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A REVIEW OF THE ROCK MECHANICAL AND ENGINEERING GEOLOGICAL RESEARCH AT GJØVIK OLYMPIC CAVERN

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Abstract

The 62 m span Olympic Ice Hockey cavern in Gjøvik, Norway, is located in jointed gneiss of average RQD = 70% and has a rock cover of only 25 to 50m, thus posing challenging design problems. The investigations prior to construction included two types of stress measurements, cross-hole seismic tomography, special core logging, Q-system classification and numerical modelling with UDEC-BB. Predicted maximum deformations were 4 to 8 mm; surprisingly small due to the high horizontal stresses recorded. Extensometer (MPBX) installations from the surface prior to construction, precision surface levelling and MPBX installed from inside the cavern give a combined measure of maximum deformations in the range 7 to 8 mm with the 62 m span fully excavated, and three adjacent caverns for the Postal Services also completed.

1 INTRODUCTION

The idea of locating an Olympic arena in rock was first conceived by Fortifikasjon A/S, who, together with the City of Gjøvik, were responsible for the project development and marketing during the 18 months from when the project was first presented until a contract agreement was reached. As is typical when one is extending the limits of experience and technology, the initial scepticism that had to be overcome was formidable.

The ice hockey cavern was to have a span of 60m, a length of 91 m and a height of 24 m. The spectator capacity is currently 5,300, making it by far the largest cavern for public use in the world.

In 1991, NGI and NOTEBY of Oslo, and SINTEF of Trondheim cooperated in a series of geological and rock mechanics investigations with rock cavern designers Fortifikasjon A/S of Oslo as client.

During the first phase of feasibility studies, existing, nearby rock caverns and access tunnels were mapped in the same hillside in the PreCambrian gneiss. Good rock exposures were available in the arch of a nearby swimming pool cavern, in the arch of a parallel cavern housing the changing rooms, and in the nearby Telephone Exchange caverns. This mapping was done in Phase I before drill holes from the surface were available. Figure 1 shows the location of the swimming pool cavern in relation to the ice hockey cavern.

The PreCambrian gneiss had a frequency of jointing perhaps more than in Norwegian basement rocks in general. However, the joints were generally irregular, rough walled, and with quite large variations in dip and strike. The spacing of the more persistent jointing was often several metres.

The hillside 25 to 50 m above the planned roof of the cavern had a generally smooth

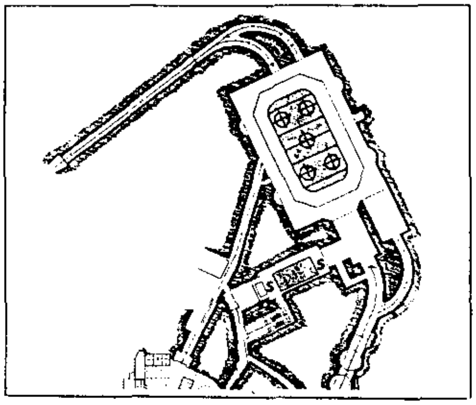

Fig. 1 The location of the swimming pool cavern in relation to the ice hockey cavern.

relief without marked depressions or traces of weakness zones.

The most typical rock quality visible in the existing caverns, which are located between 25 and 100 m from the proposed site was the following:

$$Q = \frac{90}{6} \times \frac{2}{1} \times \frac{1}{1} = 30$$

(RQD) = 90%, two to three joint sets, smooth undulating joints, no alteration, little water in flow, no stress problems.) The poorer quality rock could not be observed due to areas of shotcrete.

2 DRILL CORE ANALYSIS

In Phase II of the investigations, four diamond cored holes of 50 to 70 m length were drilled by NOTEBY. Two were vertical and two inclined at 45°, all of them more or less within the potential 90×60 m footprint of the cavern, which in practice was nearer 100×100 m since cavern orientation had yet to be finalised.

Analysis of joints in the core indicated that a total of five different joint sets could be identified, but these seldom occurred in the same location, and jointing could also be described as sporadic. Some joints were healed due to mineralization. The most typical joint coatings or fillings were rust stains, epidote and quartz. Chlorite and calcite coatings occurred occasionally, while clay fillings appeared to be absent.

While part of the core showed evidence of minor brecciated or crushed zones of up to 0.5 m in thickness, no core loss or marked alteration were registered. In general the joint frequency (F) was 4 to 8 per metre, but perhaps half the breaks in the core were due to diffuse weakness planes and not technically speaking joints. Through-going, well developed joints generally showed a frequency (F) of only 1 to 3 per metre. RQD was generally in the range 70 to 85%, though 15% of the core had RQD = 100%.

3 ROCK MASS CLASSIFICATION

As part of the quality control procedures the first application of the Q-system in the existing rock caverns, and the second application using drill core were carried out by different engineering geologists. Important minor differences were the observations of some poorer quality rock and coated joints in the drill core logging. Such areas were presumably coated with shotcrete which hindered observation in the cavern mapping.

Based on the combined cavern mapping and core logging the following typical rock qualities are expected in the Olympic Ice Hockey cavern.

1. *Typical best quality* $Q = \frac{90}{6} \times \frac{2}{1} \times \frac{1}{1} = 30$

2. *Typical poorest quality* $Q = \frac{30}{9} \times \frac{1.5}{3} \times \frac{0.66}{1} = 1.1$

3. *Weighted average* $Q = \frac{73}{6.6} \times \frac{2.2}{1.8} \times \frac{0.9}{1.0} = 12$

Figure 2 shows the planned 60 m span Ice Hockey cavern plotted on the Q-system rock support diagram. The exceptionally large span and the high safety requirement (ESR = 1.0 to 0.8) places the cavern right at the top or even above the present data base. The

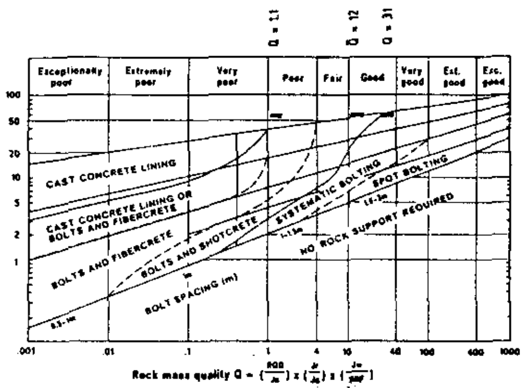

Fig. 2 *The 60 m span places the Olympic Ice Hockey Cavern at the top of the reinforcement data base. (GRIMSTAD et al., 1986)*

need for careful numerical analysis to support the empirically derived reinforcement prediction is evident.

4 PREDICTION FROM CROSS-HOLE SEISMIC TOMOGRAPHY

Important information for subsequent modelling was obtained from NGI's exploratory cross-hole seismic measurements between vertical boreholes 1 and 3 (approximately 52 metres apart) along the planned long axis of the cavern, and between borehole 3 and the 45° inclined borehole No. 2, along a section perpendicular to the planned axis (BY, 1987). The results of the tomographic analysis for these two profiles are shown in Figure 3.
The P-wave velocity of the rock mass surrounding the cavern, which is located between 151 and 176 m.a.s.l., was generally in the range 5000-5500 m/s. In the first 20 m above the arch the velocity was somewhat reduced, lying generally in the range 5100 m/s down to 3700 m/s, with the poorer quality some 10 to 20 m above the arch.

A feature of the results that has indicated good correlation between the prognosis and the excavated conditions is the reduced velocity and rock quality predicted at the ends of the caverns. Subsequent Q-system mapping within the cavern indicated mean Q-values reducing from between 13 and 20 in the central areas to about 5 at the East end and between 2 and 5 at the West end.

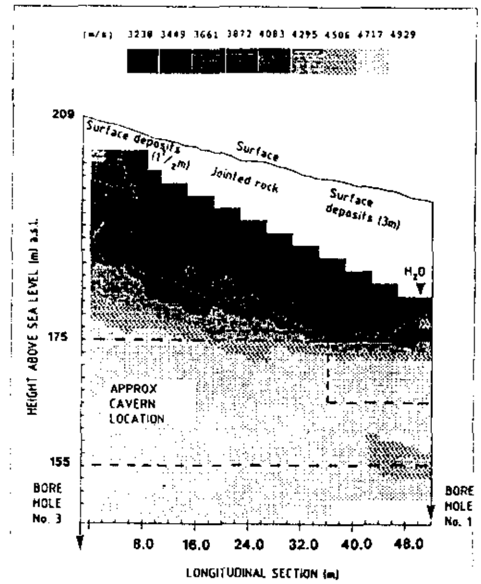

Fig. 3 *Cross-hole seismic tomography for longitudinal cross-section through the cavern site.*

According to BARTON, 1983 rock mass deformation modulus can be estimated on the basis of Q and P-wave velocity. This is further described for the Gjøvik olympic cavern in BARTON et al. 1992.

5 ROCK STRESS MEASURMENTS

Preliminary measurements of rock stress using overcoring were performed by SINTEF, utilising a single 9 m deep hole drilled from an existing cavern. These Phase I measurements indicated a surprisingly high horizontal major principal stress of about 4 MPa with an NE-SW orientation. The minor horizontal stress was measured to 2.2 MPa with an SE-NW orientation, approximately normal to the cavern axis. However, these first measurements needed to be verified by additional stress measurements since they were carried out close to an existing cavern.

Subsequent hydraulic fracturing and hydraulic jacking measurements carried out in Phase II by NGI confirmed the generally high horizontal stress levels, but suggested a N-S principal stress orientation (N 170° E), which was consistent with the N-S set of vertical tension joints and Permian dykes in the Oslo-Region to the south.

Due to the frequency of jointing in the upper 30 m or so of both holes, no measurements were recorded more than 8 metres above the arch. The major horizontal stress was estimated to be 3.5 MPa, 5 metres above the arch of the planned cavern, and oriented N 174° E (approximately N-S). The intermediate stress estimated from joint jacking tests was estimated to be 2.0 MPa, 5 metres above the arch, and oriented N 084° E (approximately E-W). The vertical stress was calculated to be approximately 1.0 MPa at this same location, some 40 m beneath the surface.

Shortly before cavern construction commenced in April, 1991, a further set of hydraulic fracturing stress measurements were made in the upper 30 m of rock. The combined data sets are shown in Figure 4. The high stress to within about 10 m of the surface is a very positive aspect of the site for ensuring the stability of large span excavations.

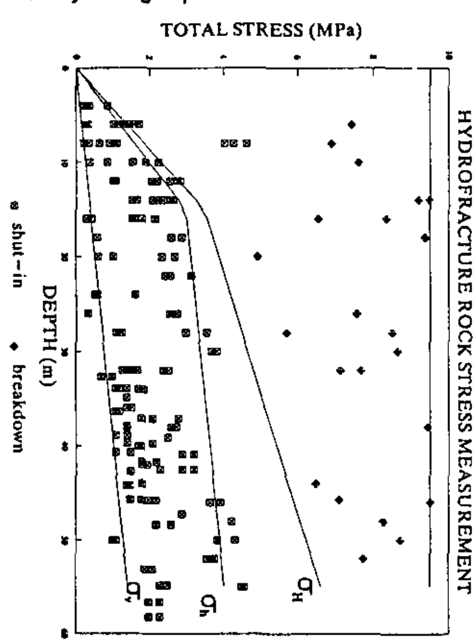

Fig. 4 Results of rock stress measurements using hydraulic fracturing and joint jacking-shut-in.

6 INPUT PARAMETERS FOR NUMERICAL MODELLING

6.1 E-Moduli and Uniaxial Strengths

As part of the overcoring stress measurements performed by SINTEF in Phase I studies, the E-modulus and Poisson's ratio of the rock were measured. Mean values were E = 51.5 GPa, $\nu = 0.21$. According to tests on six 61 mm diameter samples the unconfined compression strength varied from 63 to 94 MPa reflecting the tectonized nature of the gneiss.

6.2 Deformation Moduli for UDEC Studies

In Phase I studies, NGI utilised both the Q-system and the RMR method of BIENIAWSKI (1976) to estimate the rock mass deformation modulus (BARTON, 1983). The estimated range of values was approximately 10 to 50 GPa, with an average of 30 GPa. This value was used in Phase I UDEC-BB calculations. However, in Phase II, deformation modulus was estimated to take mean values of 20, 30 and 40 GPa, in three increasingly deep zones, as interpreted from drill core and seismic tomography results.

6.3 Shear Strength Parameters for the Joints

Joint roughness profiles were measured along 1 m and 2 m long joint surfaces in the existing rock caverns, and gave an approximate indication of large scale waviness, with average (i) values of about 6°. The smaller scale features of joint roughness were measured by performing tilt tests on 101 of the joints recovered in the four drill cores. The most typical value of JRC_0 (joint roughness coefficient, laboratory scale) was about 7.0. When corrected for block-size (approximately 0.5 m *in situ*) the large scale value of JRC_n was 5.2.

Following BARTON & BANDIS (1990), the following assumptions were made concerning the peak friction angles for joints with small scale roughness (without undulations) and for joints with large scale roughness (with undulations that could not be sheared through):

1. *Laboratory scale* $\phi_f = JRC_0 \log \dfrac{JCS_0}{\sigma_n} + \phi_r$

2. *Field scale* $\phi_f = JRC_n \log \dfrac{JCS_n}{\sigma_n} + \phi_r + i$

where σ_n is the effective normal stress.

Input parameters for the Barton-Bandis (BB) joint behaviour sub-routine that is used in NGI's version of Cundall's distinct element two-dimensional (i.e., conservative) code (UDEC-BB) were as follows (MAKURAT et al. 1990):

$JRC_0 = 7.5$ $\quad \sigma_c = 100$ MPa $\quad _n = 0.5$ m

$JCS_0 = 75$ MPa
$\phi_r = 27°$
$i = 6°$

7 NUMERICAL MODELLING

7.1 UDEC-BB modelling by NGI

UDEC-BB modelling was undertaken in three phases. For the phase I modelling, which was carried out ahead of most field investigations, a simplified joint geometry and excavation procedure was assumed. The model was run with three different horizontal stress levels. The very positive effect of high horizontal stress levels were clear from these preliminar runs.

The second phase of UDEC modelling was performed following the core drilling, stress measurement and cross-hole seismic tomography. Input data was therefore more refined. However, the excavation procedure was still simplified, since no final designs were yet developed for the cavern. Figure 5 shows the fracture pattern and the boundary stress conditions assumed in the modelling. This modelling verified the stability of the cavern and gave maximum deformations of 5 mm. The very small deformations calculated suggests that the present depth, span, stress condition and joint character are favourable for the very large span.

The UDEC -BB model geometry used in the third phase of modelling is shown in Figure 6. Phase III modelling was also predictive

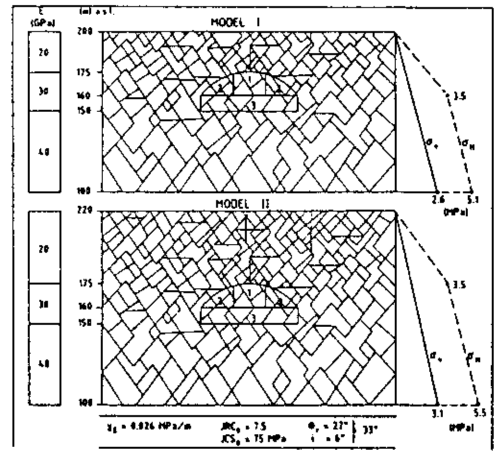

Fig. 5 Phase II assumptions concerning boundary stress conditions, deformation modulus (E) and joint properties. Model I = 25 m overburden, Model II = 45 m overburden.

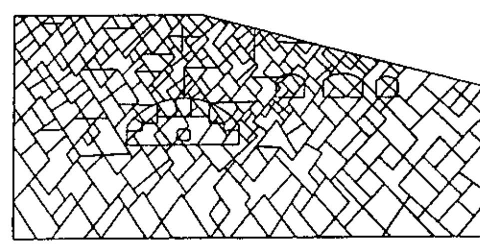

Fig. 6 Phase III UDEC-BB model, showing additional Postal Service caverns.

(Class A); the same basic joint geometry, input data and boundary stress conditions were applied to the Phase III model as those applied in Phase 2 (Figure 5). In this phase the opportunity was provided to follow the actual stages of excavation, and also to model the influence of three new adjacent caverns, which have been excavated since January 1992, for the Norwegian Postal Services.

A diagram of the bolting pattern and excavation sequence that was modelled is shown in Figure 7. The numbers in the cavern area refer to the excavation steps. The rock mass in the cavern arch and the walls were numerically reinforced by untensioned fully grouted

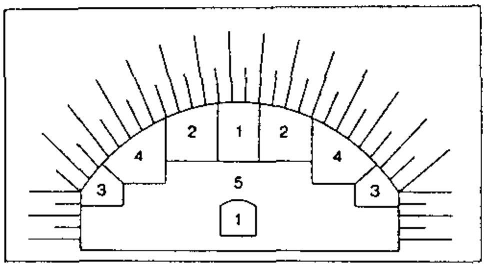

Fig. 7 Excavation sequence and bolting pattern.

rock bolts and anchors after each numerical excavation step.

The UDEC-BB results for each of the excavated steps are given in Table 1. The redistribution of stresses that occurred between the 4th and 5th excavation step are shown in Figure 8.

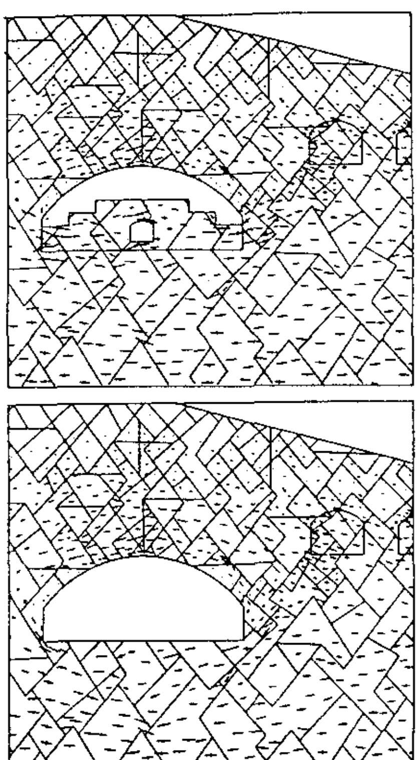

Fig. 8 Redistribution of principal stresses between the 4th and the 5th excavation steps, Phase III.

In the arch of the cavern, some nearly stress free blocks can be observed. These blocks will be secured in practice by the systematic rock bolting. Shotcrete (fibre reinforced) which was not modelled in this study, will secure smaller blocks, representing the detailed joint structure.

Table 1 shows that in the central cavern arch, the maximum downward displacement at Stage 5 was 4.33 mm. This increases slightly with each Postal Services cavern excavation, especially with the last. Figure 9 shows this development rather clearly. Presumably, the excavation of adjacent caverns gradually reduces the horizontal stress levels and allows greater arch displacement.

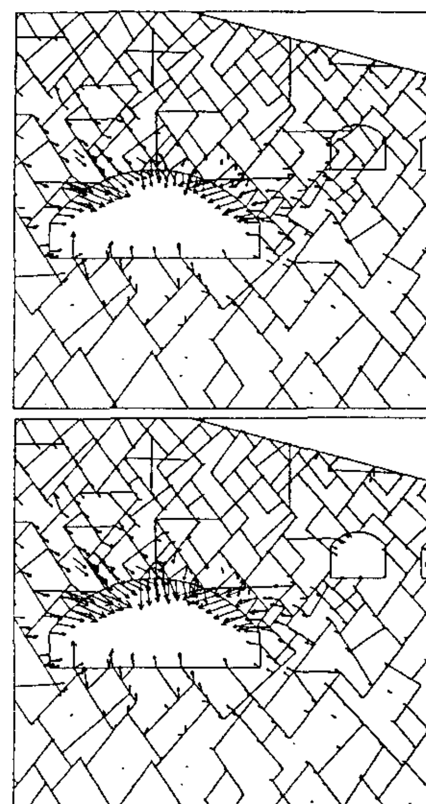

Fig. 9 Development of deformation vectors between the 5th excavation step and the excavation of the third Postal Service cavern.

Table 1. Summary of Gjøvik Ice Hockey Cavern run (with Postal Service caverns).

Parameter	Step 1	Step 2	Step 3	Step 4	Step 5	Excav. of 1st cavern	Excav. of 2nd cavern	Excav. of 3rd cavern
Max. Stress MPa	9.29	11.49	9.91	8.39	8.37	8.56	8.71	8.83
Displacements (mm)								
• maximum								
• wall	1.85	1.80	2.63	6.99	8.16	8.28	8.43	8.65
• crown (vertical component)	-	-	-	1.33	3.78	3.88	3.92	3.97
	0.50	1.08	2.62	4.05	4.33	4.39	4.87	7.01

7.2 FEM modelling by SINTEF

The modelling by SINTEF was conducted subsequent to cavern excavation. The aim of the modelling was to compare results from FEM- and UDEC analysis. A 2D contieous, non-linear finite element program was used for the modelling.

The study included four different combinations of *in-situ* stresses and Mohr-Coulomb parameters. Real ground surface profile was modelled due to the limited overburden, and a five-stage excavation sequence including the rock support was followed (Figure 7). Excavation of the Postal Service Caverns has not been included in this modelling.

One set of Mohr-Coulomb parameters are obtained from a linear fit of the Barton-Bandis (BB) peak shear strength envelope based on parameters measured by NGI. In this way, M-C parameters c=0.3 MPa and ϕ=39.5° are derived from JRC=7.5, JCS=75 MPa, ϕ_r=27° and i=6° of the BB model. It should be mentioned that these parameters are for joints and consequently used as a lower bound of the rock mass parameters in the modelling. As the upper bound c=0.5 MPa and ϕ=45° have been used.

The mesh adopted for the analysis, shown in Figure 10, represents the central cross section of the cavern. Figure 10 also shows the two different horizontal stress levels used in the FEM calculations. Based on the stress measurements, these are believed to represent the upper and lower bound. Rock bolts are taken into account in each excavation stage by means of incorporating corresponding bolt elements.

Table 2 summarizes input parameters and predicted maximum displacement of cavern crown in each excavation stage for the four combinations of stress levels and Mohr-Coulomb parameters. Due to the way the

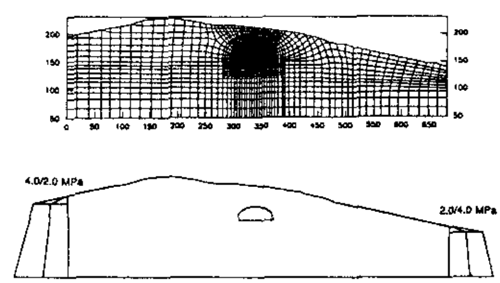

Fig. 10 Mesh and horizontal stress levels used in SINTEFs FEM-modelling.

Table 2. Results from FEM-modelling.

Boundary stress level	Mohr-Coulomb parameters		Maximum crown deformation (mm)
	c (MPa)	ϕ	
Low	0.3	39.5	3.9
High	0.3	39.5	14.5
Low	0.3	45.0	3.5
High	0.3	45.0	8.7

parameters are chosen, the real crown displacement should be expected to be within the range defined by these calculations. This is also the case, the range of crown displacement in Table 2 is 3.5-14.5 mm while the measured subsidence is 8 mm.

The calculations confirm the general stability of the cavern although a yielding zone extending more than 15 m above the cavern crown has been identified.

8 DEFORMATION MONITORING AND COMPARISON WITH UDEC-BB RESULTS

NGI, NOTEBY, and SINTEF have all been involved in extensive monitoring studies. The near surface location of the cavern has meant that extensometers could be placed in boreholes from the surface using holes of 30 to 40 m depth. Figure 11 shows the location of these extensometers (E1 to E7) which were installed prior to cavern construction, and reach to 1.5 or 2 m above the cavern arch. Surface precision levelling was carried our at E1, E4 and E7 installations (above the cavern centre-line), and showed a gradual increase of subsidence to between 2.5 and 3 mm during the four months it took to excavate the full 62 m wide x 91 m long x 8 m high top heading. The initial deformations recorded by the six Interfels MPBX are shown in Figure 12. Some episodes of slight heave (arching upwards) are seen adjacent to excavation faces due to the high stresses. The centre of the cavern area (at E4) has subsided at this stage approximately 3 mm relative to the surface 35 m above. This 3 mm has to be added to the surface subsidence of 2.5 to 3 mm to obtain the preliminary net deformation.

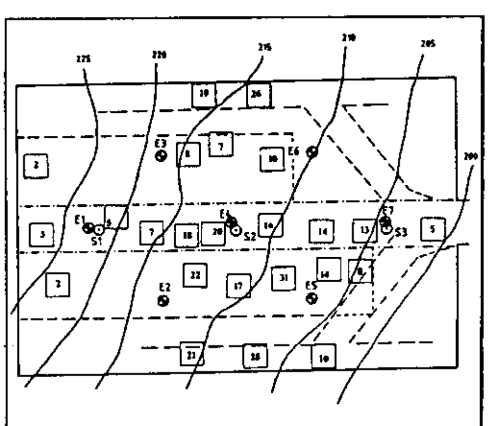

Fig. 11 Location of surface MPBX extensometers (E1 to E7) and cavern extensometers (S1 to S3) shown in relation to local rock mass Q-values (squares) as mapped in the cavern arch and access ramps.

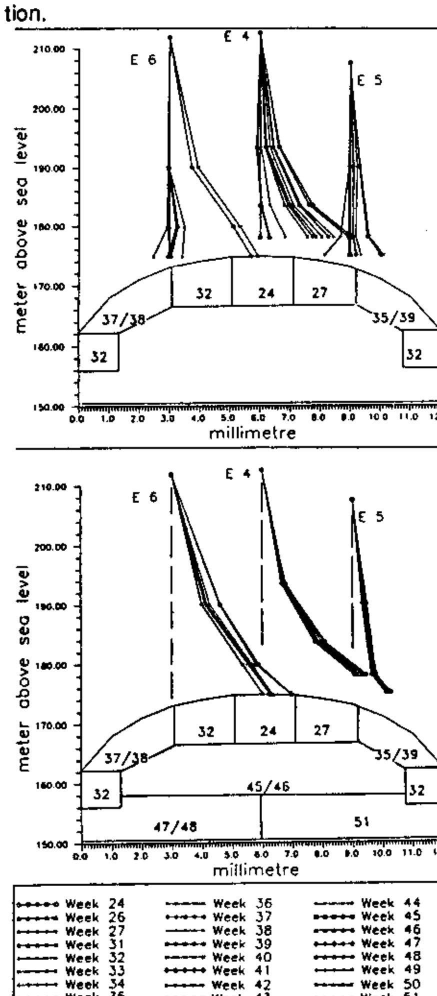

Fig. 12 Six MPBX installed from the surface prior to construction, monitored the weekly excavation. (Heave to the left, subsidence to the right). Results for E4, E5 and E6.

A further source of deformation was the dilation of the blast damaged zone within the cavern. SINTEF's twin anchor S1 to S3 extensometer bolts of 2 m and 13 m length were located in the arch behind the advancing face in the 80 m^2 pilot tunnel. They showed maximum values of 2.8, 1.3 and 2.9 mm.

To obtain the maximum deformation of the centre of the arch, it is necessary to add E4 and S2 results to the maximum surface subsidence of approximately 4.5 mm. A value of 8.2 mm is obtained. The equivalent result at E1/S1 towards the SW end of the cavern is 7.0 mm, and towards the NE end (E7/S3) the result is 7.5 mm. (NOTEBY's sliding micrometer (E7) showed a total 2.23 mm of deformation over its 28 m length.)

Figure 13 shows the cumulative results for the three sets of instruments located along the cavern centre-line.

9 MONITORING OF BOLT PERFORMANCE

The systematic bolt-pattern consists of one bolt or cable per 2.5 m x 2.5 m square. In this pattern every fourth bolt is a 12 m long twin steel-strand, each with a diameter of 25 mm. The rebars are 6 m long and have a diameter of 25 mm. All bolts and cables are fully grouted. Mechanical properties of the bolts and cables are provided in Table 3.

To check the performance of the rebar bolts, 8 bolts placed approximately midspan, were instrumented with axial resistance strain gauges. The instrumented rebars are a part of the ordinary bolt pattern, and were installed in the same way as the others (MYRVANG et al., 1992)

The instrumented rebars were installed during the first stage of excavation. The instrumented rebars were installed close to borehole extensometers. This gave the opportunity to compare the load build-up in the bolts with deformations monitored in the nearby extensometers. Each instrumented bolt is supplied with 10 measuring points along the axis of the bolt. The distance between each point is 540 mm.

Two of the bolts were destroyed by blasting or shortcircuiting of leads while the remaining six still are operating. In general, the load increase in the bolt is low and occurs only over a limited length of the bolts rather close to the roof surface.

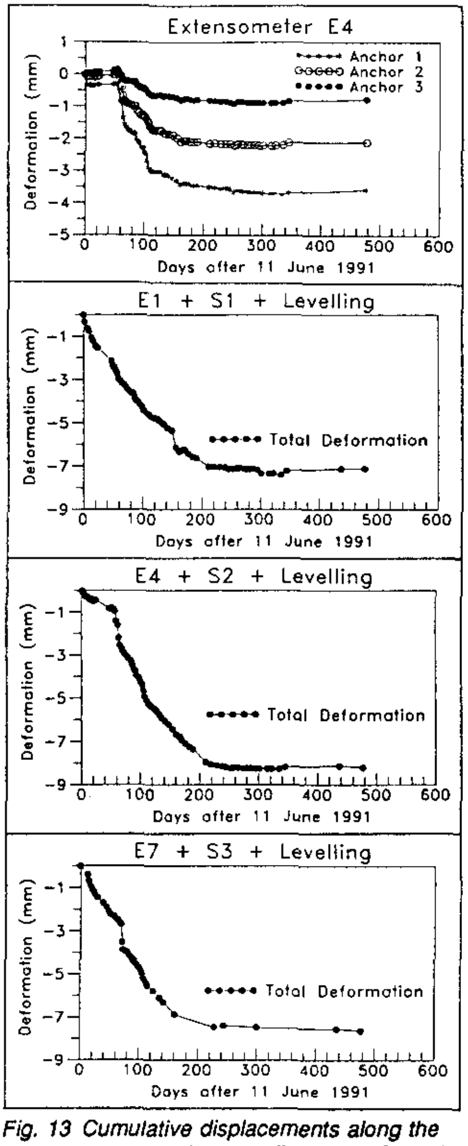

Fig. 13 Cumulative displacements along the cavern arch centre-line. The Olympic cavern was completely excavated at 200 days, the Postal Service caverns at 350 days. (7.0, 8.2 and 7.5mm = maximum centre-line displacements.)

Table 3. *Mechanical properties of the rebars and the cables.*

Type of bolt	Yielding load [kN]	Ultimate load [kN]	Corrosion protection
6 m long 25 mm rebars	220	250	Hot dip galvanized and epoxy coated
Twin 12.5 mm steel-cables	330	370	None

Three instrumented bolts close to the extensometer S1 show maximum load in the range 1-1.5 kN. Instrumented bolts number 5 (close to extensimeter S2) and number 6 and 7 (close to extensometer S3) all shows a maximum load in the range 40-105 kN (18-47% of yielding capacity). Figure 14 shows the load distribution along bolt number 6, which is the most stressed one.

Comparison of deformations at extensometer station S2 and the load build-up in bolt no. 5 indicates good correlation between deformatins in the cavern roof and the load build-up, as shown in Figure 15.

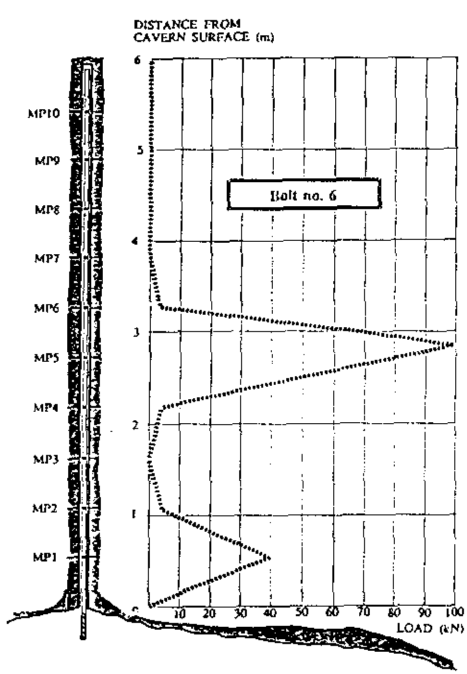

Fig. 14 Load distribution along instrumented rebar number 6.

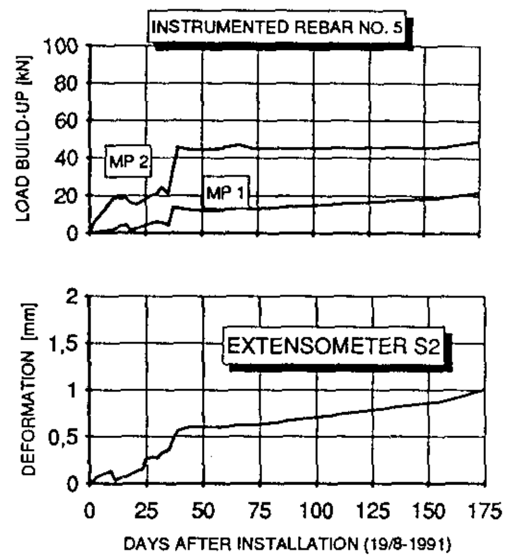

Fig. 15 Load build-up in bolt no. 5 compared to deformation in extensometer S2.

10 ROCK MASS QUALITY IN THE CAVERN

During core logging in pre-investigation studies, the mean Q-value calculated by means of data from boreholes gave a slightly higher rock quality than that found in the cavern. The difference in the two values is attributed to blasting which created artificial joints and opened up healed joints.

The rock in the cavern is red and grey jointed gneiss of PreCambrian age with 25 to 50m overburden. The average RQD (Rock Quality Designation) of 60-70% represents a fair rock quality. The PreCambrian gneiss has a network of micro-joints and isolated zones with clay fillings. The geotechnical investigations during the construction involved registration of rock quality using the Q-system and detailed joint surveys in the excavated portions of the cavern provided data on joint orientations, joint conditions and spacing. Measurements of strike and dip of the main discontinuities were made throughout the cavern. Data from 35 areas which together make up the majority of the upper part of the cavern were collected (LØSET AND BHASIN, 1991) and the relevant Q-values are shown in Figure 11 (see numbers in squares). These confirm the lower quality of rock at the ends of the cavern, as seen in the cross-hole seismic tomography (Figure 3).

Key geological parameters including the Q-system data for rock mass classification and for subsequent jointed rock modelling studies have been recorded in histograms as shown in Figure 16. This method of recording the six Q-system parameters and other geotechnical information during field work for small or large areas has been found to be very useful. Incorporating all the information in a PC-based spread sheet makes it possible to see the variation in the different parameters through the cavern. Hence, data from different areas may be manipulated and combined. The geotechnical chart contains information for setting up input data files for numerical modelling of critical sections of the cavern.

The jointing in the cavern is irregular, rough walled and with quite large variations in dip and strike. These joint properties along with the rather high horizontal stresses of about 3.5 to 4 MPa at a depth of 45 m below the surface (and perpendicular to the long axis of the hall) have been found to be very favourable for the stability of the cavern, and explain the rather moderate displacement magnitudes.

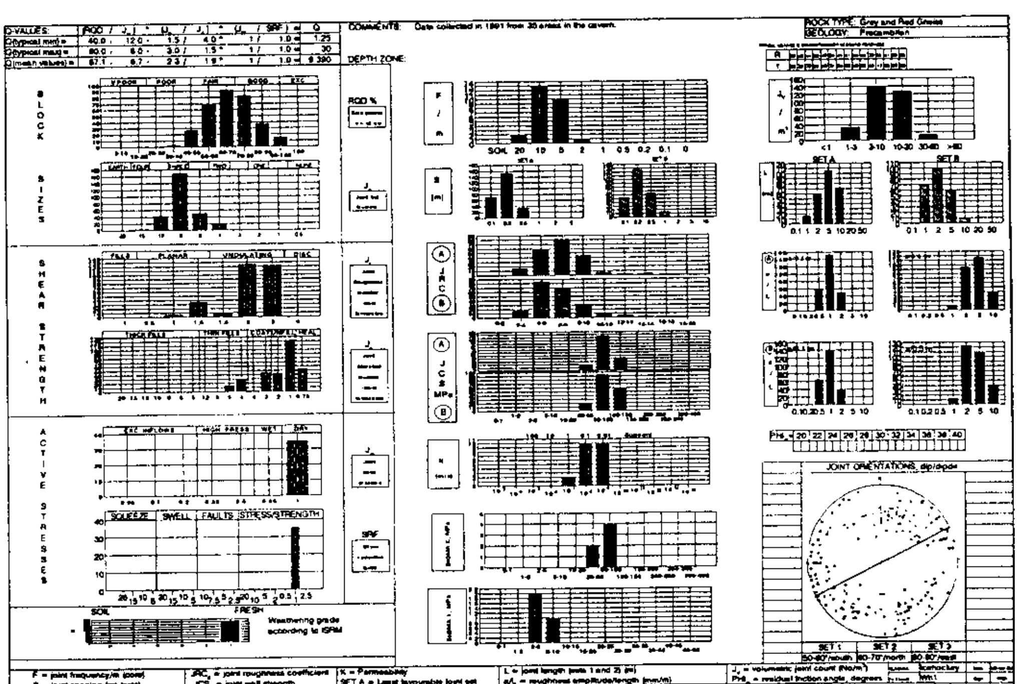

Fig 16 Geotechnical logging results for the arch of the Olympic cavern. (LØSET & BHASIN, 1991)

11 CONCLUSIONS

An exceptionally large span (62m) rock cavern has been constructed in rock of poor to fair to good quality (Q=1 to 31). Mean Q values observed in phase I mapping prior to construction were 12 and the predicted range was 1 to 31. Mean Q values observed in the cavern pilot tunnel of $80m^2$ cross-section were 7.4 (typical range 4-27), and in the cavern arch as a whole 9.4. The typical range observed here was 1 to 30 as predicted.

Key properties favourable to large cavern construction have been the high horizontal stresses and the relative roughness or waviness of foliation planes and of the more continuous joints. The B and S(fr) method of rock reinforcement has taken care of less favourable features such as the less favourable joint orientations and mean RQD's of only 65 to 70% in the disturbed rock exposed by the blasting.

The usefulness of rock mass characterisation, cross-hole seismic tomography and rock stress measurements has been demonstrated by the good definition of input data for forwardprediction (class A) modelling. Four different methods of deformation measurements have confirmed the relatively small deformations, that were predicted by UDEC-BB to be as little as 5 mm (Phase II simplified model) and as large as 8 mm (Phase III, realistic excavation sequence).

Precision levelling of the MPBX instrument heads at the surface, combined with deformation measurement throughout 30 to 40 m of rock overlying the cavern indicate maximum deformations in the range 7 to 8 mm along the cavern arch centre-line.

The effective prediction of behaviour prior to cavern design, the efficient excavation and rock reinforcement, and the continuous monitoring of behaviour have provided confidence in the project and have been instrumental in its completion ahead of schedule and within budget.

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