

Excavating in weak rocks with the Norwegian Method of Tunnelling (NMT)

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ABSTRACT: The updated Q-system for rock mass classification and support selection and the use of modern materials such as wet process fiber reinforced shotcrete, S(fr), anticorrosive bolts, and reinforced ribs of shotcrete, RRS, are essential elements of the Norwegian Method of Tunnelling (NMT). The Norwegian Geotechnical Institute of Oslo has been involved in a joint venture with Contractor C.J.Sarantopoulos S.A and Consulting Group Axon Ltd for the study and construction of the first tunnel project in Patras, Greece, using principles from the Norwegian Method of Tunnelling (NMT). This project comprises the construction of a bypass highway, with twin tunnels, scheduled to open by year 2001, in weak marl formation with sandy interbeddings. The twin tunnels with an approximate length of 650 m have a designed pillar thickness of 16 m. The Q - system was used for the classification of the rock mass which could be characterised as extremely poor to very poor with Q - values ranging between 0.01 and 0.3.

RESUME: Le système Q de classification des masses rocheuses et de sélection de la méthode de soutènement, ainsi que l'utilisation de matériaux modernes tels que le béton projeté renforcé avec des fibres d'acier, S(fr), le boulonnage anticorrosion, le cintres en gunite renforcés, RRS, sont des éléments essentiels de la méthode norvégienne de construction des tunnels (NMT). L'Institut de Géotechnique Norvégien d'Oslo a été impliqué dans un projet multilatéral avec le maître d'ouvrage C.J. Sarantopoulos S.A. et le bureau d'études Axon Ltd pour l'étude et la construction d'un premier projet de tunnels à Patras, Grèce, basé sur les principes de la méthode NMT. Le projet comprend la construction d'une voie d'autoroute avec deux tunnels jumelés, dont l'ouverture est prévue pour 2001, dans des formations marneuses peu résistantes avec des bancs sablonneux. Les deux tunnels, d'une longueur approximative de 650 mètres, ont une épaisseur de pilier de 16 mètres. Le système Q a été utilisé pour la classification de la masse marneuse, qui peut être caractérisé comme extrêmement faible à très faible, les valeurs Q variant entre 0.01 et 0.3.

1 INTRODUCTION

Due to uncertainties in connection with the ground conditions revealed in core logs, a pilot tunnel 40 m in length, in a nearby location of the twin tunnels with nearly quadratic cross section with dimensions 2 x 2 m, was first excavated. Several in situ tests such as plate loading tests for determination of the E modulus, deformation measurements for the elastic response of the rock mass and bolt pull out tests for the determination of shear strength of the material were performed. All these in situ tests provided valuable information for the overall evaluation of the support requirements. Two plate loading tests were conducted according to the suggested international standards (ISRM, 1981). Stress levels above 2.5 MPa resulted in plastic behaviour of the rock.

Several extensometers were installed in three different locations in the test tunnel. The roof extensometers of Section C at 35.0 m from the entrance reached values of about 13 and 11 mm at 1.5m and 2.5m from the arch crown respectively, before they were stabilised. Surprisingly good results were derived from the bolt pull-out tests on site. The five tested fully grouted bolts of effective grouting length of only 1.25m were able to take loads ranging between 7.9 and 17.2 tnf. Failure on the pull-out tests occurred between grouting and rock. The maximum shear stress during the bolt pull-out tests can then be calculated by using the simple relation:

$$\tau_{\max} = \frac{F}{\pi \times D \times L}$$

where: F = maximum pull-out load, D = hole diameter, 38 mm, L = grouted length, 1.25 m.

The maximum obtained shear strength values τ_{max} varied between 0.6 to 1.1 MPa. If we consider these bolt pull-out tests to be equivalent to undrained triaxial tests with the eventual failure envelope horizontal, then we can indirectly estimate the uniaxial compressive strength, σ_c , of the marl which is twice as much as τ_{max} . From the 1981 ISRM handbook (Suggested methods for the quantitative description of discontinuities) the Patras marl and conglomerate formation can be classified as weak rock.

R0 - R1 Weak rock crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife $\sigma_c=0.25-5.0$ MPa

The above bolt pull-out calculation gives values of σ_c ranging between 1.2 and 2.3 MPa, which are consistent with the above index classification.

Four simple numerical models using the discontinuous code UDEC were performed first to simulate (back calculate) the behaviour of the excavated pilot tunnel. These models were run with different parameters (variation mainly in the deformation modulus E). This was done in an attempt to model as closely as possible the monitored results. All models were first consolidated (run to equilibrium) with an assumed in situ $\sigma_H/\sigma_V = K_0$ ratio of 0.5. The 2 x 2m pilot tunnel was then numerically excavated in one single step and the model run to equilibrium. The numerical run with E = 0.1 GPa and Poisson's ratio of 0.33 and elastic approach gave the best fit to the in situ results.

2 ENGINEERING GEOLOGICAL DESCRIPTION

Field inspection and mapping of the test tunnel, a geological map and longitudinal profile along the tunnels, photographs of drill cores and some mineralogical analyses have been the basis for geological description. The site consists of a hilly landscape with ridges more than 50m in height. The slopes are generally steep; small sections of them may have slope angles of 60-70°, locally with 80-90° cliffs of a few meters in height. There is usually a cover of vegetation, but on very steep slopes outcrops are seen. The rock could also be studied in the 40m long test tunnel. The area consists of young, soft, sedimentary rocks. Two rock types are found: conglomerate and marl.

The marl has a bedded structure with alternating sand and marl beds. In the test tunnel the bedding has a dip angle of about 10°. The sand layers are usually a few centimetres thick and consist mainly of quartz sand. There is no cementation, and the sand can easily be dug out by the fingers. The marl layers

are usually thicker than the sand layers, often about 20 cm. According to mineralogical analyses the marl layers consist of about 50% calcite, 20% quartz, 20% illite and a few per cent of chlorite, montmorillonite and feldspar. The marl is plastic and can easily be cut by a knife. In these marl layers there occur some harder, siliceous layers, 1-3 cm in thickness. On the slope outside the test tunnel and in the outermost 15m of this tunnel some joints can be seen in the marl. The most prominent joints are parallel to the slope with a dip angle of about 80° and a strike direction of 90° to the tunnel axis. The joints are rather planar with 0.5-1m spacing in the slope and the outermost metres of the tunnel. The joint spacing increases inwards along the tunnel, and about 15m inside the tunnel no more joints are seen. Some of the joints have apertures of 1-2 mm with rust coating. In the outermost metres of the tunnel a joint with a filling of plant roots is seen. In between these joints some short, vertical, irregular fractures striking about parallel to the tunnel axis are seen. These fractures may be open (1-2 mm), but usually have no filling or coating. Laboratory testing of the marl shows a compressive strength ranging from 0.3 - 2.0 MPa.

3 ROCK MASS CLASSIFICATION, Q-SYSTEM, SUPPORT

The Q -system is based on 6 different parameters, and the Q-value is calculated by means of the following formula:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where

RQD = rock quality designation J_n = joint set number
 J_r = joint roughness number J_a = joint alteration number
 J_w = joint water reduction factor SRF = stress reduction factor

The marl and sand layers have some sub-vertical jointing perpendicular to the tunnel face and occasional sub-vertical joints parallel to the tunnel axis. Both the sand ($\sigma_c \approx 0.1$ MPa) and the marl ($\sigma_c \approx 0.5-1$ MPa) are in the class extremely weak rock (ISRM-class R1, R0, or worse). Whether or not core is recovered in intact pieces longer than 10 cm, it would be given RQD = 0 which for Q-value calculation gives a nominal minimum value of RQD = 10. The number of joint sets is first given the optimistic value of 6 (2 sets + random) for the case of more competent marl layers in the arch, up to the extreme (but realistic value of 20 ("earthlike") for the present fragile state of the sand(stone). (It is difficult to pick pieces from the face without them

crumbling to sand in a brittle manner, *i.e.*, losing all cohesion). Since the marl is a hard, clay-like material, with layers of incompetent sand, the J_r was given the nominal value of 1.0, and for the "filling" of sand, J_a was set to 5. The marl layers are considered to be relatively tight whereas the sand layers are obviously more permeable, but since the tunnel is above the groundwater table, no water problems were expected, and J_w was stipulated as 1. The stress conditions depends on the overburden. In sections with low overburden there are low stresses, *i.e.*, $SRF = 1-2.5$. With higher overburden, mild squeezing could be encountered and $SRF = 5-10$ was estimated. (Monitoring confirmed the applied SRF values).

$$Q = \frac{10}{6-20} \times \frac{1}{5} \times \frac{1}{1-10} = 0.01(\text{minimum}) - 0.33(\text{maximum})$$

The weighted mean Q - value was estimate to be 0.1. Based on the above Q-values an estimation of the support requirements was carried out (Barton et. al. 1980). The twin tunnels were planned with a span width of 12m, and the ESR-value (safety level) was set to 1. This means support category 8 for a Q-value of 0.1 (marl, mean). In the case of the deepest overburden, it will be correct to assume the lowest Q-value for the marl, *i.e.*, 0.01. The minimum value of Q 0.01 was also used when persistent layers of sand, several meters thick and long were encountered during the excavation of the top heading. In the case of both the shallow parts of the tunnel and the deeper parts of the tunnel in the marl, the implied support was rib-reinforced shotcrete (fibre and steel bar reinforced) and fully grouted bolts of suitable length for the soft conditions in the marl. For this tunnel, bolts with a length of 4m and diameter of 25mm were recommended.

4 REINFORCED RIBS OF SHOTCRETE

The Q-classification indicates support category 8, *i.e.*, reinforced ribs of shotcrete + bolting as support for the road tunnels. An empirical equation relating permanent support pressure and the Q-value (Barton et. al. 1974), (Grimstad and Barton, 1993) is as follows:

$$P_{\text{roof}} = \left(\frac{2.0}{J_r} \right) Q^{-1/3}$$

where: P_{roof} = permanent roof support pressure in kg/cm^2 , J_r = joint roughness number. From this equation, the following support pressures are found for the marl in the table below:

Table 1. Support pressures for the marl derived from Q-system

Quality	Q-value	Proof		Support
		T/m2	tons/bolt	
Mean	0.1	ca. 50	84.5	Bolting c-c 1.3m + 15 cm of S(fr) + RRS (15cm thick)
Worst	0.01	ca. 90	90.0	Bolting c-c 1.0m + 15 cm of S(fr) + RRS (30 cm thick)

The number in the fourth column refers to the bolt load if the bolt could take the whole estimated load P_{roof} . The likely loading of the composite structure was checked later by numerical models. This RRS-type of support (see Figure 1) will be necessary when the first layer of a certain thickness of fibre reinforced shotcrete and bolting are insufficient for bearing the load. Therefore we can see that the shotcrete must take a considerable part of the load, and therefore the best way to construct the support will be by using fiber reinforced shotcrete S(fr), RRS (reinforced ribs of shotcrete) + bolting, prior to final nominal concrete lining.

Another case to consider is if the shape of the excavated opening is very irregular, and a more circular shape has to be built up in order to support the rock. The RRS or reinforced ribs of shotcrete represent an extremely flexible method in which the thickness and spacing of the ribs and also the number and thickness of the steel bars can be varied according to needs. Figure 1 shows a cross-section and longitudinal section through RRS arches. For lower Q-values a larger number of larger diameter bars can be used, the spacing between the anchoring bolts and between the ribs can also be smaller. The usual way to construct such ribs is first to use about 10-15 cm of fibre reinforced shotcrete as a general support. Unevenness in the profile can eventually be filled with shotcrete without fibre. The ribs are then constructed with additional shotcrete. As a first estimate for the Patras tunnel the ribs were given a total thickness varying from 30 cm (minimum) to 60 cm (with overbreak), consisting of 4-6 steel bars, 16 or 20 mm in diameter. The distance between the ribs varied between 1 to 2m. The use of spiling ahead of the face and monitoring of closures was recommended in these poor quality rock masses which typically have Q-values in the range 0.01 to 0.3. The contrast in early ground control when using bolted reinforced shotcrete ribs instead of regular shaped steel sets and blocking is fairly clear, and the total thickness of concrete is potentially reduced.

5 VERIFICATION OF THE EMPIRICAL DESIGN

Dimensioning of the support in the cases of hard rock types can usually be carried out directly based on the empirical knowledge incorporated in the Q-system. However, the rock is weaker than general

compared to most Q-system case records, and reinforced shotcrete ribs are a rather new method of supporting tunnels outside Norway. The empirical data base is therefore restricted for this type of support in weak rocks, and in each case calculations should be made to find the right dimensions.

A question arises when considering final tunnel support needs of whether an invert arch will be used or not. It was wise to assume that an invert arch would be needed in the final support since it is well accepted that invert arches add significantly to the stability of tunnels in weak rocks. Since the fibre reinforced shotcrete S(fr) has low permeability [the permeability of the S(fr) may be as low as 10^{-11} to 10^{-13} m/s], carefully spaced drainage channels should be used in the primary tunnel support. The ground water table is well below the lower part of the tunnel which minimises the water sealing requirements. However, during the rainy season, occasional raised water tables and water leakage were observed. Use of MPBX and convergence monitoring in the main tunnel was suggested during construction. This will ensure, at a low cost, that the Q-system prognoses are calibrated (and possibly adjusted) for selecting the final support.

Due to the existing regulations for the traffic tunnels in Greece, a nominal concrete lining of 40 cm will be used, which is not specified by the Q-system. Another question that needed to be answered is how close (pillar width) the twin tunnels could be before we get significant interaction and potential stability problems. The Q-system is not really able to answer these questions, and additional numerical studies were performed to evaluate the potential behaviour of the twin-tunnels. In addition to this we also had to note that the real twin two-lane tunnels were excavated almost 18m below the pilot tunnel. It was believed and proved in practice that even at that depth, no squeezing conditions occurred. We were also uncertain about the optimum excavation sequence that had to be followed. This had to be checked numerically and of course was checked by monitoring.

6 NUMERICAL MODELLING DESCRIPTION

The Distinct Element method (DEM) used at NGI for the analyses is a two dimensional code called UDEC (Universal Distinct Element Code). This code (Ref. Manual, November 1995) which can be installed on an IBM-compatible 486 or Pentium personal computer running DOS, has been used extensively at NGI on different projects (Barton et. al. 1992), (Chryssanthakis and Barton, 1992). NGI's version of UDEC 2.02 which is called UDEC-BB (BB refers to the Barton-Bandis joint model) contains also a S(fr) subroutine which allows for modelling the interaction of grouted bolts and of the

fibre reinforced shotcrete, S(fr), with the jointed rock mass, in this case marl interbedded with sandy layers. The input data for the fibre reinforced shotcrete are shown in table 2. The main characteristics of this S(fr) subroutine are as follows:

1. Possibility to apply S(fr) not only on idealised (circular) tunnel peripheries but also in uneven peripheries.
2. Possibility to model the variation in adhesion between the S(fr) and rock interface (e.g difference in marl and gneiss).
3. Possibility to model the ductile behaviour of S(fr) after failure.
4. Possibility to model the bolt reinforcement piercing the S(fr).

Table 2. Fiber reinforced shotcrete parameters used in the modelling work.

Parameter	Notation, units	All models
Modulus of elasticity	E(GPa)	15
Poisson's ratio	ν	0.15
Density	ρ (kg/m ³)	2.5e3
Compressive yield strength	Ycomp (MPa)	30
Tensile yield strength	Yield (MPa)	3
Residual tensile yield strength	Yresid (MPa)	2
Friction in S(fr)/rock interface	Jfric (degrees)	40
Cohesion in S(fr)/rock interface	Jcoh (MPa)	0.25
Tension (bond) in S(fr)/rock interface	Jtens (MPa)	0.43

A total of nine different numerical models of the twin tunnels were run with similar geometry, using a small range of feasible properties. The modelling has taken into account the numerous inter-marl sand layers deposits of low friction, low cohesion, which in practice had a width between 10 and 20 cm. The spacing of these sand layers generally varied between 0.5 and 1.0 m. The spacing of these sand layers is probably higher in reality. A rough estimate of 10 degrees for the dip has been given to the sand layers. This dip angle has been modelled in such a way that it is always unfavourable to the tunnel stability (see Figure 2). No data on the in situ stresses were available on this project. A $\sigma_H/\sigma_V = K_0$ ratio of 0.5 which might be rather conservative has been assumed for all numerical runs. The fiber reinforced shotcrete thickness for the models was varied between 0.15m and 0.25m. The 0.25 m S(fr) corresponds to equivalent, continuous S(fr) thickness, which means that the thicker (25 - 50 cm) RRS (reinforced ribs of shotcrete) had been taken into account. The bolt pattern of 1.3 x 1.3m, corresponding to bolt spacing of 1.3 m was derived from the Q-system (mean case), with 4m long bolts, 20 mm in diameter, at 1.3m spacing has been applied in all the models below.

Four models of the twin road tunnel were first run using the elastic approach for the behaviour of the

intact rock mass. An initial pillar width of 11 m was the same in these four elastic models 1,2,3 and 4 and the parameter variation used was as follows:

Invert arch or flat bottom

1. Variation in the overburden (20 and 40 m)

2. Variation in the deformation modulus ($E=0.3$ GPa and $E=0.1$ GPa.)

3. Variation in the S(fr) thickness (15 and 25 cm).

Four extra models of the twin road tunnel were run using the elasto - plastic constitutive model according to the Mohr - Coulomb failure criterion for the behaviour of the intact rock mass (Hoek and Brown 1980). The overburden of 40 m, deformation modulus of $E = 0.1$ GPa, internal friction angle $\phi = 22$ degrees and cohesion $c = 40$ KPa, thickness of S(fr) of 25 cm and the use of invert arch were the same in the elasto - plastic models 5,6,7 and 8. The parameter variation was as following:

1. Pillar width (11m or 16 m).

2. Variation in the excavation sequence (2 or 3 phase excavation).

3. Final lining 40 cm in thickness or not.

6.1 Excavation sequence

The upper part of the left tunnel designated by 1 in Figure 2 was excavated first. When approximately 50% of the expected total deformation, in practice this was between 100 and 120 mm, (the total defined as just before the initiation of the collapse of the upper part occurred) the S(fr) was applied to be followed by the application of the bolts at about 70% of total expected deformation. The model was then run to near equilibrium (no major deformation movement was recorded). When the near equilibrium was achieved the lower part of the left tunnel bench designated by 2 was excavated to be followed again by the application of S(fr) and bolts at 50% and 70% of the expected total deformation of the lower part. The excavation of the lower bench in the invert arch in the lower part designated by no 3 followed next. S(fr) was then applied when approximately 50% of the expected total deformation for the lower part had occurred. The application of bolts on the lower walls followed next at about 70% of the total expected deformation of the lower part and the model was left to come to equilibrium. When equilibrium was achieved the same procedure was followed for the excavation of the right tunnel. Some deformation results from elastic, plastic models are shown in Figure 3. Results from the models where S(fr) and bolts were used are shown in Figure 4.

support. A 4m span pilot tunnel (1) with variable wall height was originally recommended. In practice it has been proved that there was no need for pilot tunnel since the face stability was satisfactory. The aim was to minimise need for temporary wall support, and keep the wall height below 4m height. Permanent support measures B+S(fr) should be applied in the central arch. The full span top heading (2) was excavated with very small parallel advances using B+S(fr) close to the face (and on a barrel-shaped face if necessary) building RRS ribs at 1-1½m intervals (according to closure monitoring). These ribs were fully integrated with the final 4m long bolts and were continued down from the top heading, with 8m long bolts as anchors for the RRS temporary footing. The full face excavation (3) involved completion of B+S(fr) of the lower walls and completion of the two lower sections of RRS with 8m anchorage of the RRS overlap. The permanent (base) footing of the RRS ribs was effectively anchored by a cast concrete curved invert was bought forward to the bench on a continuous

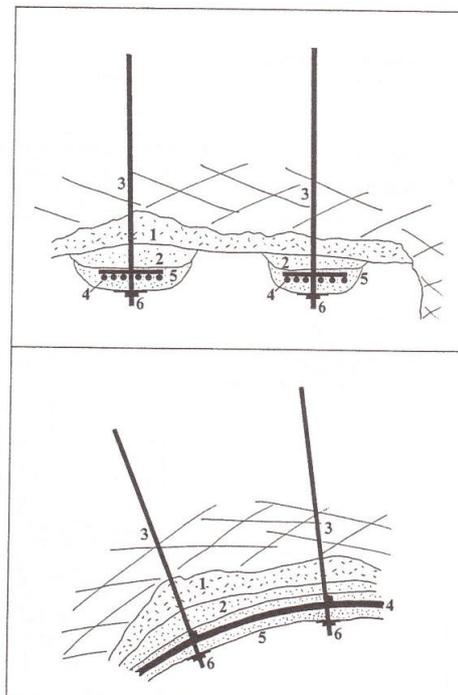


Figure 1. Cross-section and longitudinal-section through RRS arches: 1) S(fr), 2) S, 3) Bolts/cross-piece, 4) six ribs, 5) S, 6) washers and nuts. Each layer of S(fr) or S should exceed 100mm, but may be built in 4 to 6 cm layers.

7 PRACTICAL ASPECTS OF TUNNEL CONSTRUCTION

Figure 5 illustrates the recommended sequence and approximate dimensions of phased excavation and

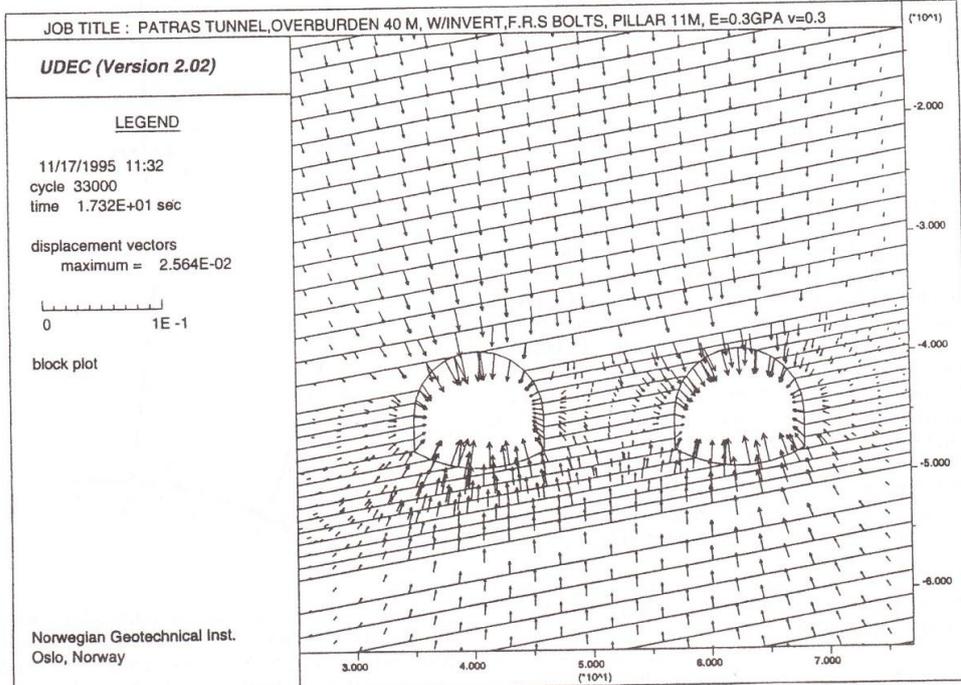
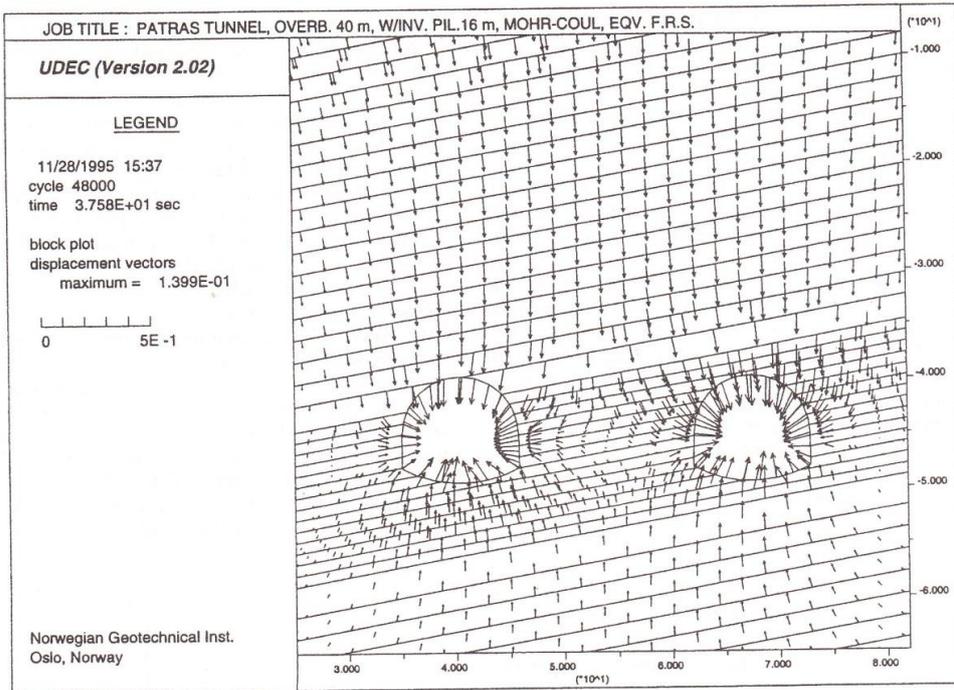


Figure 3. Some deformation results from the elastic (right) and plastic model with 3 excavation phases.

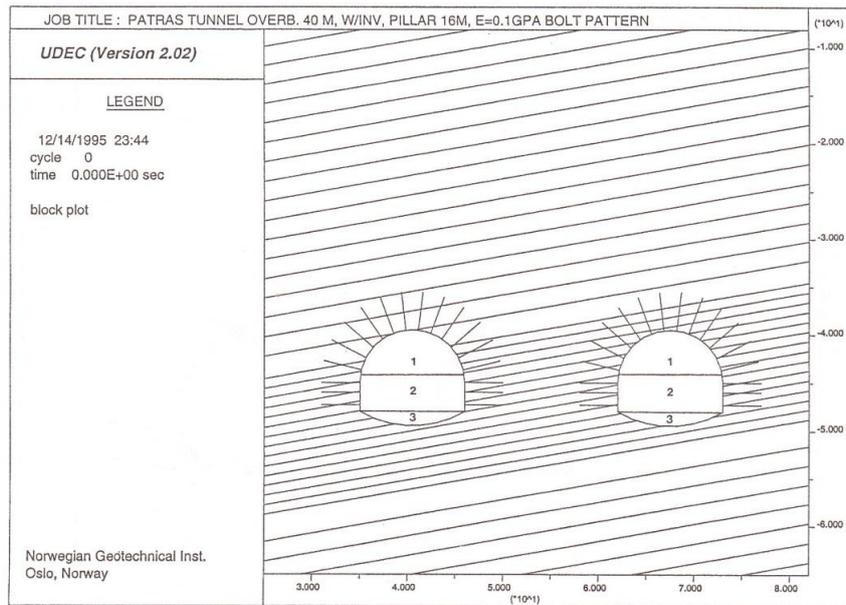


Figure 2. Jointed rock mass geometry with the applied rock bolt pattern and excavation sequence.

basis since the large radius invert has least resistance to deformation, and may be subject to the highest stress.

It was strongly recommended that the full span and full face excavation stages A and B respectively (Figure 5) were staggered, since the parallel tunnel driving has been proven to be the cause of many collapses. In other words, Stage A_I should not be performed at the same time as A_{II} or B_{II}, if A_I is within say, 50m of A_{II} or B_{II}. Likewise, stage B_I should not be performed at the same time as A_{II} or B_{II} if B_I is within 50m of these stages of excavation in the parallel tunnel. The same of course applies to Stages A_{II} and B_{II} in relation to A_I and B_I. In this way, the potentially disturbing effect of adjacent excavation on an incompletely supported tunnel could be avoided. The general trend from our modelling work was that considerably higher S (fr) stresses were exerted on the invert arch compared to other areas. Special attention was also paid to the stability of the pillar walls, with continuous monitoring of closure and additional or more closely spaced bolts and RRS installed as necessary.

8 CONCLUSIONS

1. Design and construction of 12 m span twin tunnels in relatively weak rock has been carried out

at a site in Western Greece, Patras by using the Norwegian Method of Tunnelling (NMT). The rock quality is in general extremely poor, to very poor. The geomechanical properties of these rocks have been assessed based on in situ tests and field investigations for input to numerical modelling studies. The rock mass characterisation approach (Q-system) has been applied extensively to predict and evaluate appropriate rock reinforcement requirements for the tunnels. The Q-values measured ranged between 0.01 and 0.3. The initial design was based on a mean Q-value of about 0.1.

2. The input data for the UDEC-BB models have been derived from rock joint and rock mass characterisation. Nine numerical models were performed with variations mainly in the E modulus, pillar width, S(fr) thickness and overburden. The bolt properties and bolt pattern were derived by means of the Q-system. The discontinuum code UDEC-BB (Barton-Bandis joint constitutive model) was used for the two-dimensional modelling of the twin tunnels. This is a rather conservative approach since several features of the in situ rock behaviour cannot be modelled in 2 D.

3. The need of using an invert arch in the areas of lower overburden (about 20 m) was investigated. The recorded invert deformation when using a flat (non arched) invert were almost three times as high as the deformation in the rest of the tunnel

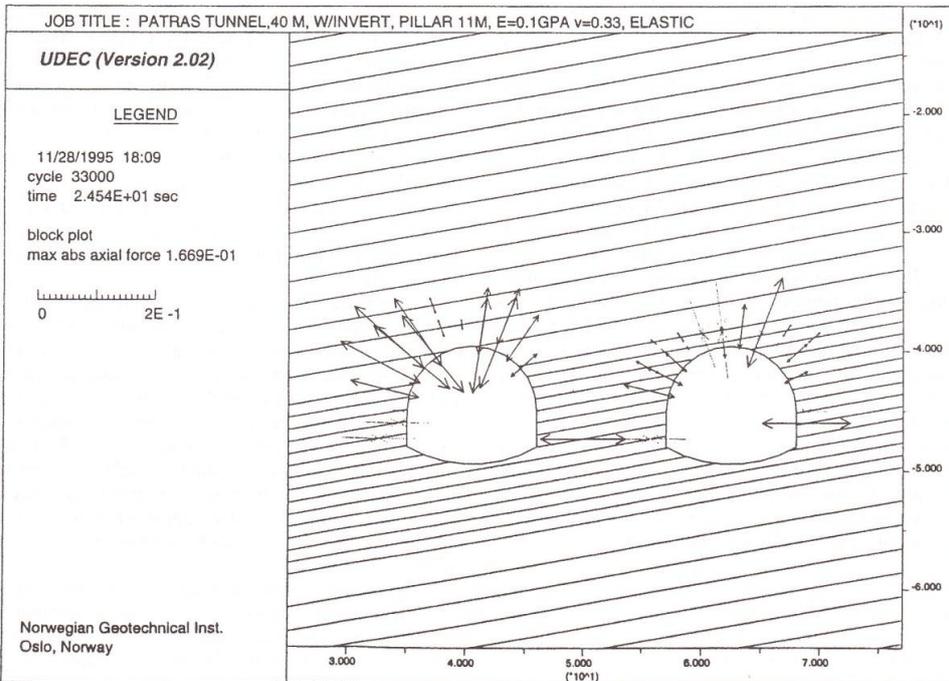
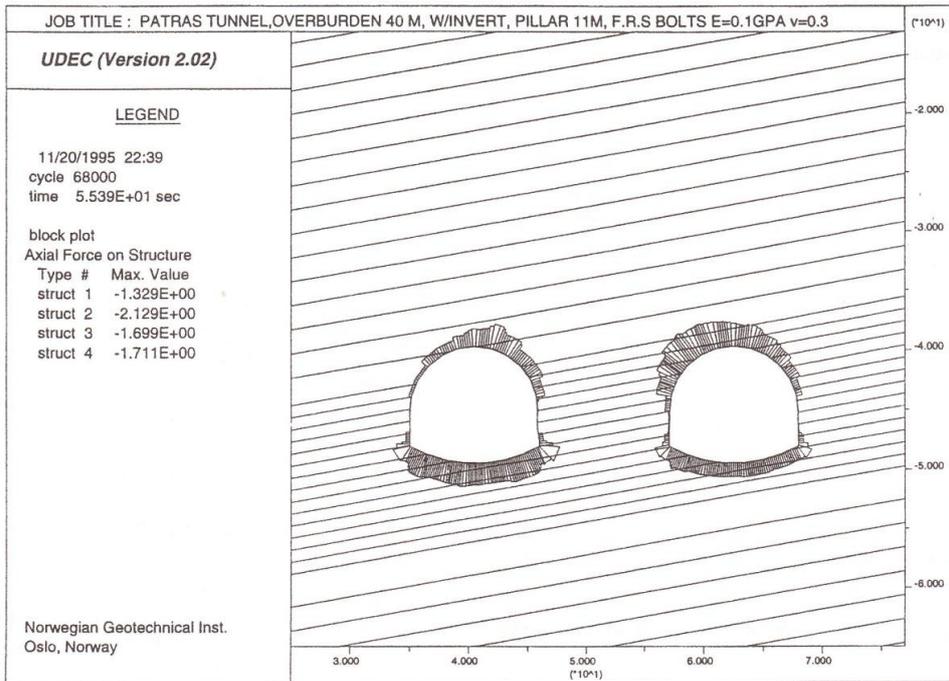


Figure 4. Some results from the models with S(fr) properties. Axial forces in the S(fr), (left) and the bolts.

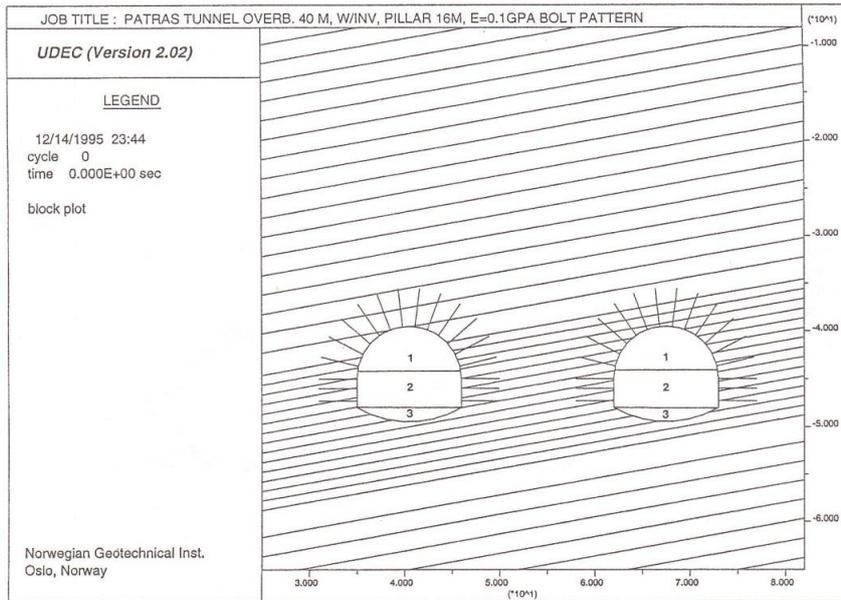


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SEQUENCE EXAMPLE OF FULL SUPPORT

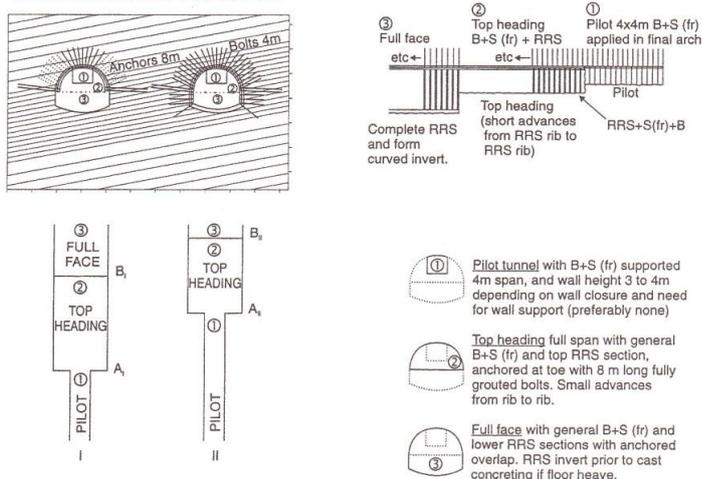


Figure 5. Cross section and plan view of the the twin tunnels, showing the originally suggested excavation sequence (with pilot tunnel).

suggesting that the use of an invert arch was necessary in the whole length of the tunnel whether with 20 or 40 m overburden height.

4. Since there was a possibility for designing the twin tunnels with pillar of 16 m distance it was recommended to do so. Modelling results have shown that it is also possible to excavated the twin tunnels with a pillar width of 11 m, but this could create wall stability problems i.e. significant shear deformation along the sand layers.

5. The reinforced ribs of concrete seem to have been adequate structural support for these rock conditions. The numerical models performed relatively well with the RRS simulation. In situ practice showed that the recommended spacing of 1 to 1.5 m for the RRS section is adequate for these type of tunnelling conditions. A final nominal concrete lining of 40 cm was nevertheless needed for this type of tunnel, due to the existing tunnel regulations in Greece.

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