analyses possible even if a specific rock face is does no longer exist.

2 WORKFLOW

Figure 1 outlines the required steps when applying the system.

Firstly, images using the high resolution panoramic image scanner are taken from two different angles (stereoscopic images). In order to enable measurements later, the imaging system should be calibrated, i.e. known deviations of the optical system from the ideal behaviour are determined and thus correctable.

Secondly the images are processed in order to get a 3D image using photogrammetric principles complemented by computer vision strategies. This step is done by a purpose-built software that can deal with panoramic images.

Then the 3D image is ready for taking measurements using another purpose-built software component that allows the quantification and management of geometric entities such as positions, lengths, areas, or spatial orientations given by dip and dip direction.

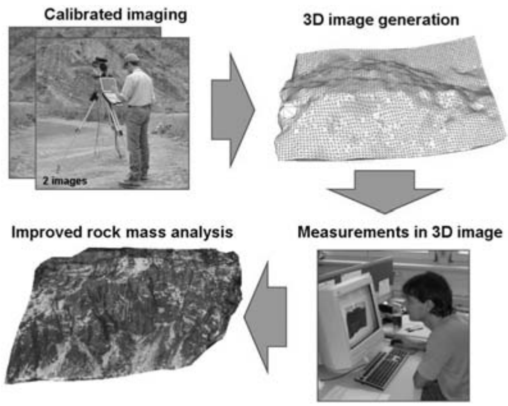

Figure 1. Data flow using the JointMetriX3D system.

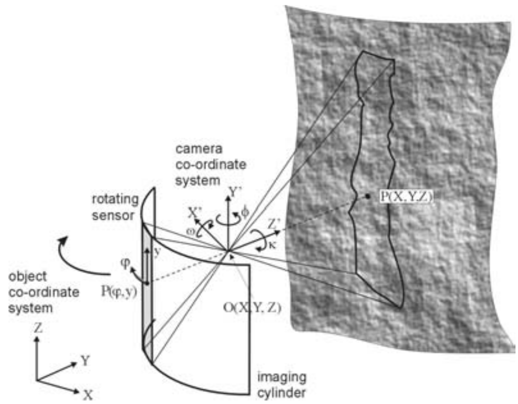

Figure 2. Geometry of panoramic line-scanning.

The results are structural maps, stereonets, descriptive and statistical rock mass parameters normally not determinable with reasonable efforts using traditional mapping.

3 IMAGING AT THE TUNNEL SITE

When taking images at a tunnel site one has to deal with several circumstances. Besides good visibility and *high resolution*, a sort of *referencing* mechanism is required in order to get a relationship between the data acquisition system and the surrounding which subsequently relates the derived measurements to the tunnel. A practical way to perform this relationship is to use reference points which means points of known positions in a superior co-ordinate system.

High resolution denotes that fine details are visible within the images which is crucial for documentation and analysis purposes. When doing conventional data acquisition the human observer also relies on the power of his/her visual capabilities. So it is obvious that any recording system must try to get as detailed information as possible when stating to be a documentation system. This is by the way a counting argument to use images instead of other sensors, such as lasers for example. If there are just intensities representing a rock face lacking of colour, lithological units and their borders as well as discontinuity traces might be misinterpreted or even not identifiable.

3.1 *The panoramic scanner*

A panoramic image is one that shows a full 360° field of view at least in one direction of the image. One approach to get a panoramic image is to move a camera along an rotation axis. When posing the rotation axis nearly vertically one gets an image that geometrically resides on a vertical cylinder, if an ideal rotation is assumed.

Figure 3. Imaging at the tunnel site. Note that only one person is required to operate the scanner.

The actually used imaging system is a rotating line-scanner. It has three CCD arrays one for each of the colour channels red, green, and blue. Each of the arrays has more than 5300 sensor elements which is a determinant magnitude for the resulting resolution that is composed by the vertical field of view in meters divided by the number of sensor elements.

The scanner is mounted on a tripod and controlled by a field notebook computer that instantly stores the acquired image data. During imaging the scanner rotates along a defined axis taking the panoramic image column by column. This principle decouples the vertical field of view (then depending only on the used lens) from the horizontal one (depending on the rotational motion). Figure 2 shows the imaging principle when using a panoramic line-scanner.

Since it is an imaging system there is of course a need for illumination. Usually one big flood light posed vaguely at the tunnel axis is sufficient to light the tunnel face up (see Figure 3).

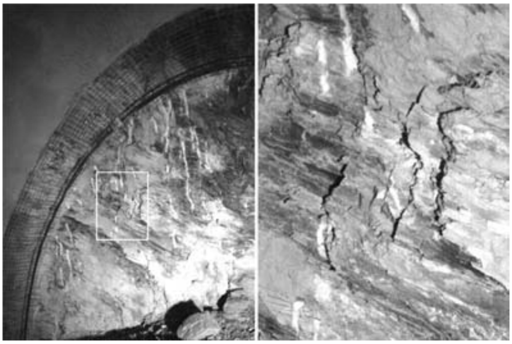

Figure 4. Parts of a tunnel panorama; the left side shows the tunnel face of a top heading excavation, the right side a detail taken from the same image. The geometric resolution is about 2 mm/pixel.

The panoramas taken at tunnel sites show typically a resolution in the mm-range. The resulting panoramas have a size of about 100 million picture elements (megapixels) which is clearly beyond conventional digital cameras. A section of a panorama is given in Figure 4. It shows a part of a tunnel face and a detail zoomed out from it. The image was taken at a two-track railway tunnel site in Austria (top heading).

3.2 *Referencing*

In order to get a relationship between the images and the tunnel, points with known co-ordinates in the tunnel system are required to be visible within the images. These points, denoted as reference points, are often available at conventional tunnel sites anyway where they are used to monitor displacements (Schubert & Steindorfer, 1996). For that reason the points are gauged regularly, often on a daily base.

An arising question is how to use those points optimally? If a conventional camera is assumed and one takes an image farther away from the tunnel face in order to capture also the reference points, the tunnel face covers only a small area of the whole image, thus giving required resolution away. Getting a closer view of the tunnel face entails that reference points disappear from the field of view of the camera.

This problem can be solved if panoramic images are used which allow the combination of high resolution at the tunnel face and a large field of view, thus making also the reference points visible. Figure 5 shows the geometric arrangement for imaging at a tunnel face. Reference points, in photogrammetry also denoted as ground control points, are indicated as well as possible camera standpoints. Note that there is no need for setting up the imaging system specially. The actual scanner poses are determined right from the observation of the reference points. In order to get proper echoes from the reflective reference points visible in the back, small light sources are mounted directly on the panoramic scanner.

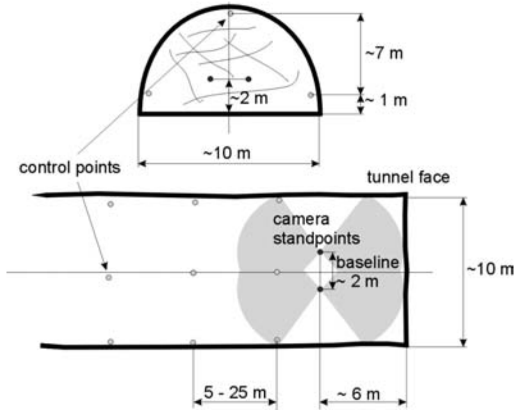

Figure 5. Schematic arrangement for imaging a tunnel face. The grey wedges indicate the useful parts from the 360° panoramas.

Once the relationship between the reference points and the acquired images is established, the results are referenced as well which means that derived measurements are related to the co-ordinate system of the reference points.

3.3 *Geometric arrangement*

As mentioned in section 2 there is a need for taking two images for getting measurements later. The two images can be taken simultaneously or subsequently. Each of the two panoramas contain the tunnel face and a view of the already excavated area as indicated with the grey wedges in Figure 5.

The time need in front of the tunnel face is some minutes typically. Within this time slot that is used usually before shotcreting all geometric data of rock surface and the discontinuity networks, as well as the referencing information is recorded.

4 3D IMAGE GENERATION

The generation of a 3D image uses principles of classical photogrammetry (Slama, 1980) complemented by more recent insights from computer vision where among others the calibration of off-the-shelf cameras was addressed (Faugeras, 1993). Having two images of the same region taken from different angles the so-called Shape from Stereo principle is applicable (Figure 6). Several tasks are required before a 3D image is ready (cf. Gaich, 2001):

– Calibration of the imaging system
– Determination of reference points

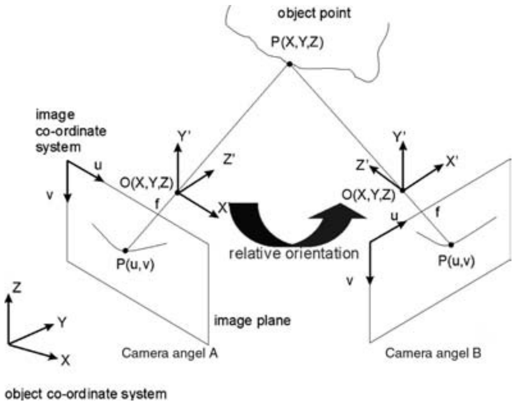

Figure 6. Shape from stereo principle.

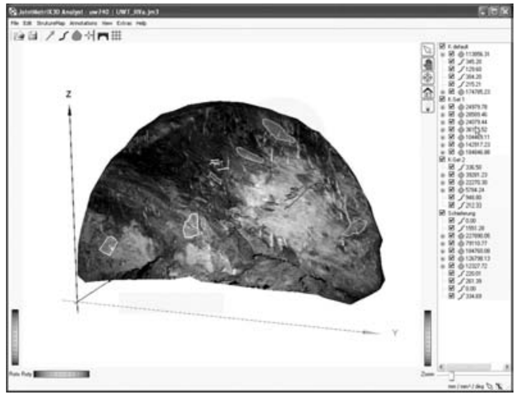

Figure 7. Snapshot showing the 3D image of a tunnel face and some measured structure data.

– Identification of corresponding image points
– Computation of 3D point cloud
– Connection of the 3D points to a surface mesh
– Geometric alignment of image and mesh

5 ASSESSMENT OF 3D IMAGES

5.1 *3D viewing*

Once a 3D image is ready, measurements can be taken from it. A purpose-built software is ready that allows to inspect a 3D image from any designated side. A turn and zoom mechanism brings any portion of the 3D image into any wanted pose. As additionally the 3D image is highly resolving, the human assessor gets this way a real impression on the rock mass conditions. Figure 7 shows a snapshot of the software with the 3D image of a tunnel face.

5.2 *Direct measurements*

From the 3D image geometric measurements are taken. At any of the possible poses of the 3D image, graphical markers can be placed "onto" it using the computer mouse. These marks denote points or regions of interest, e.g. visible discontinuity traces or discontinuity surfaces. The graphical marks themselves consists of 3D sample points and each of the sample points is given in the used object co-ordinate system, thus all measurements taken from the 3D image are inherently three-dimensional and digital.

5.2.1 *3D measuring point*

Any visible location can be marked by a measuring point getting it in the 3D Cartesian co-ordinates of the superior co-ordinate system, thus the position of any wanted location can be measured.

5.2.2 *Linear elements*

Discontinuity traces as well as lithological borders or geological strata are marked by linear elements providing instantly the true lengths in meters. Linear elements are represented as polygonal lines that follow the surface in 3D.

5.2.3 *Area elements*

Any region within a 3D image can be annotated by an arbitrarily defined closed polygonal. The sample points of the polygonal are determined directly from the 3D surface which brings the marked region also into 3D. The area of the marked patch is instantly provided in square meters.

5.2.4 *Orientations by discontinuity surfaces*

If a discontinuity surface is identified during the 3D analysis, it can be marked by an area element. The area element usually covers a number of smaller surface elements from which the 3D image is composed of. The software takes all affected surface elements and calculates a mean normal vector which represents a robust way to get an orientation measurement. Output is provided instantly by dip and dip direction and a graphical marker.

5.2.5 *Orientations by traces*

There is a second way to determine a discontinuity orientation mostly not possible during conventional field work. If a discontinuity trace is present that has a significant change in depth, i.e. the trace is not observed to be on a planar surface, the software calculates dip and dip direction from the trace alone by fitting a plane to the 3D polygonal. The plane is visualised as a spatial triangle that intersects the 3D surface as indicated in Figure 8.

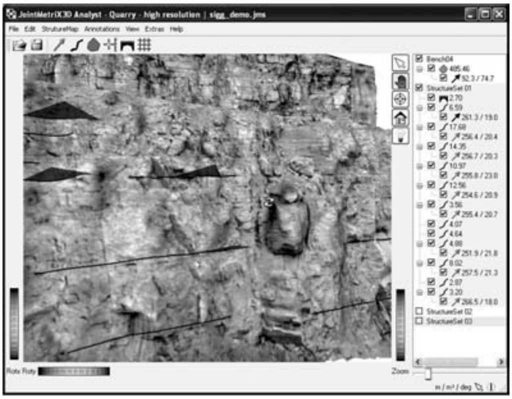

Figure 8. Discontinuity traces and computed orientation measurements indicated by spatial triangles.

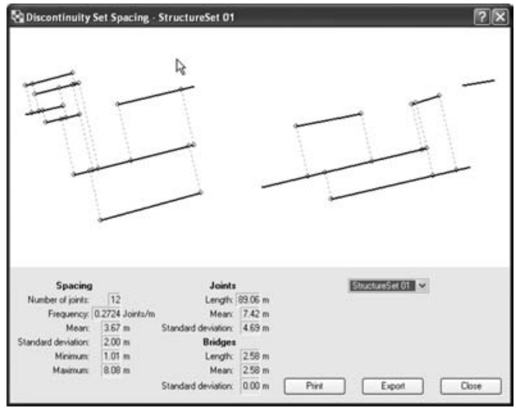

Figure 9. Computer generated sketch of the discontinuity network marked in Figure 8. The dashed lines indicate the scanline direction for determining the spacing.

5.3 *Derived measurements*

As the system allows 3D measurements that lead to geometric magnitudes in a given co-ordinate system, all descriptive rock mass parameters that base on geometric information of a rock face and the discontinuity network can be derived from those measurements.

5.3.1 *Spacing*

Spacing in this context is referred to as normal spacing, according to definitions given in the textbook by Priest (1993). For each discontinuity set, a plane of projection perpendicular to the mean orientation value is determined automatically. The plane of projection ensures that not the apparent spacing that is dependent from the actual shape of the surface is determined as it would be if just a single two-dimensional image is used.

All discontinuity traces of a set are then intersected with the plane of projection. Now the distance between adjacent discontinuities is measured along scanlines. The direction of the scanlines is also determined automatically which happens to be perpendicular to the mean orientation. Using a plane-sweep algorithm it is determined which discontinuities are adjacent.

Figure 9 shows the automatically generated sketch from the measurements shown in Figure 8. Continuous lines indicate the intersections of discontinuities and the plane of projection and dashed lines represent scanlines. This plot serves for visual inspection to verify a plausible spacing determination.

5.3.2 *Stereograms*

The measurements taken from the 3D image are grouped to sets by the user. Each set can be instantly visualized in polar nets in order to get an impression of the spatial distribution fastly where it is possible to choose between equal area or equal angle projections.

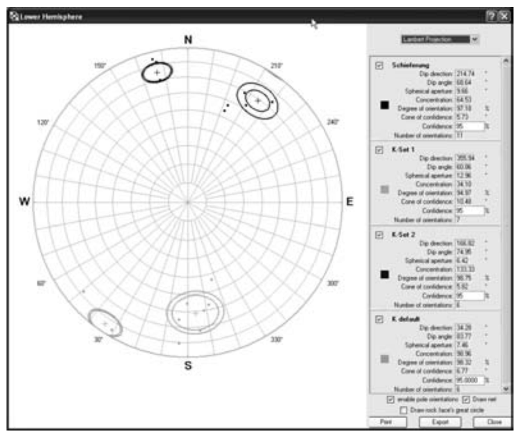

Figure 10. Lower hemisphere equal-area projection polar net of the identified discontinuity sets which is instantly available together with statistics on their distribution.

The stereograms deliver also some statistical figures, such as the spherical aperture or the cone of confidence for each discontinuity set. Figure 10 and Figure 11 outline such plots. The output is instantly updated when new orientation measurements are applied.

5.3.3 *Histograms*

Structural information that is annotated on the 3D image is stored as 3D information. Therefore true lengths of discontinuity traces are available. An export functionality allows to generate data files from which histograms as shown in Figure 12 are derived using external standard software.

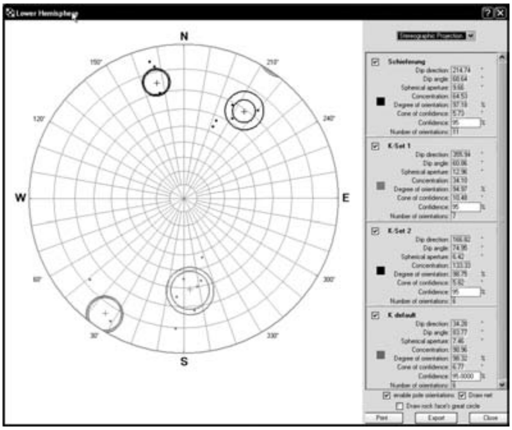

Figure 11. Stereographic projection of the same data set as shown in Figure 10.

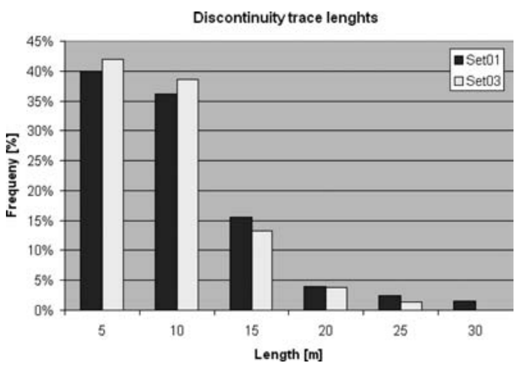

Figure 12. Discontinuity trace length distribution of two sets from a quarry assessment.

6 CONCLUDING REMARKS

The described system allows to record a tunnel face in a reproducible, objective way. Without needing knowledge on photogrammetry or computer vision one acquires relevant parameters of a rock wall quickly and preserves this way complete (visual) information of the actual rock mass conditions. Using this approach the data acquisition task is decoupled from the analysis task – the analysis can be performed even when the actual tunnel face does no longer exist.

Getting measurements from 3D images implies that this is an indirect measurement principle, therefore it is obvious that not all rock mass parameters can be determined by the system, for example discontinuity filling or strength parameters. However, indirect measurements increase safety as measurements can also be taken at inaccessible and possibly hazardous locations.

A 3D image resulting from the system represents an objective record of a tunnel face due to its high resolution, true colour, visual information and its three-dimensionality. It is no problem to do assessments at any later point in time or even support rock mass analyses from a distant location, as the data give a realistic impression to the observer.

Taking 3D images of a tunnel face in a regular manner, leads to an improved rock mass model that supports the understanding of the observed behaviour and serves as input for decisions on excavation and support on site. This specially bears the potential for saving construction cost since decisions on site are better if better information on the ground conditions is present.

An unbiased and reproduciberecord of the rock mass conditions can also a valuable asset in calim support and defence.

The JointMetriX3D system can also be applied on larger scale rock faces such as pit walls or quarries. Images of rock walls up to 300 m high can be taken from a distance of several hundreds of meters which allows an assessment of even hardly accessible slopes.

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Highway D5 Pilsen highway by-pass – the Valik Tunnel

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ABSTRACT: The Valik Tunnel is the last construction to finish the D5 highway, which connects Prague as the capital of the Czech Republic with the German border. Construction of this tunnel will enable to finish the Pilsen highway by-pass. Work preparation of the construction is characterized by a social and ecological antipathy. Design preparation and the choice of a technical solution has a longtime process. The final tunnel design, its statical assessment, the technical solution proposal and complex geological conditions have predicted the execution of a unique tunnel construction in the Czech Republic. The tunnel is designed as a twin-tube connected by a central cast-in-situ pillar in the primary lining. The tunnel is 380 metres long with the 330 metres mined section, width of the whole structure is 33 metres. The tunnel will be bored by NATM with double shell. Geological conditions are very complex, rock class 5a is the result of the final classification proposal. It has resulted from the investigation regarding the tunnel cross section and a low overburden. The design of the tunnel technological equipment for the operation fully corresponds with all safety requirements and meets the demands of European standards.

1 INTRODUCTION

Car transport in the towns is a phenomen whose negative impact on population and environment is increasing together with increasing number of cars. In the last few years very unfavourable situation has developed in most Czech towns, especially in Prague, Brno, Ostrava and Pilsen (the centre of the Western Czech District). This situation needs radical and prospective solution.

Current route net of Pilsen is no longer able to satisfy demands of its users. Therefore it is needed to quickly finish the construction of capacity road outside city aglomeration. In Pilsen, by-pass, which is the part of the higway D5 (Prague-Rozvadov-Germany), is being built. Originally, open cut was designed in the alignment of the tunnel. However, on the basis of people's and ecologists' claims it was replaced by driven tunnel.

The project of the whole by-pass road itself was being prepared for more than 9 years and was being accompanied by legal disputes. The legal disputes finally ended by decision of the Supreme Court of the Czech Republic in February 2000.

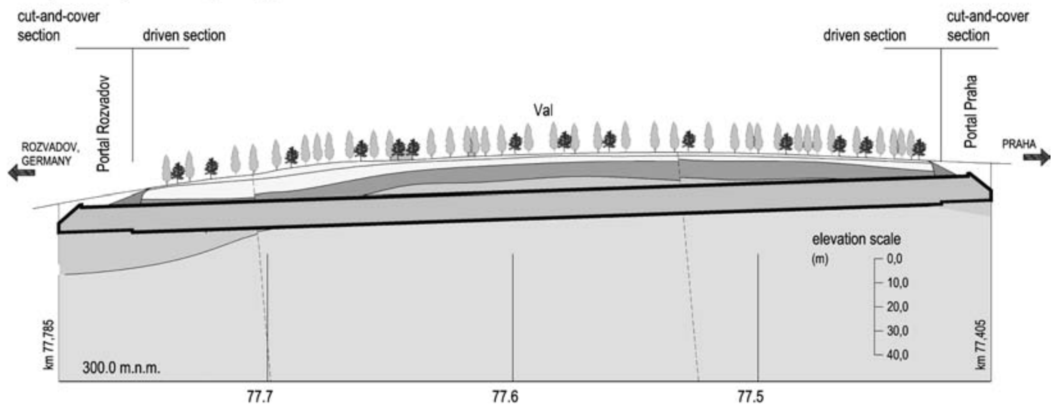
The motorway by-pass around of the town Pilsen is in the centre of attention of professional and non-professional public in the course of the last years of the old and new century. The technical experts, environmental specialists and environmentalists have got into argument during the preparation of the project. The future tunnel with a double tunnel profile will be 380 m long, 8,2 m high (in the vault head) and width of 11,5 m between kerbs. Tunnel tubes are proposed to be close without central rock pillar.

The maximal longitudinal gradient is 4%. Tunnel tubes are situated closely to each other without a central rock pillar which is replaced by a reinforced concrete pillar. This solution brings crucial saving in total range of permanent land occupation, especially in the area of pre-portal open cuts.

By this the demand for minimal intervention to the nature from ecologists and residents was fulfilled. Technical solution of tunnel construction is further more difficult than of common motorway tunnel. Therefore it needs solid knowledge of rock environment. The client "Czech Highway Directorate" decided to realize detailed exploration by a geological exploratory gallery situated in the axis of the motorway in the middle of the future-tunnel profile.

The future tunnel Valík is the part of completion of motorway D5, construction 0510/IB Černice – Útušice. This contruction comprise motorway route between km 76,510 and 79,980, from which 380 m is in the

Longitudinal profil with geology, the southern tube



driven tunnel under the hill Valik. The tunnel is the dominant and crucial part of this construction.

2 GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

The hill Valik that is situated approx. 2 km north-east from village Štěnovice and 2 km south-west from Černice is from geological point of view created by slightly metamorphed proterozoic slate (moslty filets), strongly and intensively rough, weathered with clay-filled cracks of mostly up to 2 mm width. Encountered crack system in the explorary gallery signalises a high danger of overbreaks and falls of rock in the roof and sides. We can say that structural rock features (crack-to-space-volume ratio, size of rock pieces, solidity characteristics of cracks) determine excavation and stable ratios mostly in the fourth and fifth technological class of the New Austrian Tunneling Method (rock class NATM).

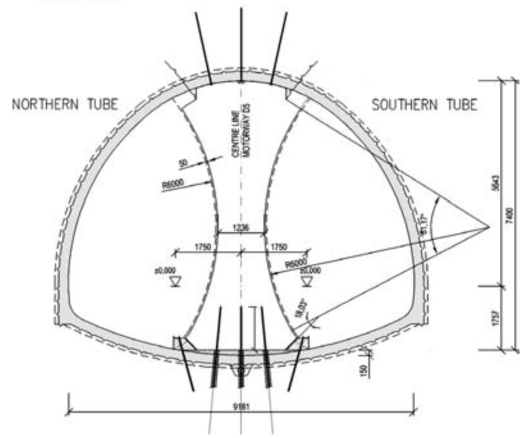
Hydrogeological conditions are rather convenient. We can assume that the boring of motorway tunnels will run in dry rock environment. Thin groundwater seepages in invert can be assumed only in the middle part of the tunnel in the chainage approx. 150–190 m.

Already during preparatory exploratory works and the processing of documentation for construction assignment, the board of geotechnical monitoring was working. The board coordinated exploratory and design works and nowadays it continues in its work during the whole tunnel boring.

3 TECHNOLOGY OF CONSTRUCTION

The construction technology of Valík tunnel is based on the principle of New Austrian Tunneling Method (NATM). Because of the aggravated geology and on the basis of exploratory gallery results a new vertical excavation sequencing is designed. The primary lining is made from sprayed concrete that is strengthened by lattice girders and by the system of anchors and forepoling.

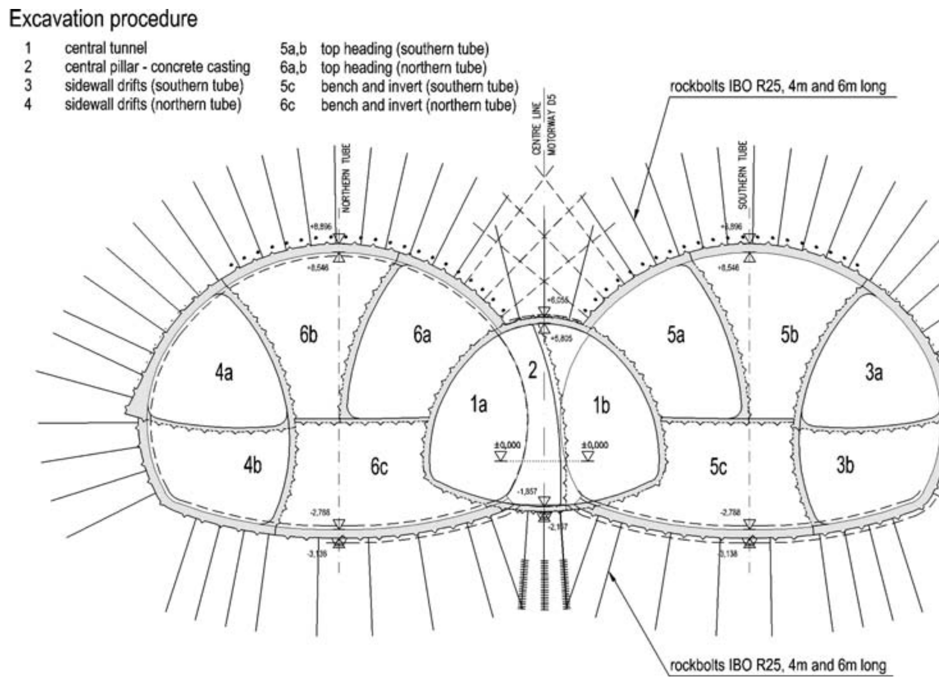
Central pillar
Cross section



Under and above the central pillar the part of rock environment is strenghtened by anchors, micropiles and control cement grouting. The inner tunnel lining is reinforced concrete, cast in-situ, with intermediate waterproofing umbrella isolation. Moreover, “vivid” model of geotechnical monitoring is also a part of construction management, that is run by a company independent of the contactor. Constistenly an observation method is used.

4 CONSTRUCTION MANAGEMENT

The construction is pursued on the principle of an observation method, in which original (basic) proposal



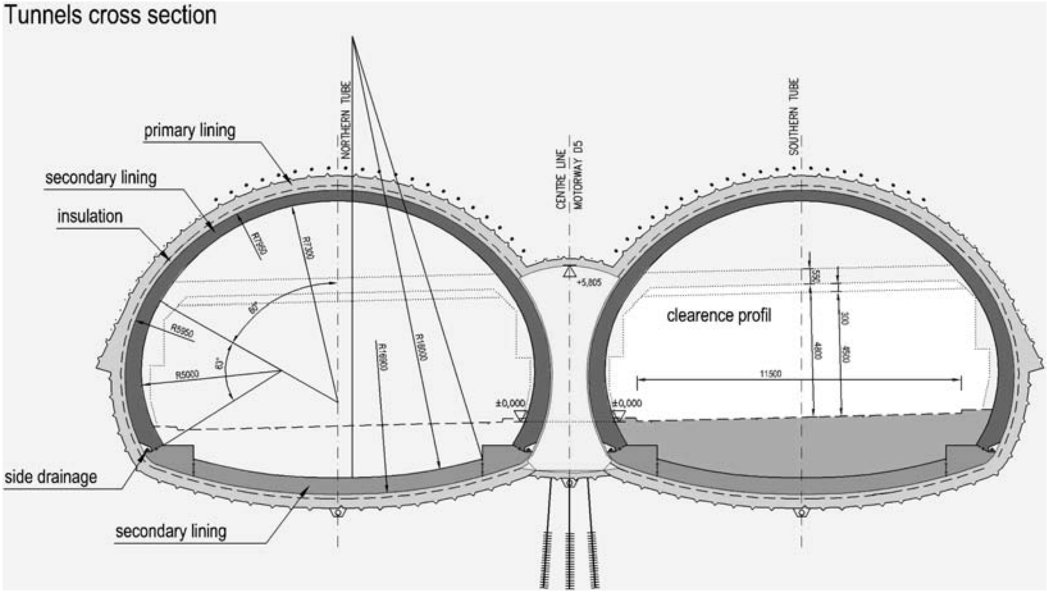
of the construction is continuously judged and can be changed even during the construction. The basic construction solutions are designed, including the range of reconstruction. Moreover, the limits of behaviour, e.g. deformation settlements of overburden, convergence etc., are determinated. The upfront solution (project and material as well) has been prepared earlier to replace the original solution in case of necessity. The detailed design of geotechnical measuring, including limits and trends, has been elaborated. In the construction, the responsible relationship of all participants is elaborated and approved. These participants have got competences and also communication means. They are able to react quickly as a team in case of exceeding of fixed limits of behaviour, which signalize an insufficiency of basic solution and call for its replenishment or replacement by upfront solutions. In the construction, the "Board of geotechnical monitoring" works under leadership of construction designer. The Board has clearly fixed competences and rights.

5 THE STATIC SOLUTION

The geological conditions are much worse than the building permit's design expected. Therefore, two independent static calculations were elaborated for the documentation of the construction assignment. These calculations were judged by an expert of Czech Highway Directorate. On the basis of expert opinion of Czech Highway Directorate, the third control static calculation was elaborated.

Static and stabilizing solution revealed all general features, especially from a view of process of the construction and support of individual rock classes. Moreover, it confirmed the reliability of designed size of central, highly loaded pillar. The area of failure is concentrated in the area between the tunnels above the central pillar, in the area in bedrock of the pillar in the right tunnel and the contact place of the pillar and invert, in the area of the left connection of invert and the side of the left tunnel. These areas will be needed to strengthen by grouting and anchors. During the construction it will be necessary to consistently use the system of safety umbrellas.

The first static and stability solution was carried out with the help of programme system PHASES 2.2 by the finite element method with the use of propability approach with the settlement of preliminary and output quantities of the solution. The second static calculation was realized in the programme PLAXIS for different combinations of preliminary parametres in the way that it is possible to optimize the proposal of preliminary tunnel lining with the regard of high variability of rock environment. The static calculations were realized in two professional workplaces in Prague and Ostrava.



6 TECHNOLOGICAL FACILITY

The construction of the tunnel is equipped by extensive technological facilities in the terms of Czech technical norms for technology of tunnel facilities together with incorporation of other requirements, especially from Police of the Czech republic and Fire Brigade. The requirements concern especially air piping, tunnel lighting, radio connection, managing system of transport and technology, electric fire signalization, electronic safety signalization, fire tunnel duct with water source, etc.

7 PREPARATORY WORKS

For security of sensitive intervention to surrounding environment, passportization of wood and buildings in the zone of danger (especially cottages in cottage settlement V Americe) will be secured already before the opening of the tunnel construction. Furthermore, the monitoring of water level in wells will continue. In case of its serious fall the contractor will guarantee the well deepening or spare bores – if this method is effective. In case of total water loss the other compensation method of water source will be proposed – in accord with the water law.

For objective public informedness there will be the information system for public contact in the facilities of the contractor. A conference room together with pemanent exhibion for public is the part of information center. The exhibition includes promotional informative material for interested persons. For announced public (excursion, groups) the information center provides professional commentary with movie projection of tunnel boring and other technically interesting construction details.

8 THE TUNNEL SECURITY

Tunnel Valík fulfill "wefewfe". This directive newly regulates preventive measures and measures providing minimal security rate in case of accidents in the tunnel. Each tunnel, whose design has not been approved yet by an administrative body during 18 months after legal efficiancy of the directive, has to fulfill this directive.

The tunnel is only 380 m long, however, it has planned high intensity of transport, total 13250 vehicles a day in one direction. This number includes 3200 heavy goods vehicles (data for year 2020). Tunnel has two tubes, oneway and sorted in the class I. The construction measures, technological facility including independent fire driveways and emergency exits fulfill all safety criteria.

In the sense of this directive the administrative body will have to appoint a tunnel manager (public or private body) who will be responsible for tunnel operation. This body will elaborate explanatory reports for each important event or accident that will happen in the tunnel. For each tunnel a tunnel manager will appoint a security worker who will look after all preventive and security measures for ensuring security of users and operational personnel.

A security worker can be a member of tunnel personnel of emergency units (firemen, police), however he has to be independent in all questions regarding tunnel security. Moreover, a security workers must not be subjected to any orders from the employer in the security questions. A security worker can be responsible even for several tunnels in the region.

A security worker will pursue the following functions:

– he plans the organization of an intervention of emergency units, including operational schemas,
– he plans, realizes and evaluates safety operations,
– he takes part in the planning of safety schemas,
– he trains operational nad safety personnel, organizes regular practise,
– he looks after the maintanence, repairs of tunnel equipment and facility.

For tunnel Valík the detailed work order will be elaborated as a part of documentation of construction realization. The working system will intimately provide solutions for security operation of tunnel Valík.

9 OPERATION ORDER PRINCIPLES

Operation order priciples were elaborated for the tunnel Valík as a companion documentation of assigned construction documentation.

This documentation only deals with operation of local unmanned dispatching near Rozvadov portal, which is a regional dispatching for management of tunnel in Svojkovice. The tunnel Valík will be fully run by employees of Police of the Czech republic. Technical employees of Czech Highway Directorate will maintan the facility.

Tunnel is equipped by technological facility that serves for increasing of security in tunnel operation and for decreasing transport risk. Selected important functions are collectively processed in tunnel subcentral office placed near a portal Rozvadov and moreover they are centrally archived and run by the operating service situated in the Maintenance Highway Service Centre in Svojkovice in the building of Police of the Czech Republic. There is only one operation service workplace that consists of several rooms equipped by appropriate technics, technological and social background for the operation.

The proposal of transport and operation order of the tunnel is also a part of the “Principles”. The transport order is based on different transport situations. Basic operation alternatives are following:

– regular regime,
– special regime without the police participation,

– special regime with the police participation,
– exceptional operation,
– emergency operation.

The characteristic of critical situations of transport is intimately described in the documentation. It is mostly transport and technological excesses in the tunnel. The tunnel is equipped by transport and technological facilities that enables to run transport and technology for all alternatives of transport operation in the tunnel and in its foreground. The set of “transport situations” has been settled. The set is the basis for processing of detailed proposal of tunnel signalling.

In case the tunnel is closed, the transport will be directed to local roads in close fly-over crossings; in the direction from Prague to the fly-over crossing Černice D5 – X – 1/20, from Rozvadov to the fly-over crossing Útušice – Jih D5 – X – 1/27. By-pass routes on local roads have been discussed with the Police of the Czech Republic.

The “Plan of fire-fighting” in different parts of the tunnel has been intimately processed and discussed already in the preliminary phase. On the basis “Fire design” was necessary to finish the design of individual arrival fire roads to individual portals including fire ascending areas.

10 CONCLUSION, CURRENT STATE OF REALIZATION

Metrostav Inc. is the contractor. All design works including work design are guaranteed by Pragoprojekt

Inc. Stavební geologie Praha Inc. provides geotechnical monitoring. The construction itself started by preparatory works in the fourth quarter in the year 2003. The tunnel is expected to be finished by 30. 10. 2006. Nowadays (September 2004) the central auxiliary tunnel has been bored and casting of the temporary central pillar is being made. The excavation of transport tunnels will start in the last quarter of 20.

Underground Space Use: Analysis of the Past and Lessons for the Future – Erdem & Solak (eds)

A risk based methodology for optimum tunnel configuration

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ABSTRACT: In planning new underground metros, a primary concern is to optimize the tunnel configuration in terms of operational safety, construction cost and schedule and environmental impact, within the prevailing regulatory framework. With improvements in Tunnel Boring Machine technology over the last 25 years, it is now feasible to construct large diameter bored tunnels with two parallel tracks running in the same tunnel, within crowded urban areas in poor ground and high ground water conditions. However, satisfactorily undertaking this form of construction should not be considered the only governing parameter. As the completed tunnels will be used for long-term of passenger transportation, operational safety as well as maintainability should be of equal, if not of more importance than construction capability alone. This paper outlines many of the risks that should be considered, their probability of occurrence and the magnitude of their consequences should a risk event ever materialize and the impact of local regulatory requirements.

1 INTRODUCTION

1.1 *Metro planning*

In planning new underground metros, a primary concern is to optimize the tunnel configuration in terms of operational safety, initial and life cycle cost, construction schedule, and environmental impact, all within the prevailing regulatory framework. With improvements in Tunnel Boring Machine technology over the last 25 years, it is now both technically and commercially feasible to construct large diameter bored tunnels with two parallel tracks running in the same tunnel, within crowded urban areas with poor ground and high ground water conditions. However, the ability to satisfactorily undertake this form of construction, should not be considered the only governing parameter. Since the completed tunnels will be used for long-term transportation of passengers in a safe and efficient manner, operational safety as well as maintainability should be of equal, if not of more importance than construction capability alone.

Operational safety of an underground Metro system is subject to different standards and guidelines, depending on the country in which it is built. In undertaking the evaluation of whether to adopt twin-tube single track parallel tunnels or a mono-tube double track tunnel, the design, construction and operations risks should all be considered from both a civil and systems engineering and a regulatory perspective. This regulatory framework has led to various tunnel configurations in a number of countries as shown in Table 1.

This paper outlines many of the types of operational risks that should be considered as well as a method of rating the risks for their potential impact on the project, considering their probability of occurrence and the magnitude of their consequences should a risk event ever materialize. It considers the one form of regulatory framework as well as a discussion of perceived differences between this, EU Countries and the U.S.

1.2 *Use of risk assessments: qualitative versus quantitative*

For planning purposes, it is considered adequate to perform preliminary risk assessments using a more qualitative rather than quantitative approach, since many project details are not sufficiently defined to justify quantitative methodologies requiring input of detailed project information. Once a project advances

Table 1. Tunneling configuration adopted on recent Metro systems in Europe.

Method	Location	Configuration
Twin tube	London	Jubilee Line Extension opened in 1999. Bored single-track tunnels.
	Munich	5.74 m diameter bored single track tunnel with cut-and-cover.
	Nuremberg	Cut and cover, some bored single track.
	Vienna	Partly single track bored, partly double track cut-and-cover.
	Copenhagen	Bored single track tunnels.
Mono tube	Paris	Meteor line opened in 1998. First section of line in twin track tunnel
	Toulouse	Fully automated unmanned rubber tyred metro, twin track tunnel
	Athens	Lines 2 and 3 were opened in January 2000. Twin track tunnel.
	Barcelona	Line 9 total length 42.5 km of which 34 k. are underground. Twin track with partial vertical separation.
	Lyon	"Magally" system runs in twin track tunnels.
	Madrid	Metrosur and Line 10 extension with 27 km generally mono tube tunnels.

into the preliminary engineering phase of development, more sophisticated statistical approaches are justified.

2 OVERVIEW

2.1 *Regulatory requirements in UK and elsewhere*

Specific directives for railways in Europe apply basically to regional railway systems while urban underground systems are subject to local regulations. In European countries, safety certification is either subject to self-certification by the transit agency or compliance with country laws such as the UK requiring the submission of safety cases which require risk assessment and cost benefit analyses which show that all risks are either ALARP (As Low As Reasonably Practicable) or mitigated to this level at reasonable cost. See Table 2 for an overview of these requirements.

Table 2. Regional railway legislation.

System	Configuration	Reasons
European Heavy Rail	Up to 1 km can be mono/twin. Beyond 1 km subject to specific requirements	European Codex 779-9 relates only to regional railways. See Codex Item I 20
European Urban Transit	Should Comply with Local Policy	No European Directive
UK Railways Systems	Must Satisfy UK Health & Safety Requirements	UK Law
Irish Railway Systems	Must Satisfy Irish Health & Safety Requirements	Irish Law: must comply with Railway Order requirements.
USA Urban Transit	Self Certification Subject to State Safety Oversight	See FTA "Handbook for Transit Safety" & Security Certification

3 RISK ASSESSMENT FOR TUNNEL CONFIGURATION

The bored tunnel risk assessment considered three tunnel configuration arrangements:

1. Twin tube tunnels with single track;
2. Monotube tunnel with two tracks; and
3. Monotube with two tracks and central dividing wall.

Figure 1 shows a single-track twin tube 5.6 m diameter tunnel cross-section. The tunnel centreline is the same as the track centreline. The tunnel structure would typically be constructed of a pre-cast concrete segmental lining. A passenger walkway is provided along the inner wall of each tunnel in accordance with the Irish Draft Guidelines for the Design and Construction of Railway Infrastructure and Rolling Stock. Access to a place of safety is provided by cross passages between the two tunnels, spaced according to regulatory requirements.

Figure 2 shows a typical cross section of a monotube double track 8.5 m diameter tunnel with tracks positioned side by side. The tunnel structure would typically be constructed of pre-cast concrete segmental lining without a central wall. The centreline of the tunnel lies between the two tracks with passenger walkways located along the sidewalls. Access to a place of safety is provided by vertical shafts to the surface, again spaced according to regulatory requirements.

Figure 3 shows the monotube tunnel with the provision of a vertical central dividing wall. Construction is similar to the Figure 2 scenario, but the diameter

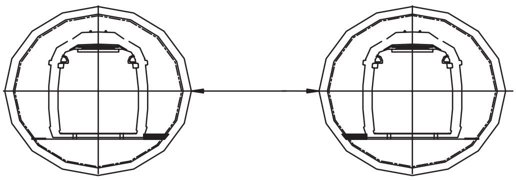

Figure 1. Twin tube tunnel configuration – nominal 5.6 m twin tube tunnels.

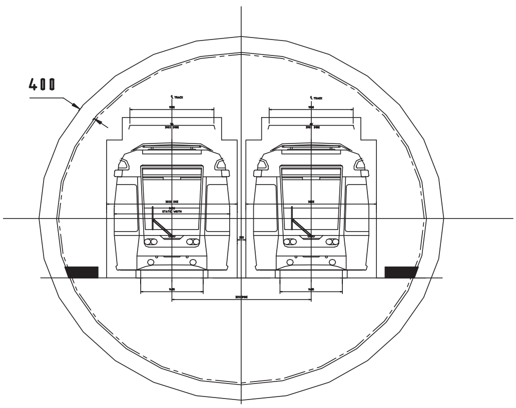

Figure 2. Mono tube tunnel configuration – nominal 8.5 m twin track tunnel – no central wall.

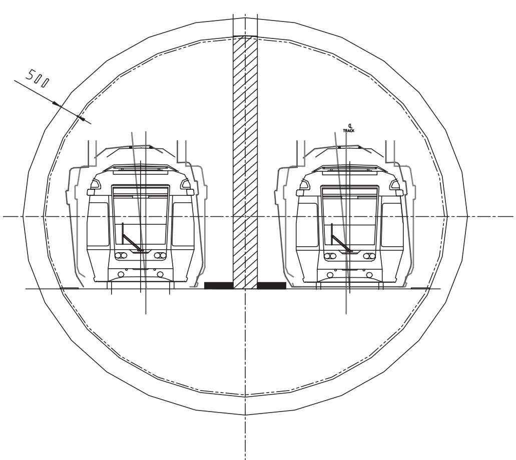

Figure 3. Mono tube tunnel configuration – nominal 10 m mono tube tunnel with central wall.

of the tunnel increases to 10 m to accommodate the central wall. Walkways are positioned in the centre of the tunnel along each side of the dividing wall leading to access points through the wall, thus providing a place of safety on each side of the wall.

4 DESCRIPTION OF RISK CONCEPTS

The acceptance of risk (i.e., a tolerable rate of dangerous failures) is based on defined principles and standards. ALARP (As Low As Reasonably Practicable) is the method used when collective risks are considered. This means that all risks affecting all persons using the system and the environment are taken into account.

The ALARP principle classifies risks according to frequency and severity. For example, for each different class of possible accidents, a maximum allowable upper limit of risk is defined which must not be exceeded. Above this limit risks are "Intolerable" and risk reducing mitigation measures must be implemented. Below this limit, and extending to a lower limit beyond which risks are "Tolerable" is the region of ALARP risk. In the ALARP range, mitigation measures should also be undertaken, but normally only if they are economically justified.

For an underground Metro system risk assessment, a multi-disciplinary team of experienced, independent professionals should be convened, each representing worldwide twin tube and monotube tunneling experience in design, construction and operation.

For purposes of a Risk Assessment Workshop, the participants should be divided into at least three teams to correspond with different phases of the project – design, construction, and system operation. As a first step, each group should identify and agree upon on the risks to be considered, ratings to be assigned and the appropriate mitigation measures required.

Following the risk assessment workshop, mitigation costs for all risks determined to be in the Intolerable and ALARP categories should be developed. In order to make informed decisions regarding implementation of mitigation measures, cost benefit assessment in the planning phase is usually limited, and restricted to (a) ALARP risks, since mitigation of all intolerable risks is mandatory; and (b) systems operations risks, because of the high priority usually given to systems performance and passenger safety.

During this process the risk assessment report evolved based on review comments from RPA and the workshop participants.

5 METHODOLOGY FOR RISK ASSESSMENT FOR TUNNEL CONFIGURATION

For the preliminary planning phase of a project, the risk assessment can be based on the tunnel section only without consideration of the risk inherent in other structures. All risks deemed pertinent to the operational safety case for the tunnel should be considered. Environmental, regulatory, contractual, legal, commercial, financial and property availability issues, which were deemed not to be relevant to the safety case, need not be considered in the planning phase.

While these limitations are important they are not expected to affect the choice of tunnel configuration based on safety, but should be considered in deciding on the overall acceptability of the three tunneling scenarios. The risk register for the assessment of bored tunnel methods should include all pertinent risks such as:

1. Systems risks – covering the rolling stock and electrical and mechanical systems
2. Civil risks – covering the physical infrastructure.

For the purposes of this presentation, civil risks are excluded since operational safety is the key issue in selecting a final tunnel configuration. These systems risks were then further subdivided into the three project phases:

1. Design
2. Construction
3. Operations.

The risk workshop participants should then rate each risk in terms of probability of occurrence and severity for each of the three tunnel scenarios: twin tube (T), monotube (M) and monotube with divider wall (M/D). The rating designations were: Intolerable (I); ALARP High A (H); ALARP Low A (L); and Tolerable. The risk model can be represented by:

$$R = F \times C$$

R = Risk Rating
F = Frequency of Occurrence
C = Magnitude of Consequences

The probability of occurrence for systems risks should be considered as:

1 = Improbable, extremely unlikely; Improbable: 1 in 100 years
2 = Remote chance of occurrence; 1 in 1000 projects; Probability: 1 in 50 years
3 = Occasional, or likely to occur; 1 in 100 projects; Probability: 1 in 10 years
4 = Probable, multiple occurrences; 1 in 10 projects; Probability: 1 in 5 years
5 = Frequent occurrences likely; Occurs every project; Probability: Once per year.

The severity of the consequences should be considered as:

1. Insignificant
   - No operational delays expected
   - No extra costs
   - No lost time injuries
2. Minor
   - Operational delays of hours expected
   - Extra costs easily accommodated
   - Potential for serious injury
3. Significant
   - Operational delay <1 day expected
   - Costs may be significant
   - Potential for a fatality
4. Major
   - Operational delay <1 week expected
   - Substantial cost impact likely
   - Single fatality likely
5. Catastrophic
   - Operational delay of weeks or months
   - Cost impact excessive
   - Multiple fatalities likely

For evaluation of mitigation measure requirements, the risk ratings were categorized as:

- Tolerable; R = 1–6
- ALARP; R = 8–12
- Intolerable; R = 15–25

6 MAJOR RISKS

As a reference case history, a study conducted for planning of the Dublin Metro will be considered. In that risk evaluation, the systems risks identified, as they relate to operational safety, included 17 design risks, 9 construction term risks and 40 operational risks. The major risks ($R > 15$) were:

Design
- Unauthorized access control
- Excessive electromagnetic interference
- Inadequate stray current protection

Construction
- None were considered critical

Operations
- Tunnel/Train Fire
- Inadequate emergency response plan
- Tunnel/station flooding
- Security – vandalism/urban violence
- Inadequate maintenance

7 RISK MITIGATION

For Dublin Metro systems, mitigation measures were considered for Intolerable risks, and the higher ALARP risks. For the risks considered for mitigation, several mitigation measures were recurrent for a solution to reducing a number of different risks.

The provision of platform screen doors occurs repeatedly as a mitigation measure for many of the key risks (e.g., vandalism, unauthorized access, passenger train interface). In the cost-benefit analysis this measure was considered to significantly reduce the risk of platform train interface hazards by a factor of 10.

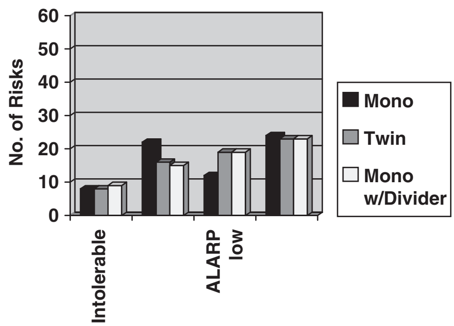

Figure 4. System risk summary – pre-mitigation.

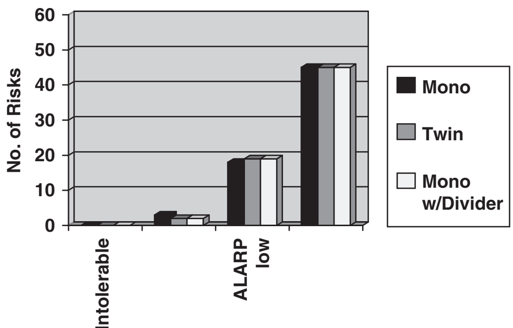

Figure 5. System risk summary – post-mitigation.

Platform screen doors could be a significant mitigation measure, but are not mandatory. The use of platform screen doors would have to be interfaced with the potential use of automatic Train Operation (ATO). It may be possible to replace ATO with an automatic train docking system at a significantly lower cost.

Tunnel fires either within the train or tunnel are one of the key risks, which can have major consequences. Mitigation measures should be designed into the system such as the provision of an escape walkway (benches), tunnel ventilation, material and equipment selection, minimizing train fire load and fire detection and suppression facilities.

Figure 4 shows the systems assessment results prior to mitigation of risk.

Figure 5 shows that after mitigation, the systems assessment results in all options being relatively risk equal and constructible with acceptable risks.

8 COST BENEFIT ANALYSES

Cost benefit evaluations were conducted only for systems related risk mitigation measures for the long-term performance stage of the project. In addition, they were limited to the ALARP risks, as mitigation measures for Intolerable risks are considered mandatory, not optional.

Metro accident statistics were collected and analysed to determine the potential safety benefits of the identified mitigation measures. The risk to the Dublin Metro tunnel section was estimated from both London Underground risk data and US transit accident statistics by scaling the operational services level (train miles).

For the major systems mitigation measures considered, the cost/benefit results are included in Table 3.

At this stage of the project, the “apparent” findings should be considered somewhat preliminary in nature for the following reasons:

1. There is an inherent uncertainty in carrying out quantitative cost/benefit analyses at this early stage of a project due to lack of detailed back-up information.
2. The analysis results presented do not reflect costs associated with damage to the infrastructure or loss of operation as a result of an incident.
3. They do not reflect the additional cost of enlarging the mono-tube tunnel sufficient to accommodate the central dividing wall.

9 CONCLUSIONS AND THEIR RELEVANCE TO OTHER REGULATORY SYSTEMS

1. All tunnel configuration options, based upon the Dublin case, are considered approximately risk equal after risk mitigation measures are applied.
2. A monotube tunnel with a centre wall could not be justified from a cost/benefit perspective due to the short length of tunnel. Because of the increased tunnel diameter, the station width and depth also must increase and the overall capital cost is likely to exceed the cost of a twin tube system.
3. Mitigation measures, both mandatory and optional (but desirable from cost/benefit perspective) in the ALARP category, should be included in the project design scope, and adjustments made to the construction cost and risk contingency cost estimates.
4. The decision on tunnel configuration must ultimately be acceptable to the local and regional regulatory agencies regarding railway safety. In the case of Dublin Metro, the over riding concern was emergency services access distance from a place of safety, tunnel ventilation efficiency in a fire scenario and reduction of collision risk in tunnel. Thus, the twin tube configuration was considered to best perform against these parameters.
5. The over-riding underground Metro operational concerns related to fire in the tunnel and train collision and these two issues predominated in the final decision on Dublin Metro tunnel configuration. Resolution of these concerns under other regulatory philosophy, could be considered, to be also offered

Table 3. Cost benefit results – systems performance ALARP mitigations.

Mitigation measures safety benefits	CPF €m	1. year	CPF €m/ year	2. per year	VPF € per year	PF
Platform screen doors						
Reduces train/platform accidents				5.3E−2		
Collisions with people				4.3E−2		
	10.0	25	0.3	9.7E−2	1.6E+5	2.5
Central Kerb – derailment containment						
Reduces derailment consequences				1.9E−2		
Reduces collision potential				3.0E−3		
	4.0	100	0.04	2.3E−2	3.8E+4	1.1
Central Divider wall						
Reduces derailment impact				2.0E−2		
Reduces derailment collision potential				3.0E−3		
Reduces tunnel fire consequence				9.2E−4		
	15	100	0.15	2.4E−2	3.9E+4	3.9
Tunnel Portal fencing						
Reduces accident potential	0.05	25	1.8E+3	3.2E−3	5.2E+3	0.4

CPF: Cost of preventing fatality.
VPF: Value of preventing a fatality.
PF: Proportion factor.
1.: Operational life.
2.: Reduction in average fatalities assessed by mitigation.

by the monotube. It is therefore considered that the basis for a decision on tunnel configuration is dependent on experience of local operating systems and consideration of emergency scenarios which sets the weighting of safety risks.

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Effect of ground water on tunnel face stability in the soft ground

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ABSTRACT: The Sambongihara tunnel, a double-track Shinkansen tunnel, is now construction to a small overburden in the soft ground by the shotcrete tunneling method or the so-called New Austrian Tunneling Method (NATM). At the construction site, the ground water level is high throughout the whole section, and the ground near the tunnel face is a sandy soil with a clay layer, whose position at the face varies as the tunnel construction work advances. The ground water flows both in the lower and upper sandy layers separated by the clay layer. Therefore, deep well method is adopted to lower the ground water level, since the ground water level normally has significant influence on the stability of the tunnel face. However, it is difficult to lower the ground water level in the upper sandy layer. This paper describes an outline of this tunnel, measurements of the groundwater level, results of face observation, results of the simulation analysis based on the plastic theory, or the so-called rigid plastic finite element analysis (RPFEA), and discusses the influence of groundwater level and conditions of ground layer on the tunnel face stability.

1 INTRODUCTION

To ensure the tunnel face stability, which is normally subjected to the influence of ground water, it is effective to lower the ground water level for tunnels constructed below the ground water level.

This paper describes a method to evaluate the tunnel face stability by rigid plastic finite element analysis (RPFEA) giving consideration to the effects obtained by the above-mentioned method. In the analysis, the effect of ground water on the tunnel face is expressed by applying a water pressure as a force on the ground.

2 CONSTRUCTING PROGRESS OF SAMBONGIHARA TUNNEL

2.1 *Outline of tunnel construction*

Tohoku-Shinkansen is planed along the 675 km-route from Tokyo to Shin-Aomori, of which the section between Tokyo and Hachinohe has already been put in service. Construction work is now in full swing to extend Tohoku-Shinkansen from Hachinohe to Shin-Aomori (82 km-route).

Twelve shallow tunnels have been planed or constructed in the soft ground between Hachinohe and Hakkoda Tunnel. Some of double- track Shinkansen tunnels are now under construction in the soft ground by the shotcrete tunneling method or the so-called New Austrian Tunneling Method (NATM). Among these tunnels, the 4,280 m-long Sambongihara Tunnel has an excavation width of 10.5 m and faces as high as 5.25 m at the upper half section. The overburden of the tunnel is about 23 m on an average, and 45 m at the maximum (Figure 1).

The geology near this tunnel is mainly composed of sandy soil, and alternation of strata is formed of

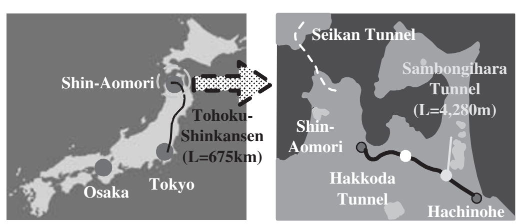


Figure 1. Location of Sambongihara tunnel.

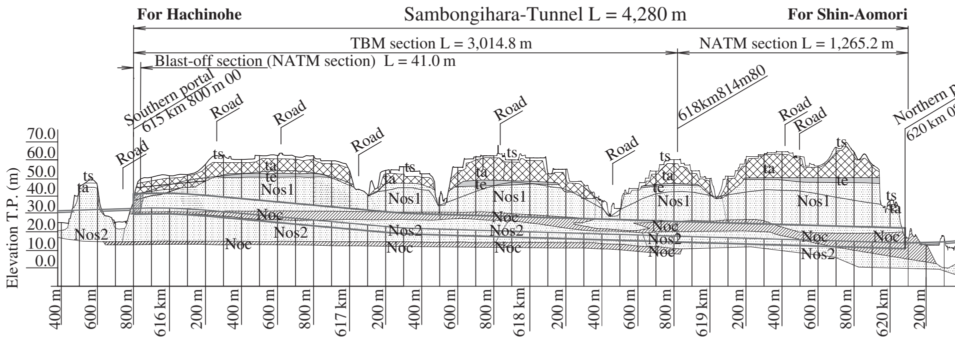
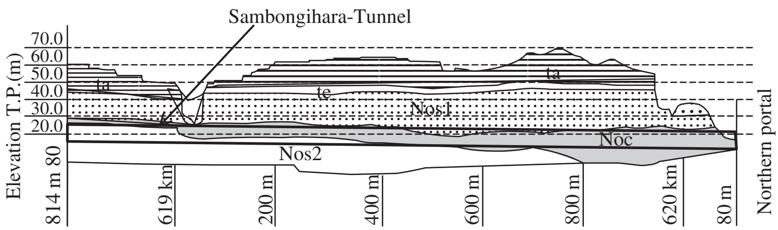


Figure 2. Geological profile of the Sambongihara-tunnel.



Mark	Geological condition	Characteristic
ts	surface soil, bank	
ta	Takadate volcanic ash layer	loam
te	Tengutai volcanic ash layer	loam
Nos1	Upper Noheji sandy layer	sandy soil
Noc	Noheji cohesive soil layer	clay soil
Nos2	Lower Noheji sandy layer	sandy soil

Figure 3. Geological profile of the Sambongihara tunnel (NATM section).

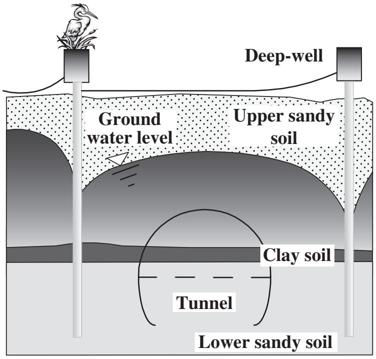


Figure 4. Situation of the actual tunnel.

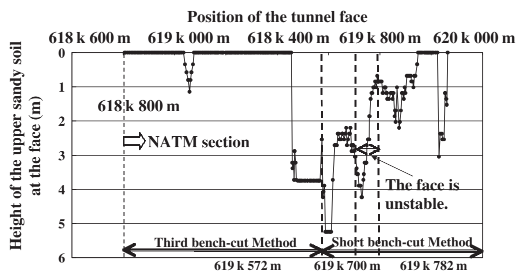


Figure 5. Relationship between the height of the upper sandy soil at face and the excavating pattern.

sandy soil and clay soil at the tunnel face (Figure 2 and Figure 3). The problems involved in the construction work are the flow of sandy soil; the ground water level of about 15 m above the tunnel crown; the fine fraction contents of 3 to 30%; and the uniformity coefficient of 1 to 9. In order to lower the ground water level near the face, therefore, this tunnel is excavated with the deep well method simultaneously applied.

2.2 *Tunnel face*

The ground near the tunnel face is sandy with a clay layer, whose position at the face varies as the tunnel

Figure 6. View of the face collapse.

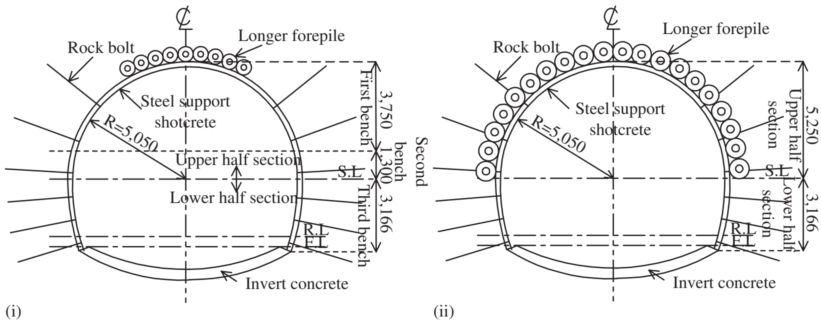


Figure 7. Excavating patterns. (i) Third bench-cut method, (ii) Short bench-cut method.

(i) (ii)

Figure 8. Excavation. (i) Longer forepile, (ii) Third bench-cut method.

Figure 9. Pumping well.

construction work advances (Figure 4). Moreover, the upper sandy soil was heterogeneous whereas the lower soil was homogeneous. Under these conditions, the ground water level in the lower sandy layer was fully lowered by the deep well method. However, it was difficult to lower the ground water level in the upper sandy layer. The face was unstable, in particular, where the upper sandy soil was high at the face (Figure 5). Small degrees of face collapse occurred twice in the tunnel, since the ground water level normally has significant influence on the stability of the tunnel face (Figure 6). Fortunately, however, nobody was injured, since the face conditions were sufficiently monitored and evaluated.

Longer forepiles were used instead of short forepiles to ensure the stability of the crown. Additionally, the tunnel was excavated by the third bench-cut method instead of the short bench cut method to lower the height of the face to maintain stability of the face (Figure 7 and Figure 8).

2.3 *Ground water lowering method*

For the shotcrete tunneling method, it is important to lower the ground water level near the tunnel face before excavation because the face is open.

In the tunnel, pumping wells bored by the deep well method are located at intervals of 20 m toward the tunnel axis in zigzags, at the distance of 10 m from each tunnel side. Additionally, observation wells are located on their fringe for monitoring fluctuation of the ground water level (Figure 9 and Figure 10).

The ground water level was lowered almost linearly before the face approach the pumping well. However, the ground water level was lowered only at the crown of tunnel (Figure 11).

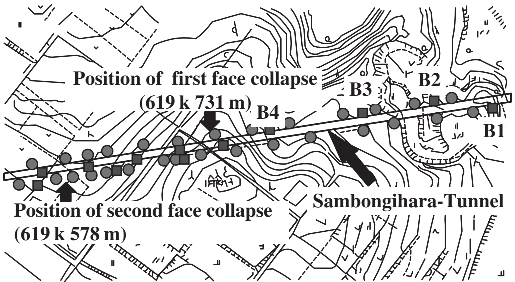
2.4 *Study of the face stability*

The characteristics of the upper sandy layer are as follows.

- The position at the face varies as the tunnel construction work advances.
- The ground is heterogeneous.
- The ground is easy to collapse due to the ground water.
- It is difficult to lower the ground water level in the layer.

The upper sandy soil exists over the entire face where the clay layer is positioned at the foot of the upper half section. Therefore, the water pressure by the ground water in the soil works on the face, to make the face significantly unstable.

Consequently, the stability of the face is studied based on the relationship between the height of the sandy soil on the face and the ground water level



● : Pumping well with deep well method
■ : Observation well
B1~B4: Point number of observation well

Figure 10. Location of the pumping wells and observation wells.

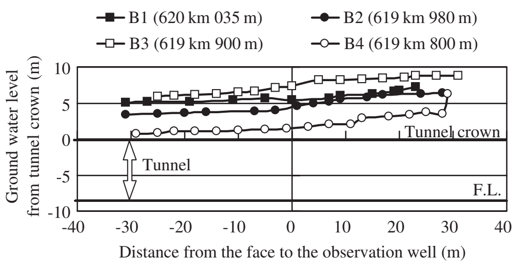


Figure 11. Relationship between ground water level and tunnel advances.

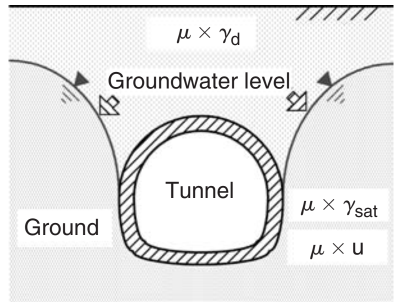
which is calculated by RPFEA giving consideration to the effect of the ground water.

3 SIMULATION ANALYSIS

3.1 *Rigid plastic finite element analysis*

Rigid plastic finite element analysis (RPFEA) pays attention only to the plastic state on the ground, and uses the upper bound theorem. Limited analysis is formulated by applying the FEA based on the plastic theory with this analysis. The method has attracted attention in the field of metal processing for the express purpose of calculating the magnitude of the force to process the metal (Hayes and Marcal, 1967; Kobayashi et al. 1973; Mori et al. 1979). In the geotechnical engineering, it can also be used as the analysis method in order to evaluate slope stability, stability of tunnel face and bearing capacity.

The method expresses only the state of the moment when plastic strain of the ground increases rapidly, or the state of the moment when the ground begins to break, and has some merits as shown below (Tamura et al. 1999; Konishi et al. 2000, 2002).



Here
γ_d : Unit weight of soil above the ground water level
$\gamma_d = \rho_d g$
γ_{sat} : Unit weight of soil under the ground water level
$\gamma_{sat} = \dfrac{\rho_s - \rho_w}{1 + e} g$
ρ_d : Dry density of soil
ρ_s : Density of saturated soil
e : Void ratio

Figure 12. Change with unit weight of soil due to the change in the ground water.

- The elastic coefficient, which is useless under the critical state, is unnecessary.
- The method can well express the elastic flow near the critical state, of which errors become large with an elasto-plastic model.
- The initial stress is not required.

RPFEA is applied with the Drucker-Prager criterion by assuming the associated flow rule. The unit weight of soil and water-pressure are treated as body forces.

In the analyses, the following three factors are taken into account (Konishi et al. 2002).

- Change in the unit weight of soil due to the change in the ground water level (Figure 12).
- Change in the effective stress due to the change in the unit weight of soil and pore water pressure.
- Strength of the ground due to the change in the effective stress.

The stability of tunnel face is evaluated by acceleration μg as a face collapse occurs. The value of μg, in which g is the acceleration of gravity and μ is the load factor, is calculated by RPFEA. The method has the same test concept of tunnel face stability as that of the centrifuge test machine (Schofield, 1980). Not only the unit weight of soil γ but also the pore water pressure u are multiplied by the load factor μ as a face collapse occurs. The load factor μ is a measure barometer of stability. We considered that a face collapse actually occurs, when the load factor becomes $\mu \leqslant 1$. Then, we investigated the relationship between the height of face and the ground water level at a face collapse by this method.

Basic equations for the analyses are as follows (Tamura et al. 1984, 1987)

Equation of equilibrium:

$$\sigma_{ij,j} + f_i = 0 \tag{1}$$

Compatibility equation:

$$\dot{\varepsilon}_{ij} = \frac{1}{2}\left(\dot{u}_{i,j} + \dot{u}_{j,i}\right) \tag{2}$$

Stress-strain rate relation:

$$\sigma'_{ij} = \frac{\sqrt{2}k}{\sqrt{6\alpha^2+1}}\frac{\dot{\varepsilon}_{ij}}{\bar{\dot{e}}} + \lambda\left(\delta_{ij} + \frac{3\sqrt{2}\alpha}{\sqrt{6\alpha^2+1}}\frac{\dot{\varepsilon}_{ij}}{\bar{\dot{e}}}\right) \tag{3}$$

Kinematical constant:

$$\dot{\varepsilon}_{kk} + \frac{3\sqrt{2}\alpha\bar{\dot{e}}}{\sqrt{6\alpha^2+1}} = 0 \tag{4}$$

Constraint on the velocity on the basis of the upper boundary theorem:

$$\int_V f_i \dot{u}_i dV + \int_{S\alpha} \bar{T}_i \dot{u}_i dS = 1 \tag{5}$$

Relation between stresses and pore water pressure

$$\sigma_m{}' = \sigma_m - \mu u \tag{6}$$

$$\sigma_{ij}{}' = s_{ij} + \left(\sigma_m - \mu u\right)\delta_{ij} \tag{7}$$

For the purpose of simplicity, we introduce the following notation:

$\sigma_{ij,j}$: Total stress tensor
f_i : Nodal force
$\dot{\varepsilon}_{ij}$: Strain rate tensor
$\dot{u}_i$: Nodal velocity vector
$\sigma'_{ij,j}$: Effective stress tensor
k,α : Material constant of the Drucker-Prager criterion

$$k = \frac{\sqrt{3}c\cos\phi}{3+\sin^2\phi},\ \alpha = \frac{\sin\phi}{\sqrt{3\left(3+\sin^2\phi\right)}}$$

ϕ : Internal friction angle
c : Cohesion of soil
$\bar{\dot{e}}_{ij}$: Equivalent strain rate, $\bar{\dot{e}} = \sqrt{\dot{\varepsilon}_{ij}\dot{\varepsilon}_{ij}}$
λ : Magnitude of the indeterminate component

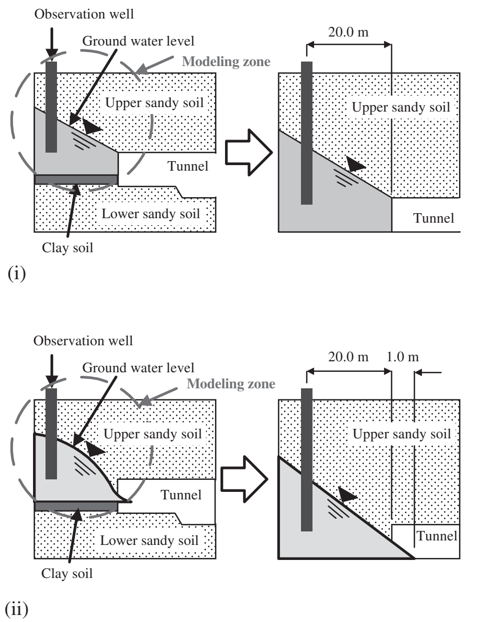


Figure 13. Model for analysis. (i) Ground water level is in the tunnel crown at the face (Model No. 1), (ii) Ground water level is at the lowermost level of the upper sandy soil at the 1 m point behind the face (Model No. 2).

δ_{ij} : Kronecker's delta symbol
$\dot{\varepsilon}_{kk}$: Volumetric component
$\bar{T}_i$: Surface traction
$\sigma_{m'}$: Mean principal stress
σ_m : Effective mean principal stress
s_{ij} : Principal stress difference

The critical values of $\dot{u}_i$, λ and μ are found out by these equations.

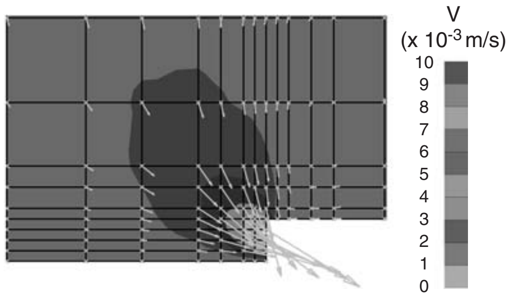
3.2 *Model for analysis*

Sandy soil, in which exists above clay soil at the observation point is modeled in the RPFEA (Figure 13). Parameters for analysis are the height of upper sandy soil at tunnel face, apparent cohesion and the ground water level at the observation well. The ground water level input at the 20 m point ahead of the face, because the pumping well is located at intervals of 20 m toward the tunnel axis.

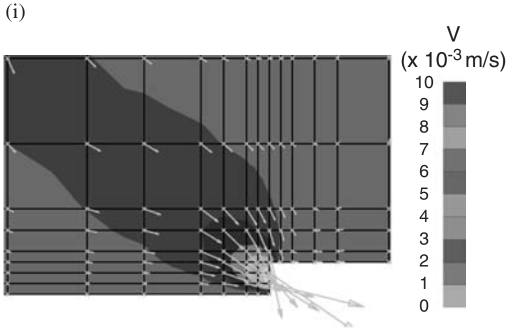
The ground water level is assumed by two types of models in this analysis.

These models are as follows.

– The ground water level is assumed to be a straight line that connects the ground water level of the observation well, which is located at the 20 m point



(i)



(ii)

Hw: Difference between the ground water level at the 20m point ahead of the face and level of the tunnel crown
Hf: Height of a face that can stand by itself

Figure 14. Velocity field at the face collapse. (i) Hf = 4 m, Hw = 0 m, (ii) Hf = 3 m, Hw = +20 m.

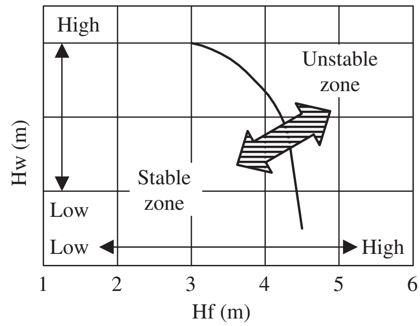
ahead of the face, and the tunnel crown at the face (hereinafter referred to as Model No. 1, Figure 13-i).

– The ground water level is assumed to be a straight line that connects the ground water level of the observation well, which is located at the 20 m point ahead of the face, and the lowermost level of the upper sandy soil at the 1 m point behind the face (hereinafter referred to as Model No. 2, Figure 13-ii).

Here, according to the observational results, the ground water level at the observation well is lowered almost linearly before the face approaches the observation well (Figure 10). Therefore, the ground water level is assumed to be a straight line in both models of this analysis.

3.3 *Analysis result*

Figure 14 shows some results of the analyses, and relative distributions of velocity vectors of displacement at the moment when plastic strain increases abruptly. As the ground water level becomes higher, the ground movement toward the face changes into the lateral direction.



Hw: Difference between the ground water level at the 20 m point ahead of the face and the level of the tunnel crown
Hf: Height of the face that can stand by itself

Figure 15. Concept of nomogram.

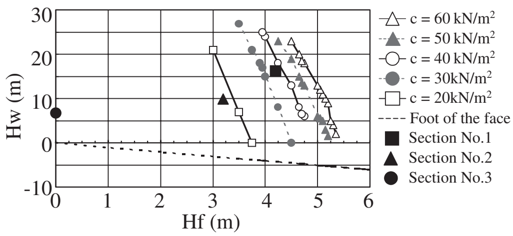
4 PROPOSAL OF A SIMPLE METHOD TO EVALUATE THE FACE STABILITY

Nomograms made up based on the results of RPFEA are proposed to simply evaluate the tunnel face stability. In the nomograms, lines are indices to evaluate the face stability. The left zone shows that the face is stable, whereas the right zone shows the condition of unstability (Figure 15). Since analysis is performed by using two types of models, two nomograms is proposed (Figure 16). These nomograms are as follows.

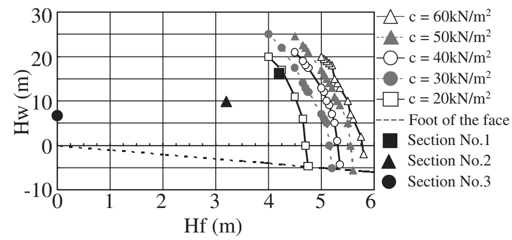
– Nomogram No. 1 is made up based on analysis results by using Model No. 1 (hereinafter referred to as Nomogram No. 1, Figure 16-i).
– Nomogram No. 2 is made up based on analysis results by using Model No. 2 (hereinafter referred to as Nomogram No. 2, Figure 16-ii).

Because the values of cohesion of sandy soil obtained by soil tests varied widely depending on the position and other conditions, several values of cohesion are input. The results indicate that cohesion influences face stability to a large extent. It is thought that the slant of the line expresses the effect of lowering ground water level on face stability. In other words, it is effective to lower the under ground water to the crown level, in order to ensure face stability. The results also show that the height of the face is effective for stability. Therefore, attention must be paid to the stability of the face, when the level of the clay layer goes down.

Next, construction results are input in these nomograms to evaluate the adequacy of analysis. Table 1 shows construction results. When clay soil exists over the entire face (section number No. 3), it shows that the face is stable according to the construction results in the nomograms. On other hand, when an alternation of strata with sandy soil and clay soil is formed at the



(i)



(ii)

Figure 16. Proposed nomogram. (i) Nomogram No. 1, (ii) Nomogram No. 2.

Table 1. Construction results.

Section number	Geological condition	Hf (m)	Face stability	Hw (m)
No. 1	Alternation of strata	4.2	Unstable	16.2
No. 2	Alternation of strata	3.2	A little unstable	9.9
No. 3	Clay soil	0	Stable	6.7

face (section number No. 1), the face is unstable. Since the nomogram based on the analysis coincides with the construction results, it is effective to evaluate the face stability by using there nomograms.

A comparison of the nomograms shows that lowering the ground water level to the foot of the face is effective for face stability.

5 CONCLUSION

– It becomes clear that it is possible to evaluate the face stability, which is affected by the ground water, by applying RPFEA while giving consideration to the effects obtained by the ground water.
– It is confirmed that three factors concerning the lowered ground water level is effective to evaluate the face stability. These factors are the fluctuation of the unit weight of soil due to the fluctuation of the ground water level, fluctuation of the effective stress caused by the fluctuation of the unit weight of soil and pore water pressure, and fluctuation in strength of the ground induced by the fluctuation of the effective stress.

We will compare analysis results and construction data, and study the methods to evaluate cohesion, set an appropriate ground water level and create an optimum model in the future.

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Development of anchor sheet segment and push resin joint

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ABSTRACT: Two technologies have been developed for sewerage tunnels, which are subjected to highly corrosive environments, to improve durability and reduce life-cycle costs. The first technology is the anchor sheet segment method, a technique of tunnel lining, and the second one is push resin (PR) joints, which are corrosion resistant joints for reinforced concrete ring segments. The anchor sheet segment method, in which primary lining is coated with a high-density polyethylene resin sheet, improves tunnel durability and reduces lining thickness and the area of excavation. The metal-free, resin-based PR joints do not require anticorrosive measures and allow speedy construction because segments can be joined by sliding them in the tunnel axis direction.

1 INTRODUCTION

When constructing sewerage shield tunnels, a general approach for the sake of cost effectiveness and construction performance is to use steel segments for primary lining and cast-in-place concrete for secondary lining. In this case, the secondary lining has functions of corrosion prevention, waterproofing, and smoothing of the inner surface, which are not sufficiently achieved by the conventional primary lining.

Measures such as prevention of segment corrosion by secondary lining, rustproofing of metal joints, and filling of bolt boxes are necessary in particular in sewerage tunnels because of possible erosion of the inner surface by sandy particles, degradation of concrete by hydrogen sulfide generated from sewage, and metal corrosion. Moreover, the secondary lining concrete would lose its anticorrosive capability if it was degraded during the service period.

Considering that some resins are durable even in highly corrosive environments, we have conducted research and development on lining technologies that eliminate the need for corrosion proofing or rustproofing even in such environments and that reduce the life-cycle costs of sewerage tunnels.

In this paper, we describe considerations made for the development of the anchor sheet segment method and push resin (PR) joints and report experimental results and the features of these technologies.

2 ANCHOR SHEET SEGMENT

2.1 *Purpose of development*

The conventional approach of secondary lining by cast-in-place concrete has been adopted in sewerage tunnels for preventing corrosion of primary lining, which is a structural component, removing tunnel meandering, ensuring inner surface smoothness, and improving cut-off of water. Recent advancement in construction technologies now allows us to ensure excavation precision, cut-off of water, and inner surface smoothness only by primary lining, but there still is no fundamental solution for the early degradation of concrete. Thus, there is a need to develop a method of durable inner lining.

Anticorrosion lining methods developed for shield tunnels include pasting, spraying, and bolting resin material on the inner surface after tunnel construction. Two processes, tunneling and lining, are required in either method, and there has been a problem of segregation of coating from segments.

The anchor sheet segment method, which uses a resin of high durability and abrasion resistance, was developed as a lining method for sewerage shield tunnels. In this method, inner surface coating is completed by performing only primary lining.

2.2 *Outline of the method*

The outline of the anchor sheet segment method is described below. When manufacturing reinforced concrete segments for the primary lining of the shield tunnel, a high-density polyethylene resin sheet of 2 to 3 mm in thickness is placed in a concrete form. The sheet has stubs on the back side for complete bonding with concrete (Figure 1).

Accordingly, anchor sheet segments with high-density polyethylene resin on their inner surface are manufactured. By using the segments to construct a shield tunnel, inner surface coating is provided simultaneously with primary lining.

2.3 *Mechanical properties of anchor sheet*

The inner coating of a sewerage tunnel needs to be resistant to abrasion and damage caused by flow of solid matter and adaptable to deformation and cracking in the reinforced concrete segments, so that hazardous matter in the sewage will not get in contact with the segments.

To test the performance of the anchor sheet as a sewerage tunnel lining, the basic mechanical properties of the sheet were tested as shown in Table 1.

The test results indicate that the anchor sheet has the following properties:

– large tensile strain, ensuring adaptability to deformation and cracking of concrete
– high abrasion resistance, with the thickness of abrasion being less than one hundredth of that of concrete
– high imperviousness, with a low coefficient of water absorption
– high impact resistance
– low roughness coefficient, compared to concrete

Figure 1. Back side of anchor sheet.

2.4 *Chemical resistance of anchor sheet*

In place of the conventional secondary lining made of cast-in-place concrete, the inner surface coating of a sewerage shield tunnel should protect primary lining from corrosion. The coating should therefore be resistant to various chemicals arising from the sewage.

To confirm the chemical resistance of the anchor sheet, the rate of mass change was measured by an immersion test (Table 2). The anchor sheet is found to be sufficiently resistant to various chemicals and can thus be used as an inner coating for sewerage tunnels.

2.5 *Features of the anchor sheet segment method*

The anchor sheet segment method has the following features:

– Because roughness coefficient is reduced to n = 0.010, compared to n = 0.013 to 0.014 on the inner surface of conventional concrete pipes, inner tunnel diameter can be reduced while retaining the same flow rate capacity.
– The thickness of abrasion is 0.8 mm, compared to 100 mm in concrete. The anchor sheet is highly resistant to abrasion caused by flow of sandy particles.

Table 1. Mechanical properties of anchor sheet.

Item	Unit	Value	Test method
Tensile strength	MPa	15.7	JIS K7113
Tensile strain	%	300	JIS K7113
Rockwell hardness	HRR	18	JIS K7202
Roughness coefficient	–	0.010 (0.014*1)	–
Abrasion coefficient	mm	0.8 (100*2)	JIS A5209 7.8
Coefficient of water absorption	%	0.07	JIS K7209
Charpy impact strength	KJ/m	61.0	JIS K7111

*1 Test value on mortar
*2 Test value on concrete (f_{ck} = 21 N/mm^2)

Table 2. Results of anchor sheet immersion test.

Item		Measured value (mg/cm^2)	Reference value* (mg/cm^2)
NaCl	(10%)	0.00	0.05
H_2SO_4	(30%)	−0.01	0.05
HNO_3	(40%)	+0.01	0.10
NaOH	(40%)	−0.02	0.05
C_2H_5OH	(95%)	+0.09	0.40

* JIS (Japan Industrial Standard) K 6761

- The anchor sheet is highly resistant to chemicals and highly impervious. Secondary lining with cast-in-place concrete, which has been done conventionally to prevent corrosion by hydrogen sulfide and other substances, can therefore be eliminated.
- The low roughness coefficient and the elimination of secondary lining allow reductions in shield diameter and construction cost.
- Stubs on the anchor sheet ensure complete bonding of the resin coating with the concrete.
- The quality of the anchor sheet, which is produced at the factory, does not depend on the construction conditions. Quality control is easier compared with methods that involve lining by spraying, painting, and sheet attachment after segment assembly.

2.6 *Practical application*

The anchor sheet segment method has been used in two construction projects in Japan (Figure 2), which involved installation of pipes having inner diameters of about 2000 mm. Up to now, the method has been applied to tunnels of about 1000 m in total length.

Favorable results have been obtained, with the ability to reduce cost and period of construction confirmed. Some more time is required to demonstrate the high chemical and abrasion resistance of this method.

2.7 *Cost effectiveness*

Cost effectiveness was compared between two cases: secondary lining is applied to a shield tunnel using steel segments in the first case, and the anchor sheet segment method is used in the second case. The results indicate that the total construction cost can be reduced by 5% by adopting the anchor sheet segment method. Because maintenance is not required during the service period in the second case, further reduction in life-cycle cost can be expected.

Figure 2. Shield tunnel completed by the anchor sheet segment method.

Furthermore, construction period can be reduced by 20% because secondary lining is not conducted.

2.8 *Prospects*

The anchor sheet segment method has been put into practical use and its performance has been confirmed successfully.

As a next step, application of anchor sheets to sharply curved sections and sections where reinforced concrete segments are not used is being considered.

Steel segments are being used in sharply curved sections of shield tunnels to increase segment stiffness. In sewerage tunnels, steel segments need to be lined for corrosion prevention and inner surface smoothing.

If there is enough space for thick lining on steel segments, secondary lining by cast-in-place concrete is applied. If there is not enough space for thick lining, anchor sheets are used for secondary lining so that the steel segments can be protected from corrosion by the thin lining.

Anchor sheets are planned to be applied to sharply curved steel segments in a sewerage tunnel being constructed in Japan, and the performance of this work is to be tested.

By applying anchor sheets to steel segment sections, the entire sewerage shield tunnel including sharply curved sections benefits from the high durability, chemical resistance, and abrasion resistance of the sheets. Furthermore, design and construction of sewerage tunnels are expected to be more economic.

3 PR JOINTS (PUSH RESIN JOINTS)

3.1 *Purpose of development*

Steel joints are generally used for shield segments. In sewerage tunnels, however, anticorrosive measures such as rustproofing and mortar finishing on the joints are required because corrosion of steel by hydrogen sulfide and other substances is expected. Steel joints that allow segments to be joined through one-touch operation have recently been developed for practical use, but the cost of these joints in the entire segment construction would be large. Therefore, development of low-cost, compact joints has been required in particular for small-diameter tunnels. Under these circumstances, we developed PR joints, which are corrosion-resistant, cost-effective resin joints for connecting ring segments.

3.2 *Development principals*

About 70% of the sewerage shield tunnels subjected to highly corrosive environments have small excavation diameters of 3 m or less. With the aim of developing joints for small-diameter sewerage tunnels, the following principles were identified:

- Use a material that does not require anticorrosive measures even in highly corrosive environments.
- Achieve cost reduction in comparison with conventional steel joints plus anticorrosive measures.
- Reduce the joint size for adaptation to segments having outer diameters of 3 m or less, which are in greatest demand.
- Design the joints to allow one-touch operation so as to support speedy construction.

3.3 *Material selection*

Among resins that are resistant to acid, alkali, and weathering, polyamide was selected as a joint material because of its high tensile strength and general versatility (Table 3). This resin is thermoplastic and its productivity can be improved by injection molding. Glass fiber is added to the resin for reinforcement, so that the resin retains strength in a compact form.

3.4 *PR joint structure*

As shown in Figure 3, the PR joint consists of male and female parts that mate with each other. The male part has a head <1> and an anchor <2>, whereas the female part consists of threaded fillers <3> and a holder <4>. The threaded fillers on both sides of the holder are held within the holder, and the male and female parts are engaged by the fillers to prevent separation.

3.5 *PR joint performance test*

To confirm the basic performance of the PR joint and make it ready for practical use, the following performance tests were conducted.

3.5.1 *Single joint tensile test*

The tensile strength of a single PR joint was tested (Figure 4).

The test results are shown in Table 4. The average tensile strength of a joint pair is found to be 43.1 kN Assuming a water seal that can cope with a water pressure of 0.25 MPa, the maximum tensile force that acts on any part of the joint is about 13 kN. A safety factor of 3 is therefore achieved.

3.5.2 *Joint insertion and tensile tests*

A joint insertion test was performed to investigate the level of insertion load during segment assembly and whether there is any effect on the concrete around the joint. A tensile test was performed by using the same joint specimen to confirm the tensile strength of the joint embedded in segments and to calculate the tensile spring constant of the joint (Figure 5).

Table 3. Properties of reinforced polyamide.

Item	Unit	Value
Specific gravity	–	1.5
Tensile strength	MPa	205
Flexural strength	MPa	290
Compressive yield strength	MPa	190
Shear strength	MPa	95

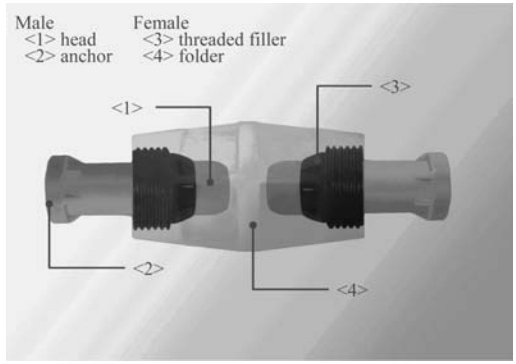

Figure 3. PR joint structure.

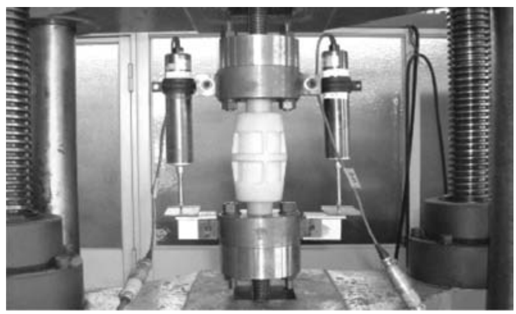

Figure 4. Single joint tensile test.

Table 4. Result of single joint tensile test.

Case	Max. load (kN)	Displacement (mm)
1	42.0	8.1
2	43.7	9.3
3	43.5	9.1
Average	43.1	8.8

The results of the joint insertion test indicate that a joint pair can be put into an engaged state by an inserting force of about 0.2 kN. This means joint insertion can be done by using a usual erector. No

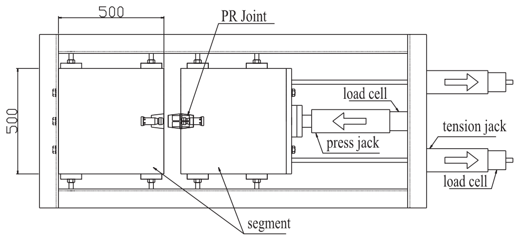

Figure 5. Schematic diagram of insertion and tensile test.

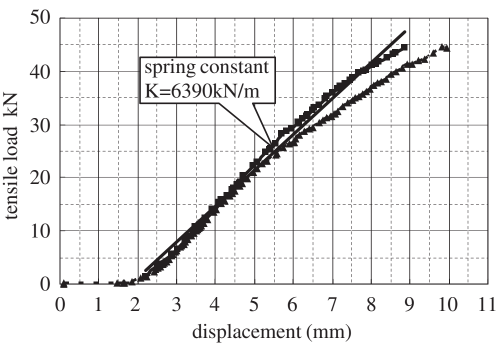

Figure 6. Results of joint tensile test.

irregularities such as cracks were found on the concrete around the joint.

The results of the joint tensile test are shown in Figure 6, where the vertical axis represents tensile load and the horizontal axis represents joint displacement. Excluding the effect of looseness in joint assembly, a tensile spring constant of 6390 kN/m was obtained. A tensile strength of 44.5 kN, which is comparable to the value of 43.1 kN obtained by the single joint tensile test, was also obtained. This indicates that the tensile strength of the PR joint is not changed even when it is embedded in concrete. We have thus confirmed that the joint functions properly without a decrease in strength even in water-added environments, such as those during concrete curing.

3.5.3 *Joint shear test*

A shear test was done by using a platy specimen to confirm the shear strength of the joint and to calculate its shear spring constant. As shown in Figure 7, a platy specimen simulating a main segment and a splice segment was connected by using four joints. A vertical load was applied to the main segment so that the joints were subjected to shear force. Two cases were tested: presence and absence of a gap between the main and splice segments.

The test results are shown in Figure 8, where the vertical axis represents vertical load and the horizontal

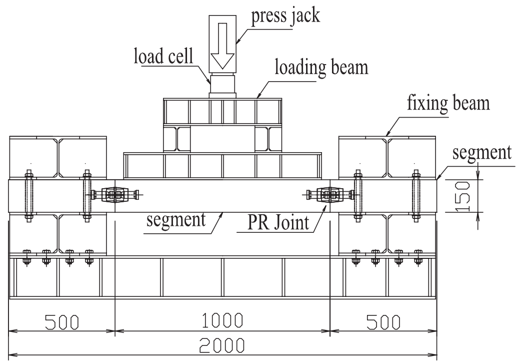

Figure 7. Schematic diagram of joint shear test.

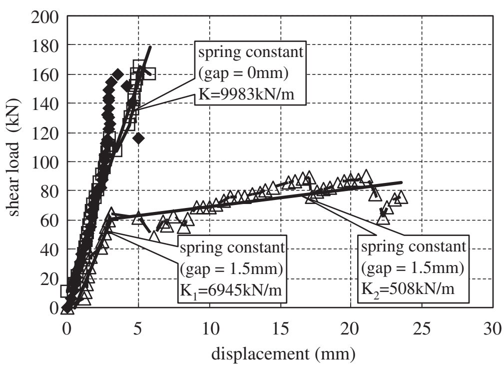

Figure 8. Results of shear test (specimen containing four joints).

axis represents vertical displacement. Maximum load was smaller when there was a gap between the segments, and different shear spring constants were obtained for the two cases. A joint shear strength of 22.6 kN/joint and initial shear spring constants of 7000 to 10,000 kN/m were obtained from the test results.

3.5.4 *Shear creep test*

Because resin materials are generally susceptible to creep deformation, we had to investigate the creep behavior of the joint under long-term shear force to bring the joint into practical use. To investigate the creep behavior, shear force was applied to a joint specimen by using a test device that is similar to the one used in the joint shear test. Creep loads of 3.2 and 4.8 kN were assumed in consideration of the maximum shear force (4.04 kN) that was expected to act on the joint from segment design on a trial basis described in the next section.

The results of the creep test are shown in Figure 9.

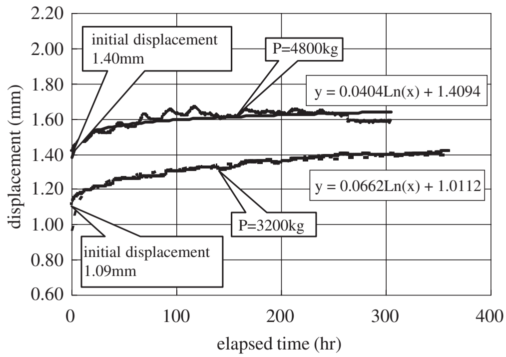

Figure 9. Results of shear creep test.

Based on an approximate expression deduced from the test results, creep deformation after the elapse of 50 years is estimated to be 1.9 mm. This is within the range where water can be cut off by usual seal.

3.6 *PR joint applicability*

Based on the results of the above-mentioned tests, we specified parameter values for the design of the PR joint as shown in Table 5.

By using the specified values of design parameters, we designed segments on a trial basis. While assuming an effective overburden of 10 m and a finished inner diameter of 2800 mm, soil conditions were varied as parameters.

The results of the trial design (Table 6) indicate that shear force τ on the joint has a maximum value of 4.04 kN when N-value is less than 2 and the soil type is alluvial cohesive soil. Because the shear strength of the PR joint obtained by the shear test is 22.6 kN even when there is a gap between the segments, safety factor Fs for the shear force that acts on the joint is 5.6. This indicates that the PR joint can also be applied to soft ground.

3.7 *Features of PR joints*

The key features of segments using PR joints are as follows:

– The joint, made of polyamide-based resin, is resistant to corrosive substances such as hydrogen sulfide and salt.
– Segments can be joined by one-touch operation without bolting, so that speedy construction is supported.
– Because joints are embedded within segments, tunnels with smooth inner surfaces can be constructed.
– Manufacturing and transport of segments are facilitated because joints do not protrude from segment sides.

Table 5. Design parameters of PR joint.

	Spring constant (kN/m)	Strength (kN)
tensile	6390	44.5
shear	6945~9983	22.6

Table 6. Results of trial design.

Case	1	2	3
Soil type	sand	clay	clay
SPN value	30<N	4<N<8	N<2
Loading Condition	loosened earth pressure	full overburden	
Displacement(mm)	−1.21	−2.35	−2.61
Shear stress (kN/joint)	1.32	2.79	4.04
Judgment	OK	OK	OK

– There is greater freedom in taper segment assembly because segments can be used regardless of their entrance and face sides.

3.8 *Cost effectiveness*

The cost effectiveness of PR joints was compared with that of steel bolt joints, which are lowest in price. A cost cut of 10% is expected by using PR joints, in comparison with the use of steel bolt joints combined with anticorrosive measures. This is because the resin used for the PR joint is low priced compared to metal in general and does not require anticorrosive measures. In the future, the cost of the PR joint may be further reduced by mass production and streamlining of manufacturing.

Because maintenance is not required during the service period, the PR joint is obviously advantageous in terms of life-cycle cost.

3.9 *Prospect*

The basic performance of the PR joint, including its structural behavior and applicability, was confirmed by various performance tests. By designing segments on a trial basis, we confirmed that the PR joint can well be applied to practical use.

The PR joint also surpasses conventional joints in economy.

In the future, trial construction will be conducted for further confirmation, and resin-based joints for combining pieces are to be put into practical use with the aim of developing segments that use no metal joints.

4 CONCLUSION

With the aim of improving the durability and life-cycle cost of shield tunnels, the anchor sheet segment method and the PR joint were developed while focusing on the high durability of resin material. The development enables reductions in the total cost of sewerage tunnels.

Adopting the anchor sheet segment method allows omission of secondary lining, reductions in roughness coefficient, excavation area, and construction cost as well as maintenance cost because of the high durability of the resin. Adopting the PR joint allows joint cost reduction, maintenance cost reduction because of improved corrosion resistance, and extension of the service lives of structures.

In the future construction of sewerage and other tunnels in highly corrosive environments, the high durability of resin can be used effectively in providing a rational anticorrosive measure to allow life-cycle cost reduction.

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Observational method for tunnel construction in difficult conditions considering environmental impact to groundwater

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ABSTRACT: This study reports on the application of observational method to groundwater work in mountain tunnel construction, which was performed for the purpose of making the work economical and rational while dealing with tunnel related groundwater issues, and describes the development of suitable analytical and evaluation techniques. The tunnel to which this method was applied is a mountain tunnel excavated in alternating sand stone and slate from between the Mesozoic and Paleozoic Eras. Some sections on the route were going straight below surface waters or near important lakes, imposing environmental and groundwater requirements. In the application example presented, the observational method based groundwater work made it possible to effectively evaluate groundwater and thereby make the work economically efficient and environmentally friendly by incorporating the proposed evaluation technique in the excavation process.

1 INTRODUCTION

In Japan, tunnel work has conventionally been based on engineering knowledge such as the discipline of rock mechanics. Due to the recent trend of revitalizing urban functions through improvement of traffic networks, there is often no choice but to construct tunnels near built-up areas. In such a case, not only the construction work itself but also the surrounding natural and social environments raise crucial issues, and as a result, it is often essential to take relevant conservation measures to deal with environmental factors[1),2)]. The authors have so far examined groundwater prediction methods to help establish groundwater environment conservation measures [3)].

2 OVERVIEW OF OBSERVATIONAL-METHOD-BASED GROUNDWATER WORK

The groundwater issues involved in tunneling have two aspects: the work itself and the surrounding environment. With regard to the work itself, it is concerned with the need for groundwater measures for safety purposes, such as through prevention of cutting face failure that could be incurred by a large amount of leakage. With regard to the surrounding environment, it is concerned with such issues as the impacts of groundwater drainage on the surrounding groundwater environment and the impacts of groundwater level reduction on existing water resources. This study reports on the application of observational method to groundwater work in the construction of Inariyama Tunnel, which was performed for the purpose of making the work economical and rational while dealing with tunnel-related groundwater issues, and describes the development of suitable analytical and evaluation techniques. The observational method based groundwater work developed in this study is a system of tracking and evaluating the excavation process by rapidly repeating the processes of validation and prediction analysis based on the immediately preceding excavation data and consists of the following analytical stages:

(i) Groundwater model development: Based on the topographical, geological and groundwater information obtained prior to tunnel excavation, a large three-dimensional prediction model representing the hydro-geological conditions, aquifer structure and surface waters is developed.

(ii) Validation analysis immediately preceding excavation data: Using the three-dimensional model, complex groundwater behavior during excavation is predicted through three-dimensional saturated-unsaturated seepage analysis[4]. The immediately preceding excavation data are feed backed to the prediction model to verify the model.

(iii) Prediction analysis: The groundwater behavior of as-yet-excavated area is predicted while attempts are made to improve the precision of prediction model. In the construction of the Inariyama Tunnel, the incorporation of proposed observational method for three sections, namely Sections A, B and C shown in Figure 1 (running close to a aqueduct tunnel, surface water or precious lake) allowed for the successful implementation of relevant conservation measures.

3 CLOSE POSITIONAL RELATIONSHIP WITH EXISTING AQUEDUCT TUNNEL (SECTION A)

3.1 *Tunneling conditions and surrounding environment*

Topographically, Section A was composed of gently inclined, highly weathered and eroded mountainous slopes having hilly areas and alluvial fans at their base. The surrounding geology consisted of the Tamba layer group (clayslate, shale, sandstone and chert) between the Carboniferous Period of the Paleozoic Era and the Jurassic Period of the Mesozoic Era. The strike of the strata ran from northwest to southeast; whose dip was generally south facing; and there were several fault-crushing belts running from northeast to southwest.

The Inariyama Tunnel had double cutting faces, namely eastern tunnel and western tunnel main tunnel, whose sectional area was both 80 m^2, as shown by the standard section in Figure 2. The existing aqueduct

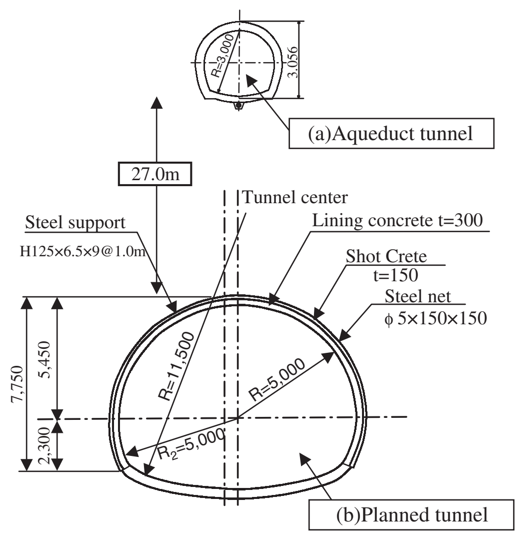

Figure 2. Tunnel cross section.

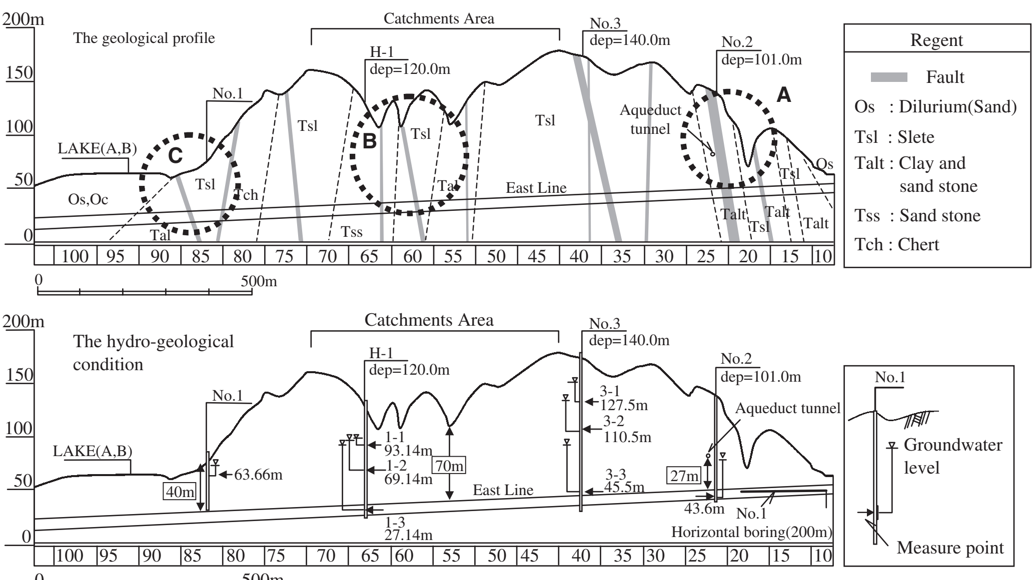

Figure 1. The geological profile of tunnel longitudinal section.

tunnel, which presented a serious issue by running closely to the planned tunnel, had a flow volume of 12,000 to 13,000 m^3/hour and was one of the crucial water supply lifelines.

The aqueduct tunnel and planned route were running closely to each other, with a clearance of approximately 27 m and a crossing angle of 57°. According to the results of Lugeon test on No. 2 borehole, the crossing

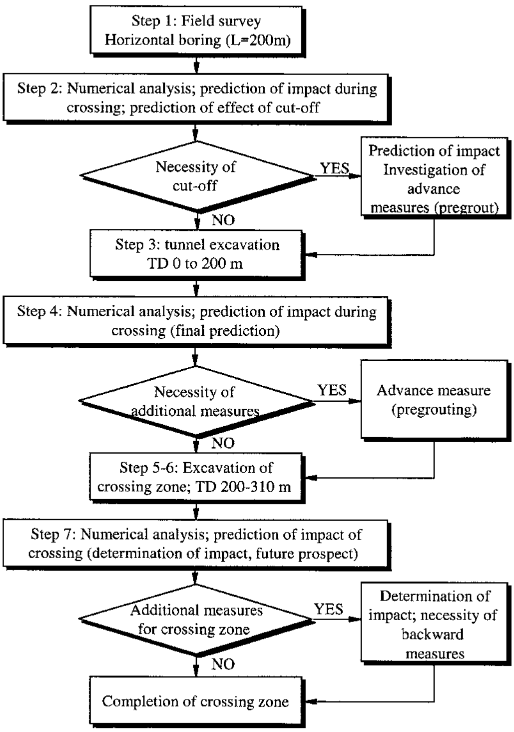

Figure 3. Flowchart of observational method.

point had the following hydro-geological characteristics: the permeability coefficient of rock with fewer cracks was $k = 3.0 \times 10^{-5}$ to 1.0×10^{-4} cm/s, while that of the fault-crushing belts showed a large value of $k = 1.0 \times 10^{-2}$ cm/s. Viewed from the well core, the fault-crushing belts were found in every 2 to 5 m, and as a result, the whole ground was forming a huge fault-crushing belt. The groundwater level inside the well core was approximately GL-60 m, which was close to the water surface level inside the aqueduct tunnel. As a result of the No. 1 horizontal boring performed before the commencement of work, a great amount of leakage water was found inside the borehole and the fault-crushing belts were suffering intensive leakage, showing the actual leakage water of 500 to 700 L/min, while other regions showed 100 L/min or less. The permeability coefficient of fault-crushing belts obtained in the Lugeon test was $k = 1.3 \times 10^{-2}$ cm/s.

3.2 *Observational-method-based groundwater work in Section A*

The procedure for observational-method-based groundwater work in Section A is shown in Figure 3. It is a procedure of repeatedly modifying the prediction model based on horizontal boring and preceding excavation data to predict the impacts on yet-excavated areas, thereby evaluating the necessity of measures or the size of predicted impacts. Figure 4 shows the prediction model for Section A.

In Section A, tunnel excavation was found to give the following impacts: either drawing a large amount of water from the drinking water lifeline, i.e. the aqueduct tunnel, through highly permeable fault-crushing belts, or loosening the ground around the aqueduct tunnel by lowering the groundwater level. However, since the aqueduct tunnel did not allow the direct observation of impacts, it was considered effective to evaluate the impacts using the impact values obtained

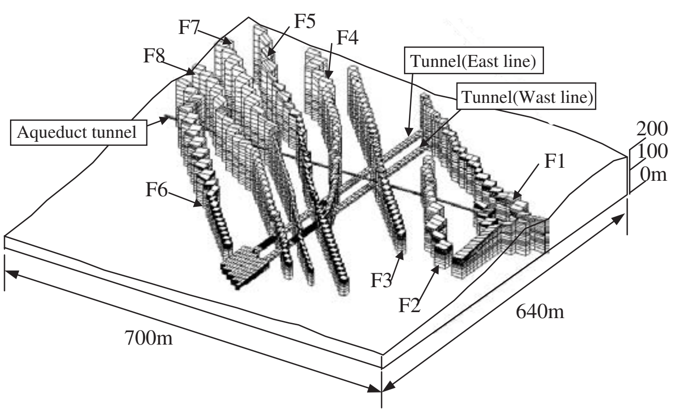

Figure 4. The prediction model for Section A.

in numerical analysis. Based on the water volume ratio to the current flow volume, the allowable impact was determined to be the seasonal fluctuation of up to 4.0 to 7.6 m^3/min.

3.3 *Validation analysis during excavation of Section A*

3.3.1 *Evaluation of hydraulic characteristics of fault-crushing belts (Step 2)*

In Step 2, the properties of fault-crushing belts (existence and width) were added or modified using the actual horizontal boring data, and the horizontal boring was reproduced on the analytical model, faithfully representing the measured drilling process. At this stage, verification was conducted based on the actual amount of leakage during boring, and the hydraulic characteristics such as the permeability coefficient were obtained. Then, the groundwater behavior around the crossing point during main tunnel excavation was predicted and the impacts on the aqueduct tunnel evaluated. Furthermore, the need for measures was examined based on the results of preliminary prediction. According to the results of Step 2, the amount of drawing water was predicted to be 1.15 m^3/min, which suggested there was no need to take any cutoff measure such as large-scale chemical injection, and that it would be adequate to pay attention to the fault-crushing belts and adopt cutoff measures when it was considered necessary.

3.3.2 *Verification of prediction results and measured data (Step 4)*

Up to the crossing point, the actual leakage showed a peak amount of 1 to 2 m^3/min, which rapidly declined in accordance with the progress of tunneling.

Figure 5 is the comparison between the prediction results and measured data of the groundwater level of No. 2 borehole and main tunnel leakage. According to the results of impact prediction in Step 4, it was predicted that the tunnel leakage would increase near the crossing point to approximately 2.0 m^3/min, and the groundwater level of No. 2 borehole would rapidly decrease accordingly. The measured data mostly corresponded to the prediction results both in terms of groundwater level and tunnel leakage. Figure 6 shows changes in pressure head distributions at the crossing point. It helps understand that the aqueduct tunnel was affected by way of the fault-crushing belts crossing with the tunnel route, and the impacts of total head reductions during excavation were extended to the vicinity of ground surface along the fault.

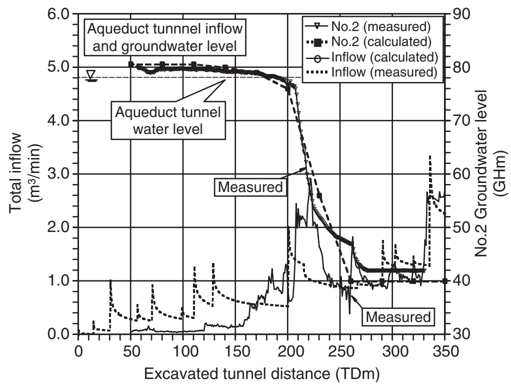

Figure 5. The comparison between the prediction results and measured data.

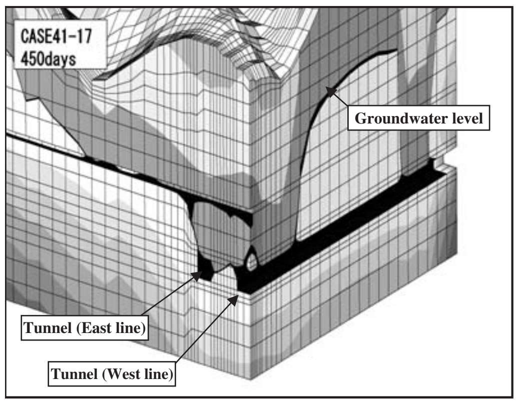

Figure 6. Result of groundwater level change(Step4).

3.4 *Reflection of impact evaluation results on actual work*

The repeated modifications of analytical model and execution of numerical analysis in accordance with excavation progress made it possible to precisely identify the groundwater behavior and impact development mechanism. Making them consistent with the monitoring data allowed the highly reliable impact prediction. In the actual work, since the amount of tunnel leakage and the impacts on aqueduct tunnel were below the allowable impact values, it was determined unnecessary to take any cutoff measures, such as grouting, which had been considered before the commencement of excavation. Thus, no specific measures against any groundwater impacts were taken.

4 EVALUATION OF IMPACTS ON SURFACE WATER SYSTEM (SECTION B)

4.1 *Observational method based groundwater work in Section B*

The prediction model used for Section B is shown in Figure 7. The allowable impact values were determined

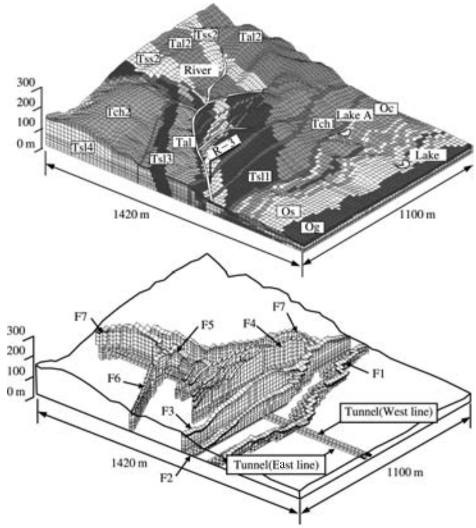

Figure 7. Distribution of fault zone in the 3-D geological profile for Section B, C.

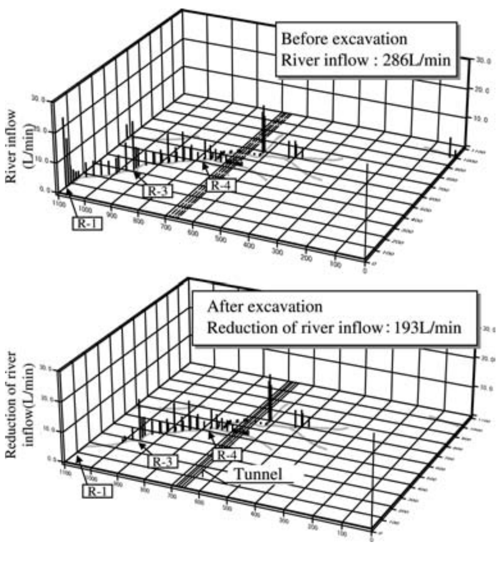

Figure 8. Results of river inflow.

using the base flow of surface waters as the reference value. As an emergency measure during work, it was determined to restore the surface water by providing publicly supplied water, the volume of which was proposed based on the results of validation analysis.

4.2 *Validation analysis during work on Section B*

Figure 8 shows the results of impact analysis for the flow of surface waters in Step 0. The upper diagram shows the condition where the excavation was not giving any impacts. The lower diagram shows the distributions of impact reduction while excavating through Section B. From the Figure, it was predicted that the base flow was 286 L/min before passing through the tunnel and then reduced by 193 L/min after passing, which would be a reduction of approximately 70%. The reduction of surface waters was particularly significant near the tunnel.

4.3 *Reflection of impact evaluation results on actual work*

As compared with the base flow of 150 to 200 L/min obtained from past hydrological survey records, it was predicted that the excavation work would almost dry up the surface waters. As an emergency measure in the actual work, the pumping-up procedure was taken, utilizing publicly supplied water.

5 EVALUATION OF IMPACTS ON NEARBY LAKES (SECTION C)

5.1 *Observational-method-based groundwater work in Section C*

Figure 9 shows the procedures for observational-method-based groundwater work to deal with lakes in Section C. The evaluation of the impacts on lakes meant the determination of end point location for mountain tunneling by NATM. In other words, it was to do with the determination of the mountain tunnel section to be excavated by the drainage type NATM, which would be able to minimize the impacts on nearby lakes. There was F1 Fault near the shield machine rotation shaft, which suggested that the section could be prone to problematic groundwater behavior in the fault-affected area and impacts on the groundwater recharge mechanism of the lakes. In other words, the observational-method-based groundwater work in Section C focused on the evaluation of impacts in accordance with the progress of main tunnel excavation, as well as the hydraulic characteristics of F1 Fault.

5.2 *Validation analysis during work on Section C*

Prior to the excavation of Section C, horizontal boring was performed for the purpose of confirming groundwater and the geology of the area near the shield machine rotation shaft. For Section C, the results of horizontal boring were treated as the evaluation target. Two cases, namely the maximum and minimum impacts were assumed (in Step 3), and the boring work itself was covered in the validation analysis.

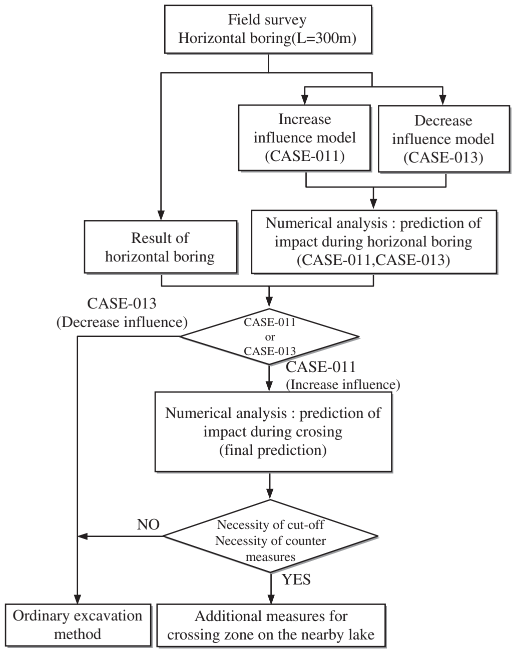

Figure 9. Flowchart of observational method.

5.2.1 *Validation of drained water during horizontal boring*

Through comparison between the prediction results for the two analytical conditions, namely the maximum and minimum impacts, and the measured horizontal boring data, it was confirmed that there would be a relatively large amount of drained water during horizontal boring, showing a relatively good correlation to the maximum impact condition (CASE-011) as far as shown by the measured data. For this reason, the CASE-011 model was used for the impact analysis of main tunnel excavation.

5.3 *Prediction results and impact evaluation in Section C*

Given the results of validation analysis described above, the prediction analysis for the impacts on lakes during main tunnel excavation was performed, assuming the maximum impact condition (CASE-011), where F1 and the disturbed sand-mud alternating layer before it both had a high permeability of $k = 10^{-3}$ cm/sec.

5.3.1 *When excavating F1 fault by NATM*

Figure 10 shows the prediction of tunnel leakage in relation to the boring length during main tunnel excavation after drainage boring, the water level of No. 1 borehole and the amount of seepage from the lakes. The main tunnel leakage was almost none at F2 before

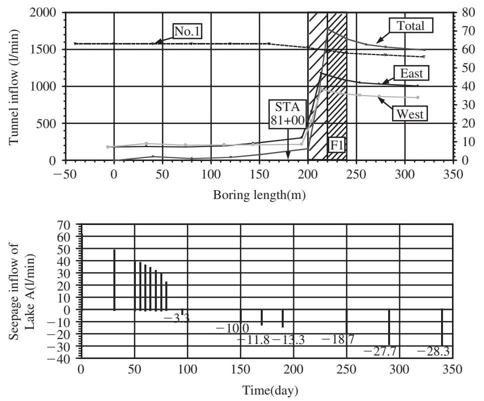

Figure 10. Seepage inflow and tunnel inflow(CASE-011).

F1 due to the drainage effect of horizontal boring, but then increased to approximately 650 to 850 L/min in the sand-mud alternating layers near and through F1 in both westbound and eastbound main tunnels. Then, as in the case of horizontal boring, the increase stopped in the Osaka group at the far end. While the amount of drained water showed such a tendency, the reduction in the water level of No. 1 borehole was the sum of the reduction caused by horizontal boring and the reduction caused by the progress of main tunnel excavation. The final water level reduction caused by main tunnel excavation was 4 to 6 m. Together with the reduction caused by horizontal boring, the total water level reduction was predicted to be approximately 10 to 12 m. The amount of seepage from the lakes was decreased in accordance with the lowering of water level. The initial amount of seepage before excavation was 64 L/min, and the progress of main tunnel excavation was predicted to result in the final seepage of approximately 28 L/min.

5.3.2 *When the excavation is stopped at No. 81+00 before F1 fault*

The prediction results for the case where the main tunnel excavation was stopped before F1. The main tunnel leakage was almost none at F2 before F1 due to the drainage effect of horizontal boring, but then increased to approximately 100 to 120 L/min in the sand-mud alternating layers just before F1 in both westbound and eastbound main tunnels. While the amount of drained water showed such a tendency, the water level of No. 1 borehole showed almost no reduction. The final water level reduction was predicted to be approximately 8 m, which was caused solely by horizontal boring. The initial amount of seepage before excavation was 64 L/min, and the progress of main tunnel excavation was predicted to result in the final seepage of 0.4 L/min. Therefore, it

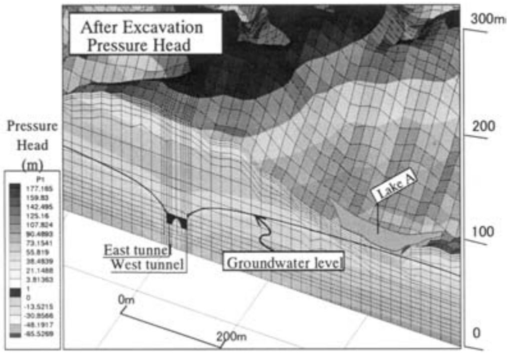

Figure 11. Results of ground water level change around the lake (Lake A).

was evaluated that the impacts on the lakes could be well controlled if the main tunnel excavation was stopped before F1. Figure 11 shows the reduction of groundwater level between the tunnel and lakes and pressure head distributions inside the ground.

5.4 *Reflection of impact evaluation results on actual work*

In controlling the impacts on lakes during tunneling work, it was considered necessary to take measures to control the amount of water drainage during main tunnel excavation. Because the impacts on lakes and surface waters are the reflection of complex groundwater behavior and aquifer structure, it is highly necessary to develop an appropriate drought prevention plan with the consideration for drought impacts and thereby control leakage and monitor surface water reduction. Based on the results of impact analysis, preparations have now been made to take necessary measures to restore the water shortage during and after the work.

6 CONCLUSIONS

The proposed observational method for groundwater work makes it possible to perform highly reliable tunneling work by accurately feed backing the data obtained for each tunneling step to the actual work. As compared with the conventional impact evaluation method using monitoring data, the use of analytical models derived from past construction records has the advantage of identifying the impact development mechanism in advance, allowing the quantitative prediction and evaluation of impacts. This would be impossible when only limited monitoring data are available.

Since the conservation of groundwater environment is difficult to evaluate in advance, it often takes the form of follow-up measures after the ex-post impacts have been confirmed in the conventional practice. However, the increasing call for environmental conservation has resulted in the pressing need for prior preparations before incurring any problems. With this regard, the importance of prior evaluation based on the groundwater behavior mechanism is supposed to be emphasized. In addition, since the proposed method can help reduce the risk of cost over-run by enhancing reliability through reevaluation during work, it is supposed to advance the efforts to implement prior preparations for environmental conservation purposes.

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Research plan and construction outline of the Mizunami Underground Research Laboratory (MIU) in large depth for geoscience and geological disposal fundamental research

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ABSTRACT: One thousand meter-deep shafts and several level drifts will be excavated in the Mizunami Underground Research Laboratory (MIU) Project. Design and layout of the shafts and drifts are determined considering the geological and project condition, schedule, and investigation plan for characterization of the geological environment and engineering technology for the fundamentals of geological disposal.

This paper describes the design and construction condition, layout and design of the shafts and drifts of the MIU, and mechanical stability, ventilation and environmental concerning. Details of the design of shafts and drifts are determined, and safety of the design and construction of the MIU are discussed. Countermeasures and safety are also described in this paper.

1 INTRODUCTION

Mizunami Underground Laboratory (MIU) is one of the facilities to be constructed deep in the ground under a long-range plan for nuclear energy development in Japan. The principal target of research is crystalline rock (granite).

The MIU Project involves research on the fundamentals of geological disposal technology for high-level radioactive wastes. The project is being implemented by Japan Nuclear Cycle Development Institute (JNC) as part of fundamental geoscience research for the research and development of geological disposal technology.

The research shafts and galleries excavated under the project are expected to serve as places for establishing the technology for comprehensive investigation of the deep geological environment and developing the foundations of engineering technology for deep underground applications. The research shafts and galleries are also expected to help further the public's understanding of the deep geological environment as indicated in the Long-term Program for Development and Utilization of Nuclear Energy.

The program calls for the excavation of shafts and galleries at depths of up to 1,000 m from four viewpoints:

1. Use as places for the demonstration of the reliability of geological disposal technology at an estimated depth of 1,000 m in hard rock formations and, at the stage (i.e., the current stage) where a disposal site has not yet been selected, for fundamental research and development to be performed after constructing shafts and galleries down to a promising depth
2. Significance from the standpoint of engineering technology of shaft and gallery excavation at great depths where higher levels of technology is needed for groundwater and rock burst control
3. Significance as geoscience research concerning deep formations required by the plan drawn up by the national government
4. Economy of shaft excavation

The MIU Project was launched in 1996. In January, 2001, it was decided to excavate research shafts and galleries at a site in a tract of land owned by Mizunami City. Then, the research shafts and galleries and facilities and equipment necessary for shaft and gallery excavation were designed. In March, 2003, the first phase of research shaft and gallery excavation (excavation to a shaft depth of 300 m to be completed by March, 2005) began.

Research on the technologies for the design of research shafts and galleries in deep formations, construction planning, the excavation and maintenance of research shafts and galleries, and quality control may be thought of as research on the foundations of engineering technology for deep underground applications.

The purpose of the present study is to design research shafts and galleries that can be used in heterogeneous geological environments and that make it possible to conduct research at depths of up to 1,000 m; show examples of construction plans and develop a systematic approach to construction planning; and demonstrate the feasibility of constructing research facilities and disposal facilities in deep formations.

This paper reports on the construction plan for the MIU Project, focusing on the design and excavation methods for research shafts and galleries and safety measures such as the research shaft and gallery layout and cavern stability.

2 CONSTRUCTION PLANNING

2.1 *Basic requirements for planning*

(1) *Site conditions*
The underground research shafts and galleries for the MIU Project will be constructed in a tract of land owned by the Mizunami Municipal Government located in the town of Akiyo in Mizunami City, Gifu Prefecture. JNC leases about 7.8 ha of land from Mizunami City and install equipment for excavating research shafts and galleries in a one-hectare area of prepared land. The construction site is close to many public facilities including a park and hands-on learning facilities. In locating the equipment, therefore, it is necessary to take into consideration the fact that many citizens including school children visit those facilities.

(2) *Topography and geology*
The municipally-owned tract of land mentioned above is a hilly area with an elevation of about 200 m. In this area, granite (Toki granite) masses from the Cretaceous Period of the Mesozoic Era are distributed widely, and the granite masses are covered by Neogene sedimentary rock formations (Mizunami group). It is estimated that the sedimentary rock/granite interface at the excavation site is at a depth of 170 m.

(3) *Investigation and research plan*
The investigations and research for the MIU Project are conducted in three major phases.

The investigations and research conducted in the phase (Phase 2) in which research shafts and galleries are excavated can be classified into two groups: (1) water pressure and water quality monitoring using boreholes drilled from the ground surface, and (2) investigations and tests concerning the geology, groundwater hydraulics, groundwater geochemistry and rock mechanics measured through investigations and tests conducted in shafts and galleries.

The investigations and research in the latter group need to be coordinated with construction activities such as shaft and gallery excavation. In cases where the effect of shaft excavation on the surrounding rock or groundwater is investigated, it is necessary to start the investigations before shaft excavation.

The research conducted in the phase (Phase 3) in which the research shafts and galleries are used is mostly carried out in shafts or galleries. The galleries need to be laid out so that investigations and research in different fields such as those listed below can be carried out in appropriate zones. When conducting investigations and research, it is necessary to have the ability to flexibly allow changes in research shaft and gallery design or construction plan on an as-needed basis.

- Geological structure investigations: geological investigation through shaft/gallery wall observation, shaft/gallery investigation of 3D geology, water quality mechanism investigation, etc.
- Hydraulic investigations of groundwater: borehole permeability test, full-scale permeability test, permeability-under-thermal-stress test, crack-by-crack or fault-by-fault permeability test, etc.
- Geochemical investigations of groundwater: geochemical monitoring, oxidation–reduction study, water quality mechanism study, etc.
- Mass transfer investigations: large-scale mass transfer test, crack-by-crack or fracture zone-by-fracture zone mass transfer test, etc.
- Rock mechanics investigations: shaft/gallery excavation disturbance test, long-term behavior test, in-situ rock mass test, stress measurement, etc.
- Engineering technology: test of influence of artificial materials on rock masses, excavation disturbance reduction/rehabilitation test, coupled behavior test, etc.

(4) *Materials and equipment to be brought in*
The dimensional requirements for shafts and galleries need to be determined taking into account such factors as the efficiency and economy of excavation work, materials and equipment to be brought in and the research space requirements. Large equipment and boring machines necessary for the excavation of galleries and for research are to be brought in through the shafts. When designing shafts and planning their construction, therefore, it is important to ensure that

those machines and equipment can be disassembled and brought into the shafts. Similar considerations apply to galleries as well. JNC's experience in past research projects indicates that the gallery diameter should be at least about 3 m.

(5) *Entrants*

Persons who enter shafts during shaft excavation may include not only excavation workers but also JNC employees, university or other researchers, and visitors.

(6) *Construction schedule*

The MIU Project aims to yield research results that can be reflected in disposal projects implemented by Nuclear Waste Management Organization of Japan (NUMO) and the safety standards and guidelines formulated by the national government. It is assumed that the shaft excavation results will be used as the technological foundation NUMO needs to select detailed investigation areas by around 2010. Major milestones of the MIU Project, therefore, are to reach a shaft depth of 1,000 m by around 2009 and complete the elevator by around 2010. In order to achieve these milestones, the excavation of the shaft entrance will be started in mid-2003, and shaft excavation using a drilling rig will be started by the end of 2004.

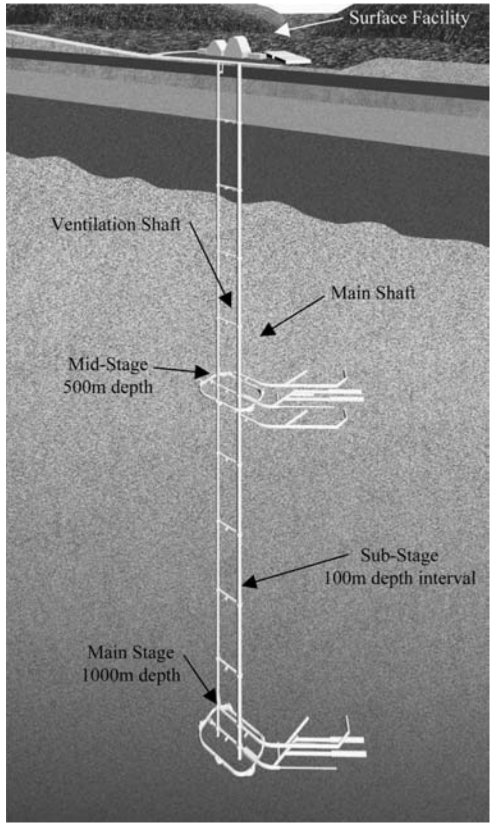

Figure 1. Layout of the research shafts and galleries of MIU.

2.2 *Facility design*

(1) *Layout and functions of research shafts and galleries*

Investigation and research items for the MIU Project have been identified, and the layout of research galleries has been determined by referring to data obtained from existing boreholes and four adjacent deep boreholes. The basic layout is as follows:

- Shafts: two shafts (main shaft and ventilation shaft)
- Galleries: horizontal drifts excavated at two depths (mid-depth level, bottom level) of the two shafts

The geology of the MIU Project site is similar to that of the adjacent areas mentioned above. The sedimentary rock formation at the MIU Project site is slightly thicker, but the bedrock is identical (Toki granite). The layout in the municipally owned tract of land in Mizunami City, therefore, was modeled after the proposed basic layouts in the adjacent areas, and the standard layout was developed, taking into consideration such conditions as the site boundary and the construction schedule.

Figure 1 shows the layout of the research shafts and galleries. The layout will be reviewed from time to time on the basis of newly acquired geological environment information and details will be added accordingly.

The functions of each shaft and gallery are as follows:

- Main shaft: serving as the route through which excavated material generated by gallery excavation is removed and heavy equipment necessary for excavation is brought in and out.
- Ventilation shaft: space in which the elevator is installed after the completion of excavation to the depth of 1,000 m and a ventilation shaft for the entire shaft and gallery system.
- Mid-depth stage and main (bottom) stage: the principal places of research in Phase 3 (research using shafts and galleries).
- Test galleries: places where tests involving gallery excavation are conducted.
- Sub-stages: passageways between the two shafts and spaces where pump stations for removing groundwater flowing into the research shafts and galleries.

(2) *Specifications for research shafts and galleries*
The inside diameter requirements for the shafts and galleries have been determined, taking into account the functionality and constructability requirements for the shafts and galleries. The inside diameter requirements for the galleries have been determined so that investigations and tests can be conducted and construction work can be carried out easily. The dimensions of the shafts and galleries are as follows:

- Main shaft: inside diameter: 6.5 m, depth: 1,025 m
- Ventilation shaft: inside diameter: 4.5 m, depth: 1,010 m
- Mid-depth and bottom stages: inside diameter: 3–8 m, length: 785 m
- Substages: inside diameter: 3 m, length: 35 m.

(3) *Analysis of mechanical stability of caverns*
For the analysis of the stability of the shaft and gallery caverns and the determination of support requirements (e.g., lining concrete, shotcrete, rock bolts, H-beams), methods used in the field of civil engineering such as tunnel engineering in which details are determined according to the rock mass conditions. Since, however, the depths involved are great and there are special sections such as connections between shafts and galleries, the numerical analysis method was used for those sections.

The rock mass conditions and the input conditions for numerical analysis were determined on the basis of the results obtained from existing boreholes (depth: 500 m) adjacent to the municipally owned tract of land in Mizunami City. Analyses were conducted, parameterizing the stress acting on the rock mass and the rock mass condition.

First, as a simple analysis, an elastic analysis was performed for allowable stress evaluation. In cases where the allowable stress was exceeded, the tunnel supports were strengthened and, by conducting a detailed two-dimensional FEM (finite element method)-based elastoplastic analysis, support conditions were modified.

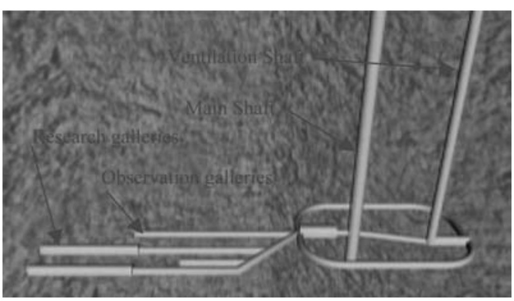

Figure 2. Detailed layout of the research galleries of main stage of MIU.

For the connections between the shafts and galleries, three-dimensional FEM analyses were conducted to evaluate their three-dimensional effects. From analysis results, strains are concentrated in the connection zone and near the corners, but their distributions are limited.

On the basis of the three-dimensional FEM analysis results, the degree of influence of the mechanical stability of the connection zone on a typical shaft cross section was analyzed, and it was ascertained that the allowable stress for the support of the typical cross section was not exceeded.

(4) *Equipment design based on ventilation analysis and fire response measures*
Fire is one of the hazards that are most likely to occur in the research shafts or galleries, so the most important thing is to ensure that there will be evaluation routes and enough time for evacuation in the event of a fire. It was thought that it would be difficult to secure a safe space, other than the two shafts, that makes it possible to escape to the ground surface in the event of a fire (i.e., a third shaft) or to secure safe fire compartments within the two shafts. It was also thought that there might not be enough evacuation time. As the basic concept of safety measures, therefore, it was decided that in the event of a fire, people in the shafts or galleries should evacuate to shelters, and requirements such as safe locations (safe compartments), evacuation time, and the size, shape and other attributes of shelters were identified. To identify shelter requirements, a ventilation network analysis, which is often conducted in the field of mining, was performed, assuming cases of fire emergency.

The construction process was divided into a number of stages, and consequences of a fire were analyzed for different fire sources. The shelter locations and specifications were determined on the basis of the analysis results thus obtained, and it was decided to locate shelters in all galleries. Ventilation equipment and fire fighting equipment were also designed.

2.3 *Construction plan*

(1) *Excavation method*
(1) Shaft excavation method
Excavation methods used for shaft construction include blasting methods such as the short step method, the long step method, NATM (New Austrian Tunneling Method) and mechanical excavation methods such as the methods using tunnel boring machines (TBM) or raise boring machines. Among the blasting methods, the short step method, in which blasting and lining are repeated at short intervals, has been used in many projects including projects that required excavation in poor rock mass conditions.

The short step method enables excavation at high advance rates. Mechanical excavation methods require the pre-existence of a shaft or tunnel at the target depth. This means that TBM excavation must be preceded by drainage boring and that raise boring requires a shaft or tunnel so that equipment can be brought in advance. For this reason, these methods are not suitable for use in the MIU Project. For the MIU Project, it was decided, in view of the advantages of the short step method, to sink the two shafts (main shaft and ventilation shaft) concurrently by the short step method. In shaft excavation, the percentage of mucking time in total excavation time increases as depth increases. For efficient mucking, therefore, the kibble change method [method in which two kibbles (iron containers) are used] was used. The relationship between the excavation cycle and the advance per shot in blasting operations was studied, and it was decided to use the a modified shot step method, in which a 1.3-meter shot is repeated twice and a 2.6-meter-thick concrete lining is placed, as a safe and most efficient method of excavation.

(2) Gallery excavation method

Excavation methods used for galleries (drifts) can be broadly classified into two types: the blasting method, which is often used for excavation in hard rock, and the mechanical excavation method, which uses road-headers or tunnel boring machines. Since the galleries at the MIU are relatively short, it was decided to use the efficient and economical NATM (blasting-based excavation method in which shotcrete and rock bolts are used for support) except in cases where the mechanical excavation method is used for research purposes such as excavation disturbance testing.

(2) *Surface plant for shaft excavation*

Excavation equipment and auxiliary equipment at the ground surface include derricks, hoists, scaffolding, ventilation equipment, water supply and drainage equipment, concrete plant, power receiving equipment and emergency power generation systems. The equipment that cannot be reinstalled or replaced, such as derricks and hoists, was designed specifically for the depth of 1,000 m, and the equipment that can be added or extended at later stages, such as wastewater treatment plant, was added or extended as shaft and gallery excavation progressed.

(3) *Safety measures*

(1) Measures against unexpected groundwater inflows

One of the phenomena that are likely to be encountered during shaft or gallery excavation is unexpected groundwater inflow. Related geological features that are likely to be encountered during excavation include unconsolidated strata, fault fracture zones and permeable fractures. These geological features could cause an unexpected inflow of a large quantity of groundwater ("unexpected groundwater inflow"). Unexpected groundwater inflow may have a serious impact on the progress of shaft and gallery excavation through such problems as the suspension of excavation or excavation equipment troubles.

In order to devise effective measures against unexpected groundwater inflows, therefore, investigations and studies were conducted with respect to the following items:

- Studying reported cases of unexpected groundwater inflows
- Devising measures against unexpected groundwater inflows in the research shafts and galleries
- Study of methods for evaluating the influence of groundwater control measures on the geological environment.

In sedimentary rock formations, groundwater inflows often occur from unconsolidated strata. Groundwater inflows might be accompanied by sediment flows. Grouting is not likely to be very effective in controlling groundwater inflows.

Groundwater, in some cases under great pressure, may gush out of fracture zones in granite formations. Large-scale fracture zones may be detectable by exploratory boring or other means of pre-excavation investigation. Representative methods for preventing unexpected groundwater inflows are drainage boring (drainage method) and the injection of cement slurry or other grouting materials (grouting method). Although both methods can be used as control measures against unexpected groundwater inflows, the grouting method, which does not cause substantial lowering of groundwater levels, was chosen in order to conserve the geological environment of the research shaft and gallery site. So, problems that need to be addressed in order to use this method during the research shaft and gallery excavation were identified, and concrete measures to be taken were determined.

One major problem is that there is no grouting pump or other grouting tool that can be used under very high pressure (since the groundwater level is close to the ground surface, the hydrostatic pressure at the depth of 1,000 m is 10 MPa). One way to solve this problem is to drill a number of drainage holes in a radial pattern from the bottom of the shaft so that, by reducing groundwater pressure in a limited area, grouting can be used effectively as a means of groundwater control. The grouting plan thus adopted calls for grouting over a horizontal distance of three times the radius of the shaft from the center of the shaft. The grouting depth is the length of the "unexpected groundwater inflow" zone plus the excavation diameter above and below the unexpected groundwater inflow zone. Grout is to be composed of normal Portland cement and a cement-setting accelerator.

(2) Rock burst control measures

Because it is possible that rock burst (bursting with a sharp sound of rock fragments from the surface of a shaft or gallery being excavated) occurs during shaft or gallery excavation, information on past rock burst events, both in Japan and abroad, was collected by literature research to review possible factors contributing to the occurrence of rock burst, such as geological conditions and shaft/gallery layouts. An analysis was also conducted to predict the occurrence of rock burst during the excavation of the shafts and galleries of the MIU. The results of the analysis indicated possibilities of rock burst in the cases where weak rock regions such as fault zones are excavated at a depth of 1,000 m or in the cases where a large-diameter shaft or gallery is bored or a new shaft or gallery is drilled near an existing shaft or galley. If, therefore, excavation is carried out under such a geological environment or in the case of such a shaft or gallery layout, acoustic emission (AE) monitoring, which has a proven track record as a means of rock burst detection, is conducted and measures such as reducing the advance per shot in blasting operations are taken.

(4) *Environmental conservation measures*

As shown in Figure 1, the construction site is close to an urbanized area and to many public facilities. In order to minimize adverse environmental effects, it is necessary to take into consideration the control of the effects of noise and vibration on adjacent facilities during construction, the influence on well water, and the water quality of water released into the river.

With respect to noise and vibration control, studies were conducted to conform to the relevant laws and regulations and find ways to reduce adverse effects in the most cost-effective manner. To be more specific, the locations and noise levels of noise sources were identified, and noise levels at nearby locations were calculated. Then, for specified types of construction tasks related to rock drills, concrete plants and other equipment, specifications for the panels for soundproof buildings on the ground surface were determined so that the standards under the relevant laws and regulations were satisfied.

In the construction phase, noise and vibration measurements in and adjacent to the construction site are conducted periodically to check on the effectiveness of noise control measures.

The influence on river water and well water is evaluated by measuring flow rates and water level. Water released into the river is treated appropriately in a wastewater treatment plant so that the requirements under the relevant laws and regulations are satisfied.

3 CONCLUSION

Foundation work at the shaft entrance locations at the MIU site began in July, 2003. The ground surface bird view of the main shaft construction site is shown in Figure 3 and the excavation work at the bottom of the main shaft is shown in Figure 4. The long term monitoring will be done using adjacent deep boreholes (a borehole length of 1,300 m).

On the basis of the information newly acquired from these investigation and research, the design and the construction plan will be reviewed and modified if necessary. Various data will also be acquired as excavation progresses so as to verify the validity of

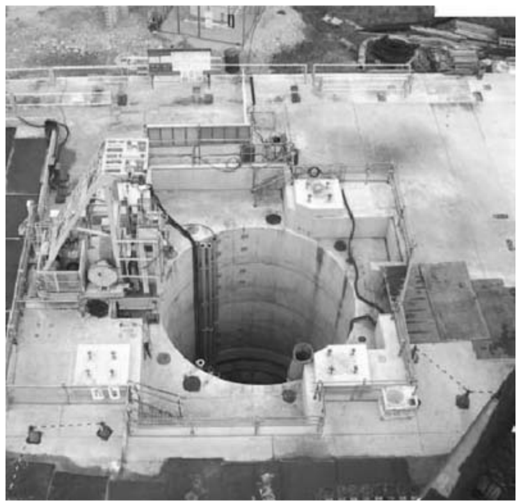

Figure 3. Ground surface bird view of the main shaft construction site of MIU.

Figure 4. Excavation work at the bottom of the main shaft at MIU.

the design and the construction plan and reflect new findings in the subsequent excavation plans for the research shafts and galleries.

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Underground Space Use: Analysis of the Past and Lessons for the Future – Erdem & Solak (eds)

Water-tightening design and construction of tailbay tunnels at Kannagawa Power Plant

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ABSTRACT: The Kannagawa Power Plant, owned by Tokyo Electric Power Company, is presently under construction and is scheduled to start operation in July 2005. The effective water head is 653 m and the maximum output is 2,820 MW. The tailbay tunnel is just downstream of the power plant; thus there might be a significant influence of high water pressure on the cavern due to water leakage from the water tunnels. Therefore, the water-tightening design of tailbay tunnels near a large cavern requires special precautions. In this paper, the authors report on the concept of water-tightness design and the results of grouting of the tailbay tunnel.

1 INTRODUCTION

The Kannagawa Power Plant, owned by Tokyo Electric Power Company, is presently under construction and is scheduled to start operation in July 2005. As shown in Figure 1, the upper dam is located in Minami-Aiki Village in Nagano Prefecture and the lower dam is located in Ueno Village in Gunma Prefecture. The effective water head is 653 m and the maximum output is 2,820 MW.

The tailbay tunnel just downstream of the power plant consists of four water tunnels with inner diameters of 4,100 mm and two water tunnels with inner diameters of 5,800 mm. The former composes the section from the large cavern (power plant) to the first junction and the latter composes the section from the first junction to the second junction. The length of the tailbay tunnel is 190 m and the maximum design internal water pressure is 2.2 MPa.

The overburden is about 500 m and the geology is basically Mesozoic hard rock masses whose matrices are sandstone and mudstone, having a modulus of deformation of 2,500 to 5,500 MPa. The permeability of these rock masses is very low, and the mean and maximum permeability are under 1 Lu and 2 Lu, respectively. And no dominant fault zones were identified.

The large cavern of the power plant has a width of 33 m, a height of 52 m and a length of 216 m. The power generation equipment in this cavern will be installed gradually, according to the power demand in the future. Therefore, the cavern will be left in a partially unequipped state just after excavation.

The design concept for the stability of the cavern is to keep the underground water confined to the bottom of the cavern by the drainage tunnels around it. Consequently, there might be a significant influence of high water pressure on the cavern due to water leakage

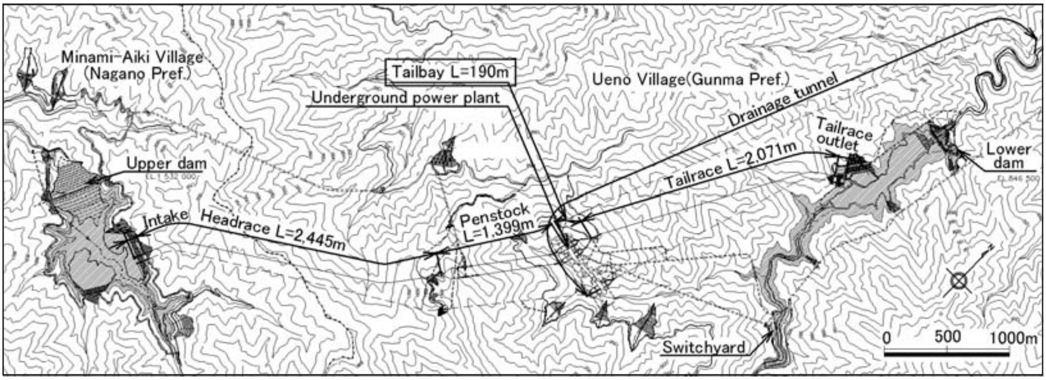


Figure 1. Location map of Kannagawa Power Plant.

from the water tunnels through the loosened rock masses, which may be damaged by blasting. From the permeability tests, the loosened rock masses may have a width of 1 to 2 m around the water tunnels and the maximum permeability is about 100 Lu or more. Therefore, the water-tightening design of the tailbay tunnel near a large cavern requires special precautions.

This paper reports on the water-tightening design and construction of the tailbay tunnel just downstream of the power plant.

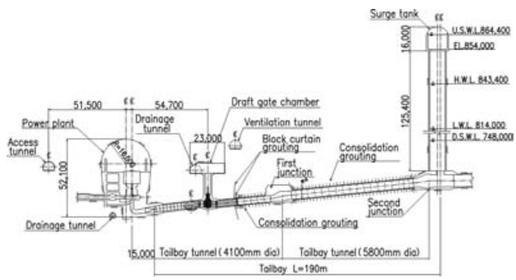


Figure 2. Longitudinal profile of the tailbay tunnels.

2 THE CONCEPT OF WATER-TIGHTENING DESIGN OF TAILBAY TUNNELS

The tailbay tunnels near the large cavern are about 60 m in length and the junction sections are made of steel linings. But for cost savings, the tunnels in other sections are made of reinforced concrete.

In view of several other power plant projects, water leakage from the connections of 4100-mm-diameter steel linings and concrete linings where the seepage paths are particularly short, was regarded as a principal risk, and so the water-tightening design of the tailbay tunnel was carried out accordingly. Figures 2 and 3 show a longitudinal profile and a plan view of the tailbay tunnel, respectively.

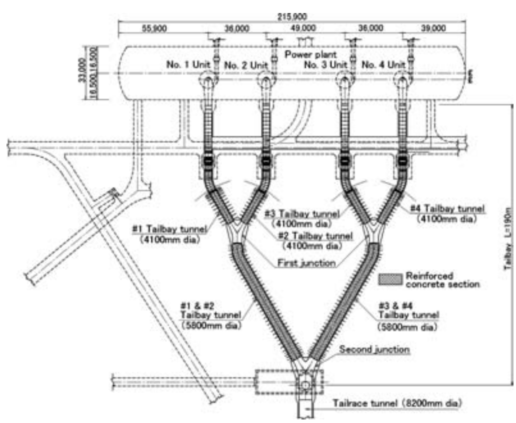


Figure 3. Plan view of the tailbay tunnels.

2.1 *Design of steel lining*

The 60-meter-long section from the draft tube was designed as a steel lining section to ensure water-tightness as shown below:

- Steel lining stress under the internal water pressure (max. 2.2 MPa) while water flow is stopped must be 216 MPa or less.
- Critical buckling pressure under the external water pressure while the steel lining is empty must be 1.5 times the external water pressure (max. 0.8 MPa).

The steel lining is designed in order to maintain stability against either internal water pressure or external water pressure. The internal water pressure is estimated based on a situation in which the four power generation machines are stopped at the same time. The external water pressure is underground water pressure,

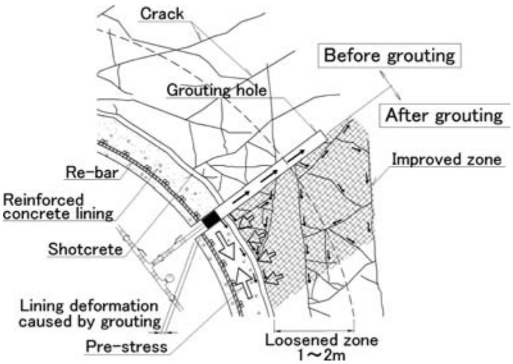


Figure 4. Pre-stressing by grouting.

which is estimated based on the investigation of the water pressure and the infiltration analysis.

2.2 *Design of reinforced concrete lining*

The 4100-mm-diameter tailbay tunnel section between the steel lining and the first junction was designed as a reinforced concrete structure.

In the sections of reinforced concrete, there is a risk that permeability will be higher than that of the steel lining. Therefore, the design concept of the RC tunnels should be a non-cracking concrete lining against the maximum internal water pressure (2.0 MPa), and pre-stressing of the lining by the consolidation grouting is expected.

In the lining design, it was decided to use a large amount of reinforcement ($105\,cm^2/m$) that can be placed at the construction site for the lining thickness of 55 cm, and to use high-strength concrete (design strength: 40 MPa).

2.3 *Design of consolidation grouting*

2.3.1 *Mechanism of pre-stressing*

Figure 4 illustrates pre-stressing accomplished by consolidation grouting. By showing the states of the rock masses and lining concrete before and after consolidation grouting, it is assumed that the mechanism of pre-stressing is as follows:

- In loosened regions, water-tightness is improved by cement milk injected into open joints in the rock masses and bedding planes. When the cement milk infiltrates, the concrete lining is moved toward the inside of the tunnel and becomes pre-stressed.
- In low permeability regions outside the loosened regions, however, penetration of cement milk required for pre-stressing cannot be expected. So, mainly concrete-rock or concrete-lining interfaces were considered as areas to be grouted.

2.3.2 *Pre-stress to prevent cracking*

The amount of tensile stress that occurs in the lining under the maximum internal water pressure (2.0 MPa) is calculated by means of the method of Otto Frey-Baer. As a result, the tensile stress of concrete is calculated at 7.0 MPa, which is larger than the tensile strength of the concrete (3.0 MPa). It means that cracks occur when the lining is subjected to the maximum internal water pressure. Therefore, the concrete has to be pre-stressed up to 4.0 MPa (=7.0 MPa − 3.0 MPa).

In this case, if the pre-stress introduction ratio is assumed to be 30%, in order to introduce the target pre-stress, the required grouting pressure is 2.5 MPa (determined by the thick-cylindrical shell theory). In order to ensure successful grouting, however, a maximum grouting pressure of 2.8 MPa has been adopted.

2.3.3 *Improvement of water-tightness*

The loosened region around tailbay tunnel #1 extends over a distance of 2 m into the surrounding rock masses. In the loosened region, there are areas where permeability exceeds 10 Lu, and there are even extremely permeable areas where more than 100 Lu was recorded. The thickness of the loosened regions in the rock masses around tailbay tunnels #2, #3 and #4 are about 1 m. The permeability of the surrounding rock masses exceeds 10 Lu locally, but never exceeds 100 Lu.

In view of the distribution of permeability values and cracks around the tunnels, it was decided that the length of the grouting hole should be 3 m into the rock masses for tunnel #1, where the loosened region was about 2 m deep; and 2 m into the rock masses for tunnels #2, #3 and #4, where the loosened region was about 1 m deep.

Anticipating a failure to achieve the required pre-stress, it was decided that a mean permeability of 1 Lu should be adopted as the target level so that piping would not occur under the maximum internal water pressure.

3 CONSOLIDATION GROUTING

3.1 *Consolidation grouting of tailbay tunnel*

The results of the consolidation grouting of the headrace tunnel in Kannagawa Power Plant confirmed that the primary grouting sufficiently lowers the permeability of the rock masses. Considering this fact, the basic policy of the consolidation grouting is set up as follows:

- The permeability of rock masses around the tunnel is sufficiently lowered only by the primary grouting.
- After secondary grouting, priority is given to pre-stressing the concrete lining; the main point is to introduce pre-stresses appropriately.

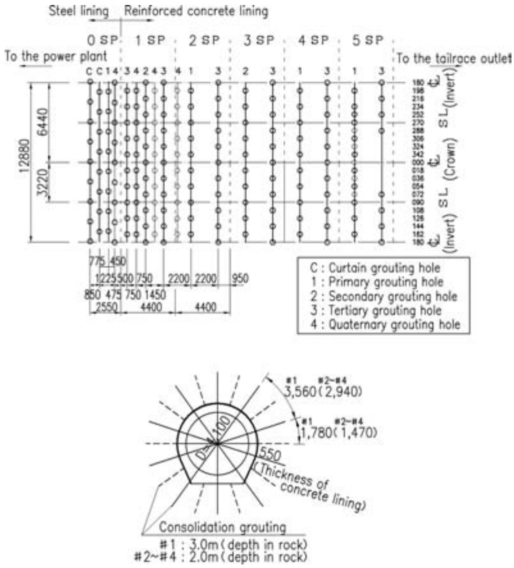


Figure 5. Consolidation grouting.

Based on this basic policy, the consolidation grouting of the 4100-mm-diameter tailbay tunnel was planned.

Figures 5 and 6 show the grouting design and grouting specifications for the 4100-mm-diameter tailbay tunnel. The grouting specifications are reviewed and modified from time to time according to the degree of improvement and the amount of pre-stress being introduced.

(1) Grouting pressure

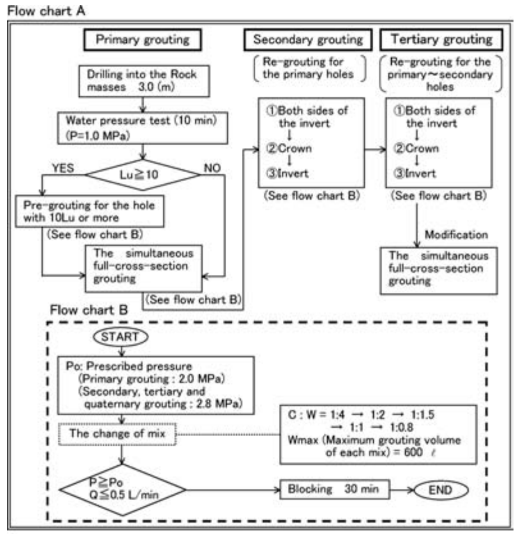
In the primary grouting, the improvement of the water tightness was the main purpose. Thus, in order to be easily pre-stressed by secondary and subsequent grouting, it was decided that the grouting pressure should be 2.0 MPa, which is equivalent to the maximum design internal water pressure.

On the other hand, the main purposes of the secondary and subsequent grouting are to introduce pre-stresses appropriately. Thus, in order to ensure successful grouting, however, a maximum pressure of 2.8 MPa has been adopted (see 2.3.2).

(2) Grouting method

All primary holes were grouted simultaneously by the full-cross-section grouting method in order to minimize pressure decreases during the cement milk infiltration process.

The grouting sequences for the secondary and subsequent grouting holes were determined according to the pre-stresses introduced by the primary hole grouting. If the pre-stress level achieved was not enough, priority was given to the pre-stressing of the invert, which had been considered to be the most difficult to pre-stress, considering the excavated shape of the tunnel.



		Length of the grout hole L (m) #1	Length of the grout hole L (m) #2~#4	Pressure P_0 (MPa)	Initial Composition C : W	Cement
STEP1	Primary	3.0	2.0	2.0	1:4	Slag cement (typeB) : Lu≧10
		3.0	2.0	2.0	1:2	Ultrafine(A) : Lu<10
STEP2	Secondary	3.0	2.0	2.8	1:2	Ultrafine(A)
	Re-grouting	0.5	0.5	2.8	1:2	
STEP3	Tertiary	3.0	2.0	2.8	1:2	Ultrafine(A)
	Re-grouting	0.5	0.5	2.8	1:2	
STEP4	Quaternary	3.0	2.0	2.8	1:1.5	Ultrafine(B)
	Re-grouting	0.5	0.5	2.8	1:1.5	

Figure 6. Grouting flow chart.

In order to minimize flexural tensile stress in the invert lining concrete, grouting was carried out in this order: (a) the walls on both sides of the invert, (b) the crown, and (c) the invert.

In the actual grouting process, it was confirmed that the pre-stress introduced was lost, especially, in places away from the grouting holes when, for example, the flow rate decreased during pre-blocking grouting (while the specified level of pressure was maintained).

We can assume that the loss of pre-stress was caused by the decrease of pressure in the region near the front end of the penetrating cement milk, and partly because the cement milk was diluted by the progress of cement milk penetration.

In view of these results, the simultaneous full-cross-section grouting method was reviewed and modified. Furthermore, the grouted holes were also drilled again (0.5 m into the rock masses) and additional grouting was carried out.

(3) Grouting material and initial composition

The grouting material used for the primary grouting holes with a permeability of 10 Lu or more was slag cement (type B), and the initial composition was C:W = 1:4.

For the primary grouting holes with a permeability of lower than 10 Lu and for the secondary and subsequent grouting holes, ultrafine cement was used to minimize the dilution of the grouting during penetration. Ultrafine cement made by A corporation (Blaine fineness: 9,300 cm^2/g; 50% grain size: 3.5 μm), which had also been used at several other power plant projects, was used. For the quaternary holes, ultrafine cement made by B Corporation (Blaine fineness: 13,000 cm^2/g; 50% grain size: 2.5 μm) was used, because a higher Blaine fineness value was desired. The latter cement cost is almost the same as that of the former one.

In consideration of viscosity and bleeding, an initial composition of C:W = 1:2 was adopted as shown in Figure 6. However, in the actual grouting process, it was difficult to pre-stress at this initial composition. Thus, in order to reduce the dilution, the initial composition was changed to C:W = 1:1.5.

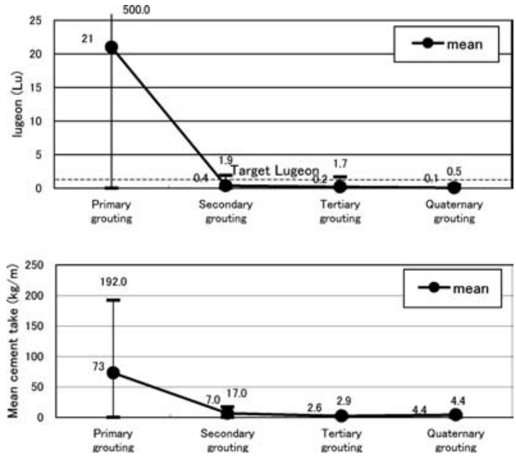


Figure 7. Lugeon and cement take vs. grouting stages.

3.2 *Improvement of water-tightness*

Figure 7 shows the quantities of cement used in grouting holes of different stages. As shown, the pre-grouting mean permeability and maximum perme-ability during primary grouting were 21 Lu and 500 Lu, respectively, while the pre-grouting mean permeability and maximum permeability at the secondary holes were 0.4 Lu and 1.9 Lu. These results clearly indicate the effects of the simultaneous full-cross-section grouting and the use of ultrafine cement for the primary grouting.

The fact that the pre-grouting permeability of the quaternary holes never reached 1 Lu (max. 0.5 Lu) indicates that a sufficient level of water-tightness was achieved by the grouting of tertiary or earlier holes.

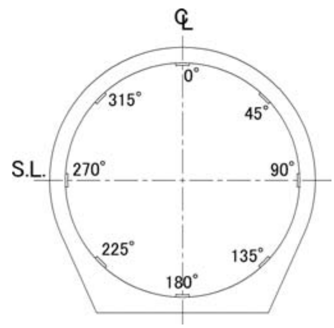


Figure 8. Locations of strain gauges.

3.3 *Pre-stresses*

In order to check the amounts of pre-stress actually introduced, pre-stress measurements were conducted by using strain gauges on the inside surface of the lining concrete, as shown in Figure 8.

Figure 9 is an example of the measurement results, and shows the surface strain time history and the distribution of strains in the cross sections. The target pre-stresses have been largely achieved mainly on both sides of the invert, but the target levels have not been attained at other locations.

Figure 10 also shows that the pre-stresses introduced were achieved mostly by primary grouting.

3.4 *Comparison with the grouting in the weak rock masses at the headrace tunnel of Kannagawa Power Plant*

Grouting was also performed in the weak rock masses at the headrace tunnel of Kannagawa Power Plant for the purpose of pre-stressing. As a result, the required level of pre-stress was achieved.

The geology of the headrace tunnel is composed mainly of serpentinite and mudstone. These rock masses are heavily weathered, and their modulus of deformation is 38 to 780 MPa, which is extremely low compared with the values of the tailbay tunnels (2500 to 5500 MPa). And permeability exceeds 10 Lu locally.

As shown in Figure 11, relatively large quantities of cement milk were injected into the weak rock masses at every grouting stage and the effects of grouting are obvious. Therefore, the quantity of cement used seems to greatly affect the amount of pre-stress introduced.

Differences in the quantity of cement injected are likely due to the composition of rock masses. Since there are partially argillized regions along cracks in the weak rock masses, relatively large quantities of cement milk can be injected. Hence, the weak rock masses are located within a relatively small area, and

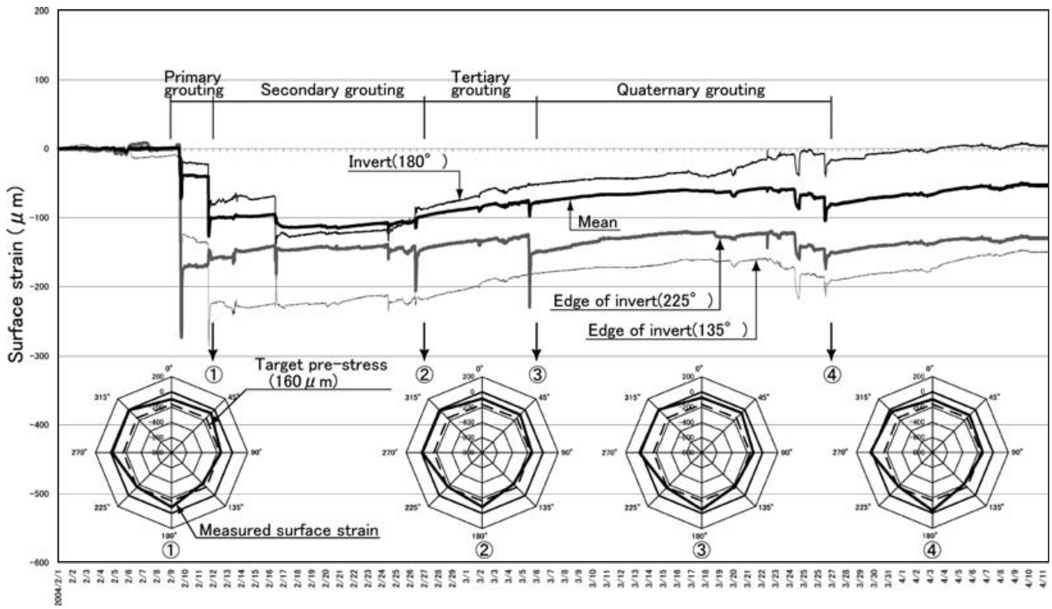


Figure 9. Strains vs. time.

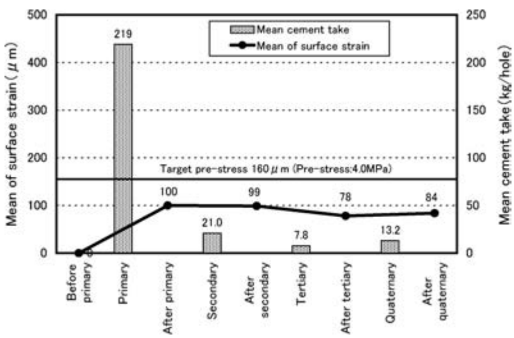


Figure 10. Strains and cement take vs. grouting stages (Tailbay tunnel).

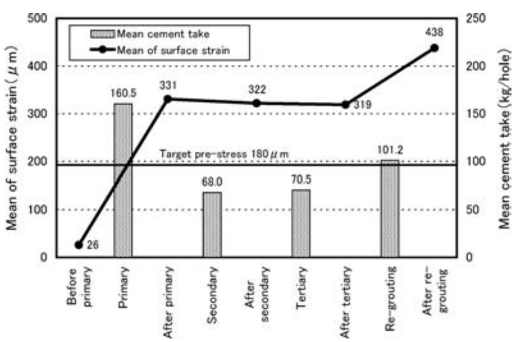


Figure 11. Strains and cement take vs. grouting stages (Headrace tunnel).

it is assumed that the surrounding hard rock masses provided enough reaction for pre-stressing.

On the other hand, at the tailbay tunnel in extremely hard rock masses, only local open cracks were grouted and, therefore, no other region could be grouted.

Considering that in cases where extremely hard rock masses are to be grouted (as in the case of the grouting carried out at the tailbay tunnel), it may be necessary, for example, to provide voids behind the concrete lining prior to grouting.

4 CONCLUSION

In the 4100-mm-diameter reinforced concrete tailbay tunnel, it was expected that pre-stressing of the lining (i.e. non-cracking concrete lining) could be achieved by consolidation grouting. But, in general, the target pre-stress was not attained.

Meanwhile, it has been confirmed that the permeability of the rock masses around the tunnel was sufficiently lowered. Therefore, there was no significant influence of water leakage from the water tunnels on the cavern of the power plant.

The grouting in the tailbay tunnel section has already been completed, and the filling of the water tunnels began at the end of July 2004. The present water pressure is 1.2 MPa, so the design internal water pressure (2.0 MPa) has not yet been reached. In addition, water leakage into the adjacent drainage tunnels has not been observed. Monitoring of tailbay tunnel behavior will be continued.

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Study on state and pattern analysis of tunnel portal slope in Korea

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ABSTRACT: The number of tunnels are in fact increasing as a part of linear improvement project of general national highway and road enlargement and pavement project. Recently, collapses of portal slope are also occurring considerably, due to local raining from severe rain storm and abnormal weather. Accordingly, it was risen a necessity to efficiently respond to tunnel portal slope damage and maintenance in Korea and oversea nations. This paper is a basic proposal to execute a survey on the current status and state of the tunnel portal slopes that were already installed and are now being operated along general national highways, and also to execute state evaluation for the purpose of managing those effectively. As a research method, domestic tunnels were analyzed in accordance with geometrical shape such as access type, portal form, and tunnel type, etc. via field survey to analyze the types of tunnel portal slopes along national highways. State evaluation classification sheet is presented to divide classes for the danger state of the surveyed portal slopes, and then the related grades are divided. It is mainly aimed at classifying the tunnel portal slopes along national highways with using this state evaluation, to use it as basic data so that continuous maintenance can be executed in the future in accordance with danger classes.

1 INTRODUCTION

In Korea, it is inevitable for tunnel to be generated following the construction of roads because more than 70% of land is mountainous area in view of topography features. It is actually tremendous national loss, i.e. cut-off of traffic and loss of lives due to collapses of Tunnel portal slope and entrance/exit slopes every year. The total number of tunnels, which are managed by nationwide 18 national highway maintenance & construction offices, are 105 tunnels, and the dispersion status of 5 regional construction management offices is as shown in Table 1. It was surveyed that there were many number of tunnels in Seoul Regional Office, Busan Regional Office, and Daejon Regional Office, and the average extension was resolutely the longest in case of Daejon Regional Office.

This research is aimed at: analyzing the overall status based on the data collected from direct surveying and observation on national highway's tunnel portal slope; evaluating the construction and stability of portal slope; and then using it as basic data for the arrangement of future maintenance measures and the related researches.

Table 1. Tunnel dispersion status along the national highways in Korea.

Regional Offices	Total	Seoul	Wonju	Daejon	Igsan	Busan
No. of tunnel	105	28	20	23	20	24
Average length	608	458	653	729	607	568

2 ANALYSIS ON THE CURRENT STATUS OF TUNNEL PORTAL SLOPE

The data acquired from field survey was used for the analysis on the current status of tunnel portal slope along national highways and its state evaluation. The fields that were utilized for this data analysis are as shown in Table 2. As shown in Table 2, the current status was analyzed dividing it into tunnel name, the number of tunnel portals, and the number of portal slopes. Those differences are due to such tunnels of national highway as being divided into up-stream and downstream, coming and going lane tunnels, and tunnels without portal slope. The number of portal slopes refer to the numbers in the field surveys, which were used in field survey of portal slope, and it were divided into as followings when we watch the portal in front of it; "front slope" that is located upper part of entrance of tunnel; "left slope" that is located at the left side connected to entrance of tunnel; and "right slope" that is located at the right side. For the respect tunnel entrance of tunnels, there are cases that all three of left, front, right slopes exist, while cases that are different,

Table 2. The numbers of tunnels and portal slopes per management offices.

Regional construction management offices	Highway maintenance & construction offices	Number of tunnel name	Number of portals	Number of tunnel sites
Seoul	Suwon	2	6	4
	Uijeongbu	7	17	14
Wonju	Hongcheon	12	26	16
	Gangneung	–	–	–
	Jeongseon	4	8	4
Daejon	Nonsan	5	10	7
	Chungju	7	17	14
	Boeun	–	–	–
	Yesan	1	1	2
Igsan	Gwangju	6	15	9
	Jeonju	1	2	1
	Namwon	4	8	5
	Suncheon	3	8	5
Busan	Daegu	–	–	–
	Jinju	2	4	4
	Pohang	4	8	6
	Yeongju	7	14	10
	Jinyeong	3	6	4
Total		68	151	105

Table 3. Number of tunnel type and tunnel length.

Tunnel type	No. of tunnel	No. of portal	Division	Length (m)	Remarks
Single lane parallel tunnel	37	89	Minimum	150	Gwangju office (Chuwolsan tunnel)
Double lane tunnel	30	60	Maximum	1,960	Chungju office (Bakdaljae tunnel)
Single lane tunnel	1	2	Average	608	

due to the topographical causes such as access type, etc. mentioned earlier. To see it by regional office, the number of portals of Seoul Regional Office are 23, which shows the least dispersion, and the numbers of portals of Wonju Regional Office, Iksan Regional Office, and Busan Regional Office are similar each other.

For the analysis on tunnel portal slope, the respect tunnel field was divided into entrance and exit. Also, to analyze the portal in detail, it was divided into the location of portal, single tunnel, and double lane tunnel. The reason why is that can be used in maintenance system in the future, which will be applied in view of maintenance. Table 3 shows the result of analysis on the number of tunnels per tunnel type and tunnel extension. From Table 3, “single lane parallel tunnel” refers to a couple of tunnels of which portal’s locations are the same and up/down streams are divided into two tunnels, while “double lane tunnel” refers to a tunnel of which up/down streams are the same. Also, “single lane tunnel” refers to a tunnel that just one lane among up and down streams is made by tunnel. As a result of analysis, it shows that single lane parallel tunnels occupy most of tunnels in Korea. In case of single lane tunnel, there is Maamsan tunnel in the jurisdiction of Jinyeong office, of which down stream lane is made by tunnel, hence it is only one tunnel of the case. For tunnel length, it was surveyed that Chuwolsan tunnel in the jurisdiction of Gwangju office is the shortest while the longest one is Bakdaljae tunnel within the jurisdiction of Chungju office.

The location and types of tunnel portals were analyzed in view of maintenance such as future maintenance and introduction of real-time monitoring system. Table 4 is the results of analysis on access types and portal form. First, analysis on access types will be presented and then the analysis on portal form will be described.

Generally, the access type of entrance of tunnel is influenced the most from topography or weather, which may be divided into total of 5 types depending on the relation between topography and tunnel central axis: “slope orthogonal type”, the best idealistic location