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GJØVIK OLYMPIC CAVERN ENGINEERING GEOLOGY AND ROCK MECHANICS

PER BOLLINGMO
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Abstract

The Gjøvik Olympic Cavern is a unique project regarding its size. Engineering geological and rock mechanical investigations have been conducted according to well proven methods used in Norway. Methods as refraction seismics, core drillings, tomography, stress measurements with both overcoring and hydraulic splitting have been applied. Also various numerical methods for analyses of stress and deformation conditions around the cavern are used.

Experience from nearby rock installations, the oldest from 1972, have also given valuable information for engineering geological evaluations on the rock behaviour, layout, design of tunnels and caverns, and supporting works.

As a cooperation between the consulting engineer, contractors, and research organizations, a comprehensive research program has been carried out during planning and construction.

Predictions made from theoretical analyses and general engineering geological experience and knowledge have proved to comply very well with the actual conditions during construction, and the excavation and supporting works were successfully completed in 9 months, 4 months ahead of schedule.

1 INTRODUCTION

When the city of Gjøvik accepted to be responsible for the construction of an ice hockey arena for the 1994 Olympic Winter Games, the idea of making this to an underground installation came up very early. The first meeting was arranged in October 1989.

As the engineering geological consultant for several other rock installations in the same area, NOTEBY was asked if a cavern with a 60 m span was possible. With our positive experience from the rock conditions in the actual area, and good opportunities for observation and investigations in the existing caverns, we where quite optimistic about the project, and with some reservations for proper investigations and documentation of the geological properties, we advised the political authorities to go ahead with a phase 1 investigation.

With a challenging project like this, and also with the opportunities for research and scientific activities, we advised that a broad range of the Norwegian competence within engineering geology and rock mechanics should be engaged in the project. Therefore, SINTEF (the Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology) and the Norwegian Geotechnical Institute (NGI) were invited to participate in the project. They conducted the research programme and cooperated closely with NOTEBY as the consulting engineers within engineering geology and rock mechanics throughout the whole period with investigations, design and construction.

As part of a quality assurance, both different methods and personnel from these institutions were used for registrations and analyses of the same subjects.

2 GEOLOGICAL BASIS

The actual area for location of the arena is within a precambrian gneiss.

The joints are mainly short, with rough and

irregular surfaces.
The experience from the nearby caverns, of which the first one was excavated in 1972, is that the rock is very stable, and rock walls and cavern roofs with only rock bolts as support have shown very little rock falls. There was also very dry in the existing tunnels and caverns, with only scattered dripping.

There are no significant faults or crushed zones in the rock mass that effected the location and orientation of the cavern.
The location of the cavern should preferably be in close connection to a swimming pool installed in a 20 m wide cavern (constructed in 1975), and with this setting the rock cover above the ice hockey arena would be some 25 m in one end of the cavern, and 52 m in the other.

The overburden consist of moraine with 5 m maximum thickness.

3 GEO-INVESTIGATIONS, PHASE 1

The most important part of this early stage of the study was to obtain information about the stress level in the area. Therefore, stress measurements by the overcoring method was performed from one of the nearby caverns (see fig.1)

Laboratory tests on rock cores obtained from the stress measuring were carried out for input to numerical analyses.

Also a thorough engineering geological mapping in the existing tunnels and caverns was carried out.

The following results were obtained:

Rock type description

The rock mass consists of a precambrian gneiss with a granitic to dioritic composition.

Typical mineral content determined from thin sections of five rock samples are:

- Quartz 20-38%
- Feldspar 48-65%
- Chlorite 2-5%
- Muscovite 0-3%
- Epidote 0-18%
- Amphibole 0-5%

A common grain size in the rock is 0.5 mm. Partly the rock is tectonized with microfractures healed with calcite, epidote or chlorite. These microfractures are rough, irregular and with wide range of orientation.

There is a faint and poorly developed foliation with a general E-W strike direction, and a dip of 35-55°S.

The jointing is generally irregular, with short fractures and rough surfaces. A joint diagram is shown on fig.2.

A rock mass classification in the existing caverns, based on the Q-system shows a typical value of Q= 30. This is most likely a too high value, because the poorer rock sections are covered by shotcrete.

Stress measurements

The stress measurements show the following values:

σ_1 = 4.3 MPa bearing 293°, dip 19°
σ_2 = 3.4 MPa bearing 202°, dip 4°
σ_3 = 1.8 MPa bearing 100°, dip 71°

Rock cover at the measuring site is approx. 40 m.

Rock mechanical properties

E- Modulus	E	51.5 GPa
Poisson's ratio	v	0.21
UCS	σ_c	77.3 MPa
Angle of failure	ß	14°
Point load strength	σ_s	15.2 MPa
Ultrasonic velocity	v	4953 m/s
Specific gravity	τ	2700 kg/m³

Numerical modelling

Three different numerical models were used to analyze stress distribution and deformation around the cavern during excavation.

NOTEBY used a finite element method, SINTEF a boundary element analysis (BESOL), and NGI used a distinct element model (UDEC-BB).(See fig. 3 and 4)

The laboratory testing indicated a deformation modulus of
51.5 GPa in unjointed rock. In the numerical modelling a range of values from 10 to 50, with a typical average of 30 GPa, and ν=0.21 has been applied.

The results from the various numerical analyses showed a satisfactory total stability of the cavern, and that only ordinary supporting means like rock bolts and shotcrete to secure the detailed stability would be needed.

The preliminary conclusion from these phase 1 investigations was that the results were promising, but further investigations should be implemented.

4 GEO-INVESTIGATIONS PHASE 2

The next phase started with refraction seismic profiling in 5 profiles, totalling 575 m.(See fig.1)

There are relatively low seismic velocities in the bedrock, 3600-4500 m/s. Some areas with velocities of 3000-3500 m/s were evaluated as more fractured rock mass in the surface, and not crushed zones or faults. This was later confirmed by core drillings and tomography.

Core drillings were carried out in 4 holes, 3 vertical and 1 inclined.

RQD-classification of the drill cores shows generally values between 50-90, and an average of approx. 70.(see fig.5)

Rock cores were also used for determination of joint parameters later applied in numerical analyses.

Tomography profiles were performed between core drilling holes 1, 2 and 3. The results as shown on fig.6, have good correlation with results from the seismics and the core drillings. No major crushed zones were detected, but the diagrams give good information on the general fracturing in the rock mass.

Stress measurements by using hydraulic splitting in boreholes 1 and 3 were then carried out in order to obtain a confirmation of the results from the overcoring tests. It was also required to obtain results from a more central part of the cavern area, and to get a distance from the influence of the existing caverns.

These measurements gave the following results:

σ_1 = 3.5 MPa, bearing 174°, horizontal
σ_2 = 2.0 MPa, bearing 84°, horizontal
σ_3 = 1.2 MPa, vertical

The difference between these results and the results from the overcoring method is due to the difference in depth. With a correction for this, the results would be nearly similar.
The difference in bearing is due to influence from the existing cavern where the overcoring measurement was carried out.

Also a more comprehensive numerical analysis was executed, using the UDEC-BB method.

The conclusion from the phase 2 investigations was still that the conditions for the cavern was favourable.

5 DESIGN

Orientation and location

The most important criterias for orientation of the cavern are normally the orientation of the horizontal stresses, and the orientation of the most prominent joint sets.

In this case both σ_1 and σ_2 are high, and there are no favourable orientation due to jointing.
Therefore more practical requirements could be considered, such as a preferred combination with the existing swimming pool in the nearby cavern.

Rock mass properties indicated that the cavern should have been located some 10m further into the mountain and preferably also some 5 m lower in order to avoid fractured rock. However, this was not major objections, and the cavern was given a location and orientation where we obtained a practical layout regarding access and combination with the existing rock installations.

Rock support

A systematic rock support was decided, based on experience from the nearby existing caverns, and the findings from the preinvestigations, mainly the character of the rock joints, and the size of blocks to be supported in order to secure the detailed stability.

Also the Q-method was applied to support the decisions on rock support.

A rock bolting pattern of 2.5x2.5 m was applied in the roof, with alternating 6 m long rebar bolts (25 mm diameter and capacity 22 tons), and twin-strand cables (12.5 mm diameter and capacity 16.7 tons at yield each). All bolts were fully grouted.(See fig.7)

Also 10 cm of steel fibre reinforced shotcrete (wet mix) was applied on all surfaces. Concrete strength was 35 MPa, and fibre dosage was 50kg/m^3 of 25 mm EE-fibres.

6 THE CONSTRUCTION PERIOD

Supervision

The construction period was supervised by an engineering geologist that continuously recorded all geological features, advised the contractors supplementary rock bolting, and did all readings and registrations from the instruments installed for monitoring.

Instrumentation

For monitoring of the deformations in the rock mass above the cavern, 6 extensometers and 1 sliding micrometer was installed in boreholes from the terrain surface down to 1.5 m above the cavern roof as shown on fig.8 and 9.

Also a precision levelling of 3 extensometers at the surface to determine the surface subsidence was performed. The accuracy of the levelling was however not satisfactory, assumed to be within approx. 2 mm.

Also extensometers (2 m and 13 m long) were installed from the pilot tunnel in the cavern arch for monitoring of the deformations in the immediate zone above the cavern roof.

Typical extensometer diagrams are shown on figs.10 and 11.

To obtain the total deformation of the rock mass above the cavern, these values should be added to the subsidence measured by surface levelling, and hence the total deformation added up to approx. 8 mm, of which approx. 4.5 mm are surface subsidence.

The results comply very well with the deformations predicted from the numerical analyses (4-8 mm).

Also blast vibrations were monitored continuously both in nearby houses, and on technical installations in the swimming pool and the telecommunication centre. No serious damages occurred from blast vibrations.

7 CONCLUDING REMARKS

The engineering geological and rock mechanical investigations for the Gjøvik Olympic Cavern were carried out using conventional and well proven methods in Norway.

The results from the investigations were compared with the experience from the nearby rock installations in caverns constructed during the last 20 years. This ended up in a simple, safe and cheap design regarding layout, excavation methods and supporting works.

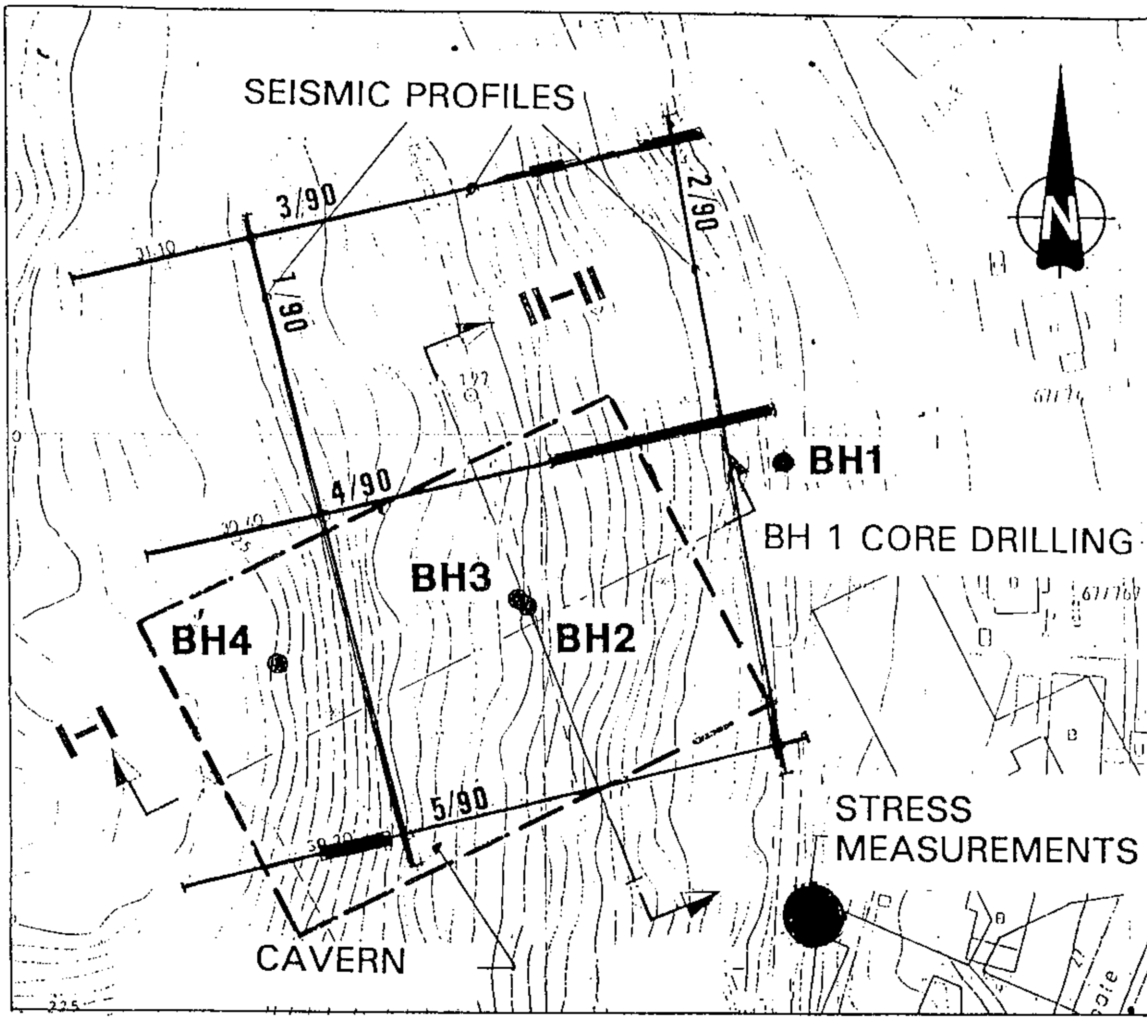
The excavation of the tunnelling system and the main cavern was successfully completed in 9 months, 4 months ahead of schedule, without any serious accidents and according to all predictions as regards engineering geology and rock mechanics.

8 ACKNOWLEDGEMENTS

The geological and rock mechanical elaborations for this project were successfully conducted during a joint effort between the consulting engineer NOTEBY A/S, and the research organisations SINTEF and NGI. We thank all the participants from these institutions for good cooperation with a unique and very interesting project.

A thank is also given to the main contractor, the Veidekke and Selmer joint venture group, and members of the User Group for good cooperation and useful support. The User Group consist of the following institutions and companies:

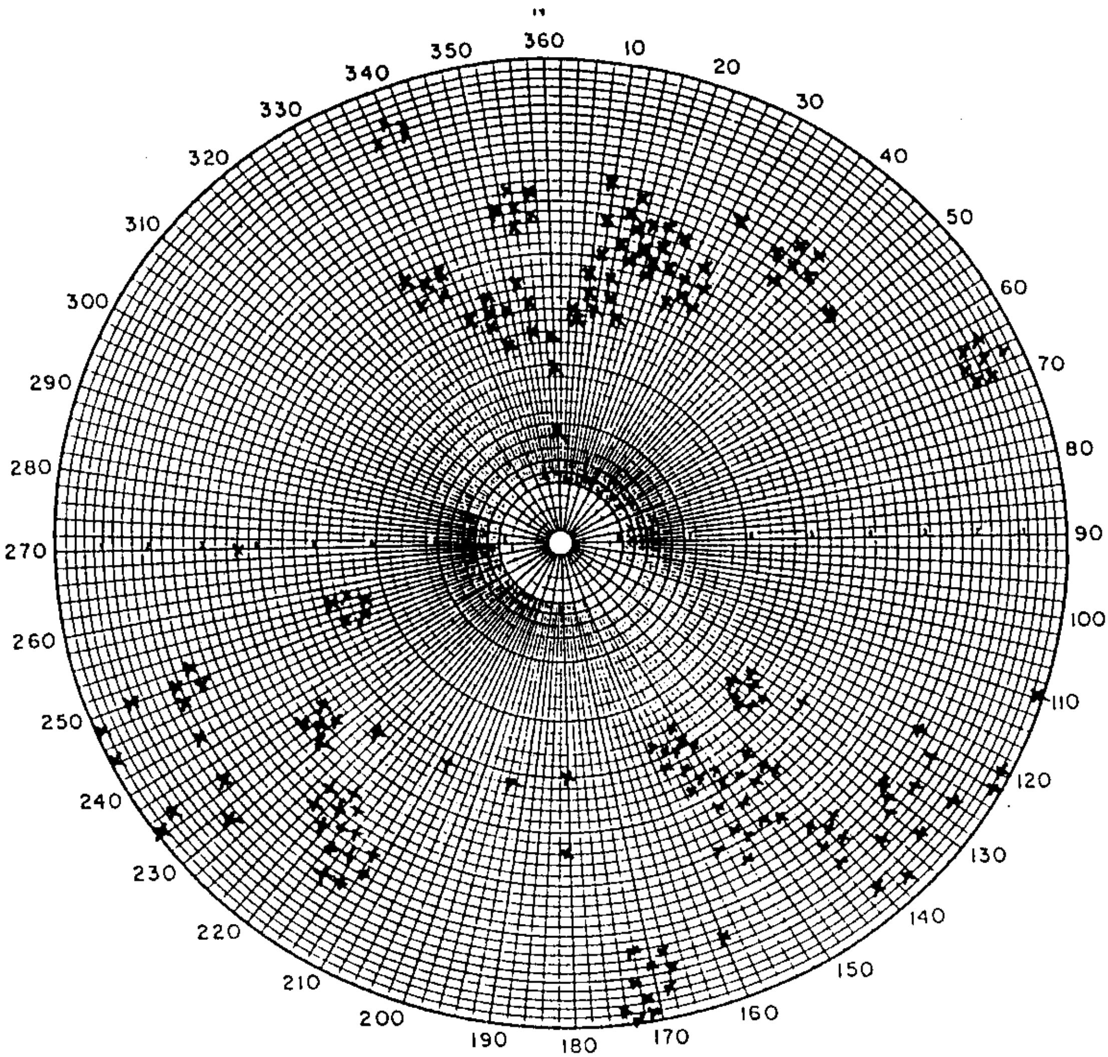
Veidekke, Selmer, Fortifikasjon, Dyno Industries, Statoil, Statkraft, Televerket, Gjøvik Municipality, Østlandsforskning, BeFo, Hyundai, SINTEF, NGI, and NOTEBY.



GJØVIK OLYMPIC CAVERN

INVESTIGATION PLAN

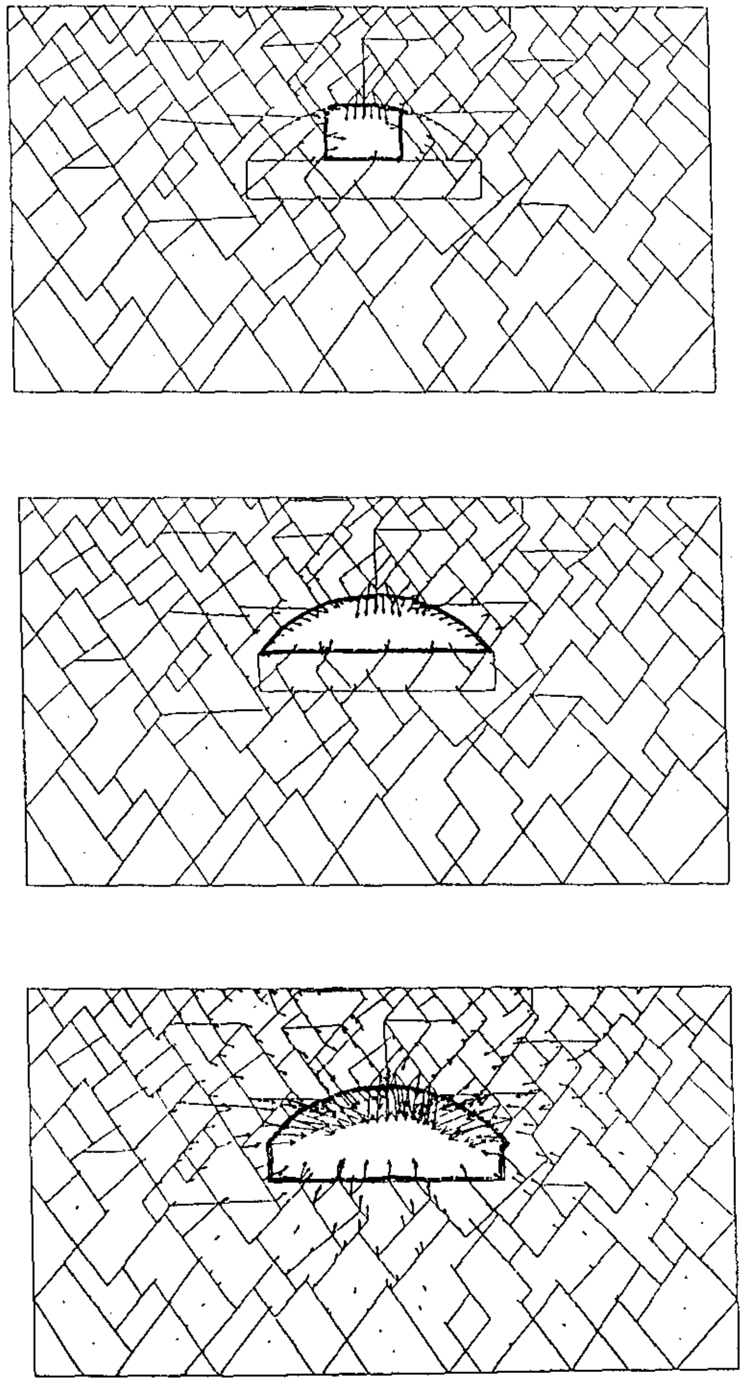
Fig. 1



Polar Equal Area Net
(Schmidt Net)

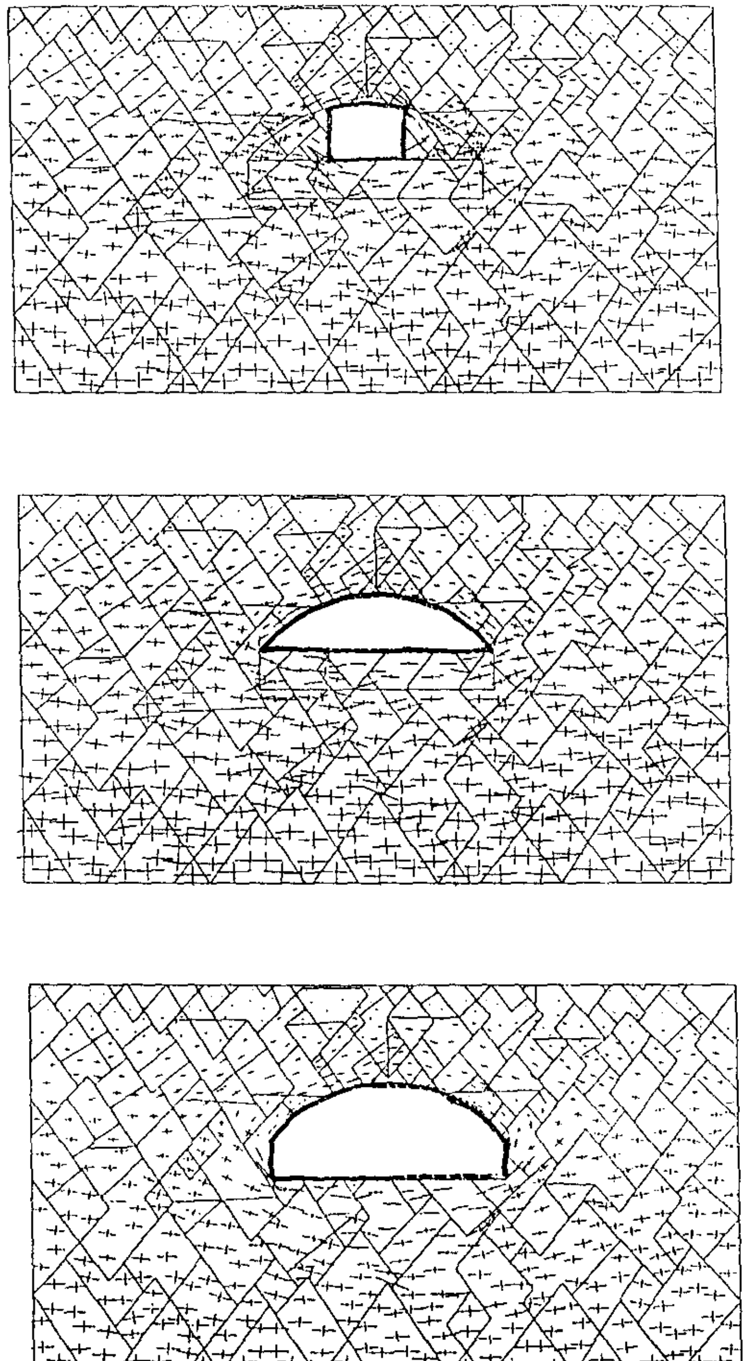
JOINT DIAGRAM

Fig. 2



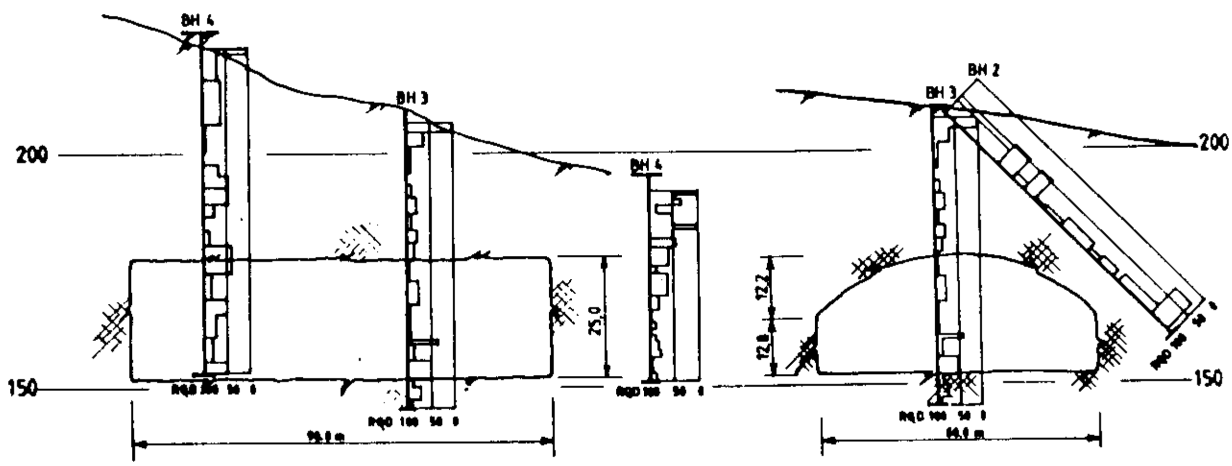
NUMERICAL MODELLING
DISPLACEMENTS

Fig. 3



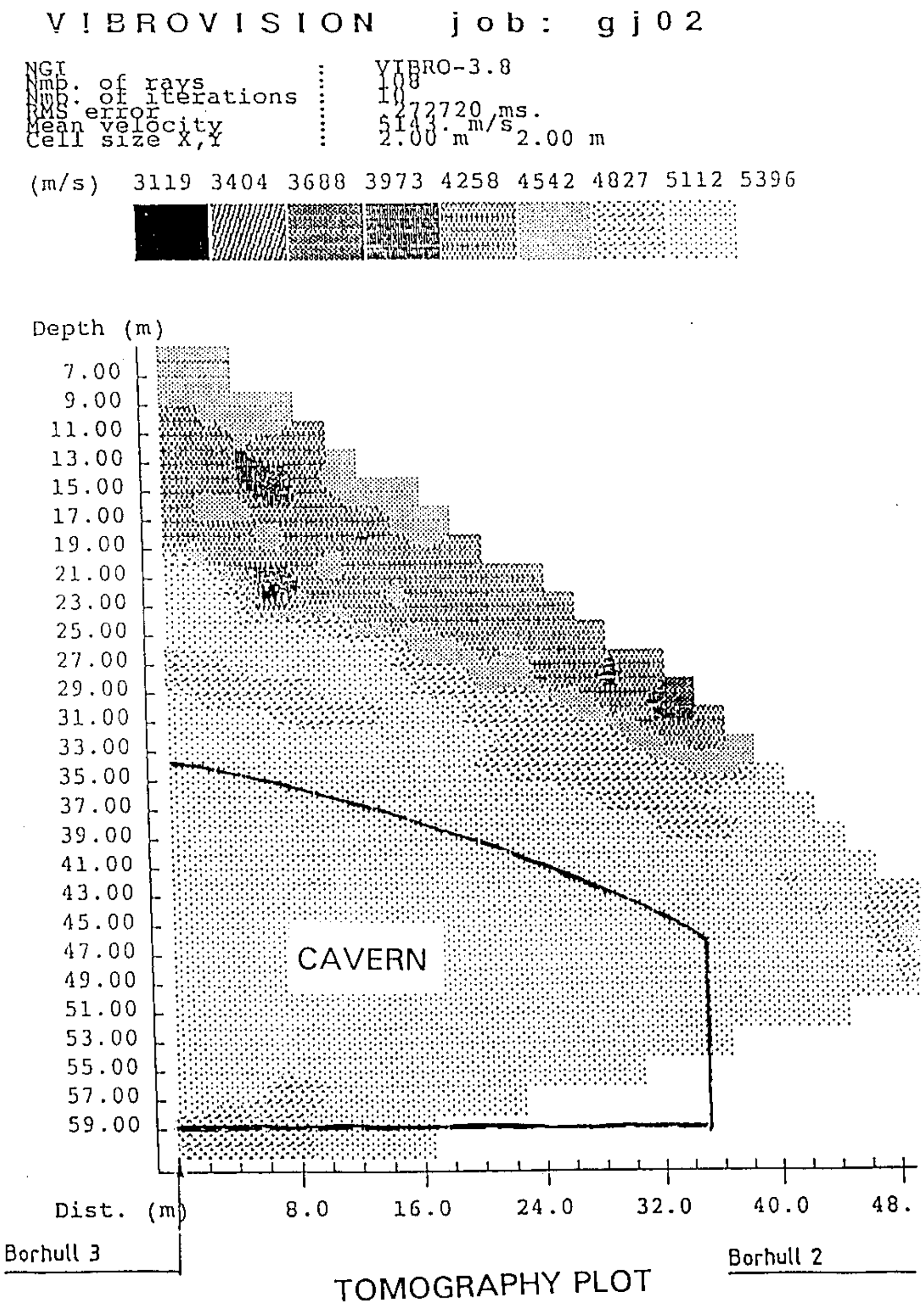
NUMERICAL MODELLING
STRESS DISTRIBUTION

Fig. 4



CORE DRILLINGS
RQD - CLASSIFICATION

Fig. 5



VIBROVISION job: gj02
NGI : VIBRO-3.8
Nmb. of rays : 108
Nmb. of iterations : 10
RMS error : 272720 ms.
Mean velocity : 5143. m/s
Cell size X,Y : 2.00 m 2.00 m
(m/s) 3119 3404 3688 3973 4258 4542 4827 5112 5396
Depth (m)
7.00
9.00
11.00
13.00
15.00
17.00
19.00
21.00
23.00
25.00
27.00
29.00
31.00
33.00
35.00
37.00
39.00
41.00
43.00
45.00
47.00
49.00
51.00
53.00
55.00
57.00
59.00
CAVERN
Dist. (m)
8.0
16.0
24.0
32.0
40.0
48.
Borhull 3
Borhull 2
TOMOGRAPHY PLOT

Fig. 6.

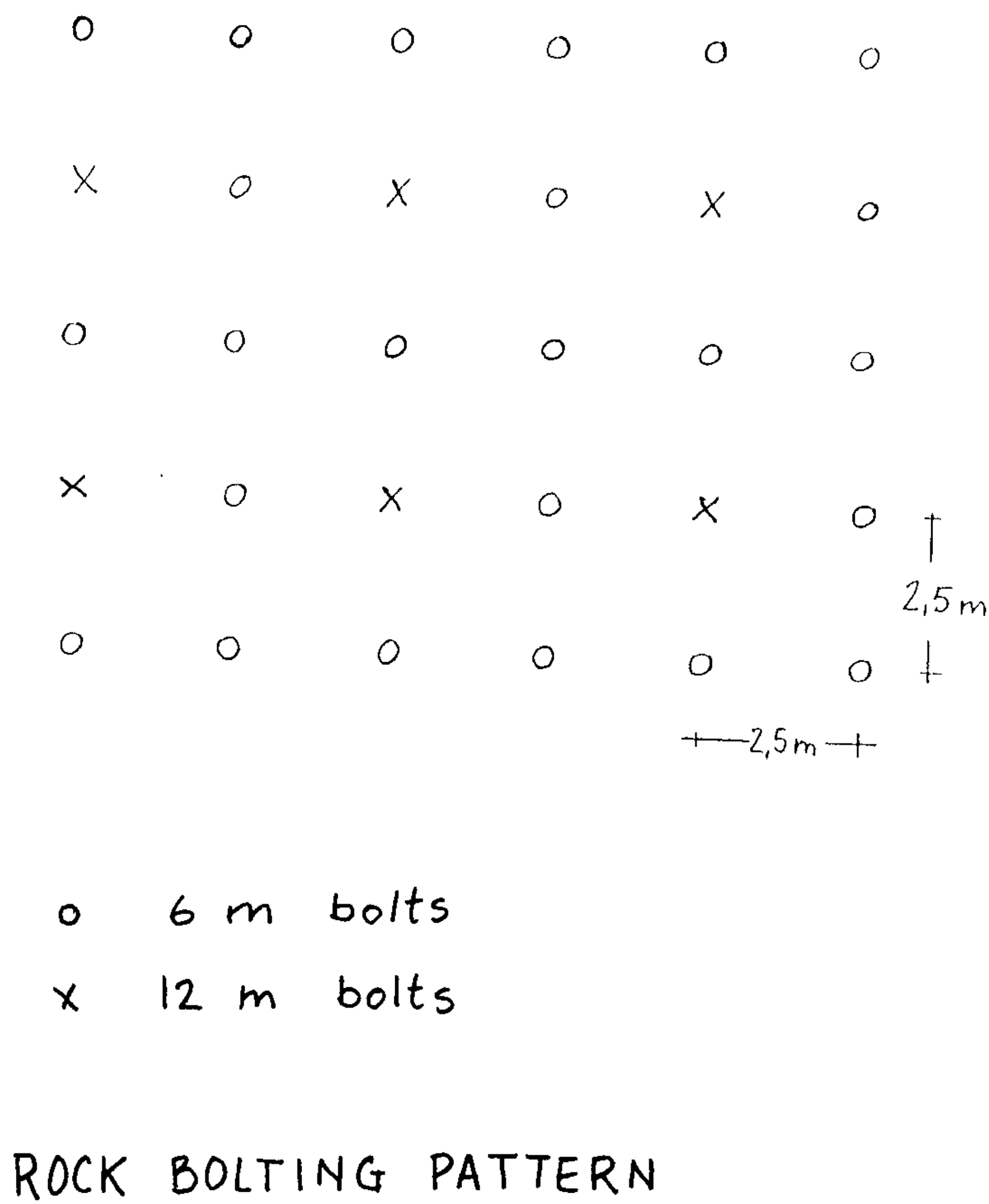
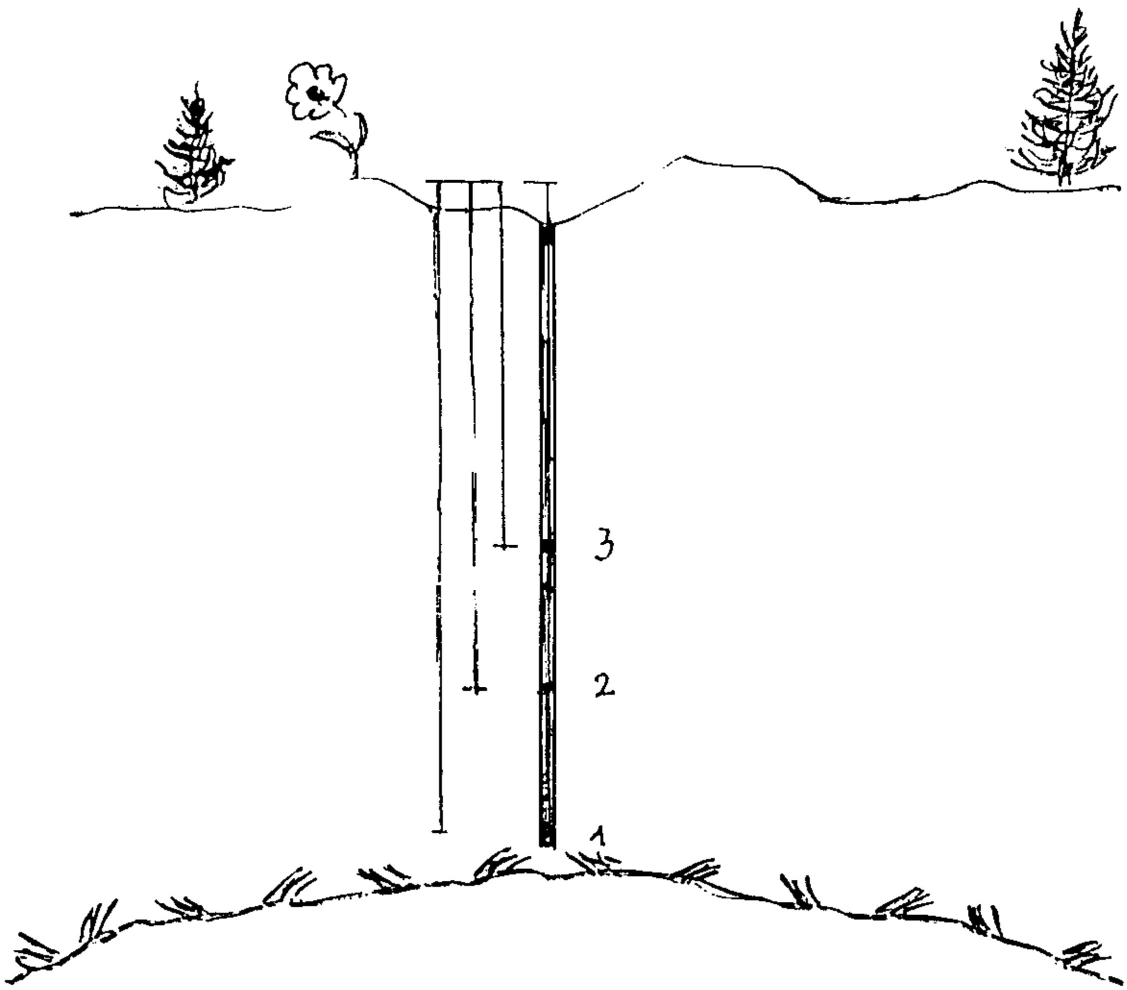
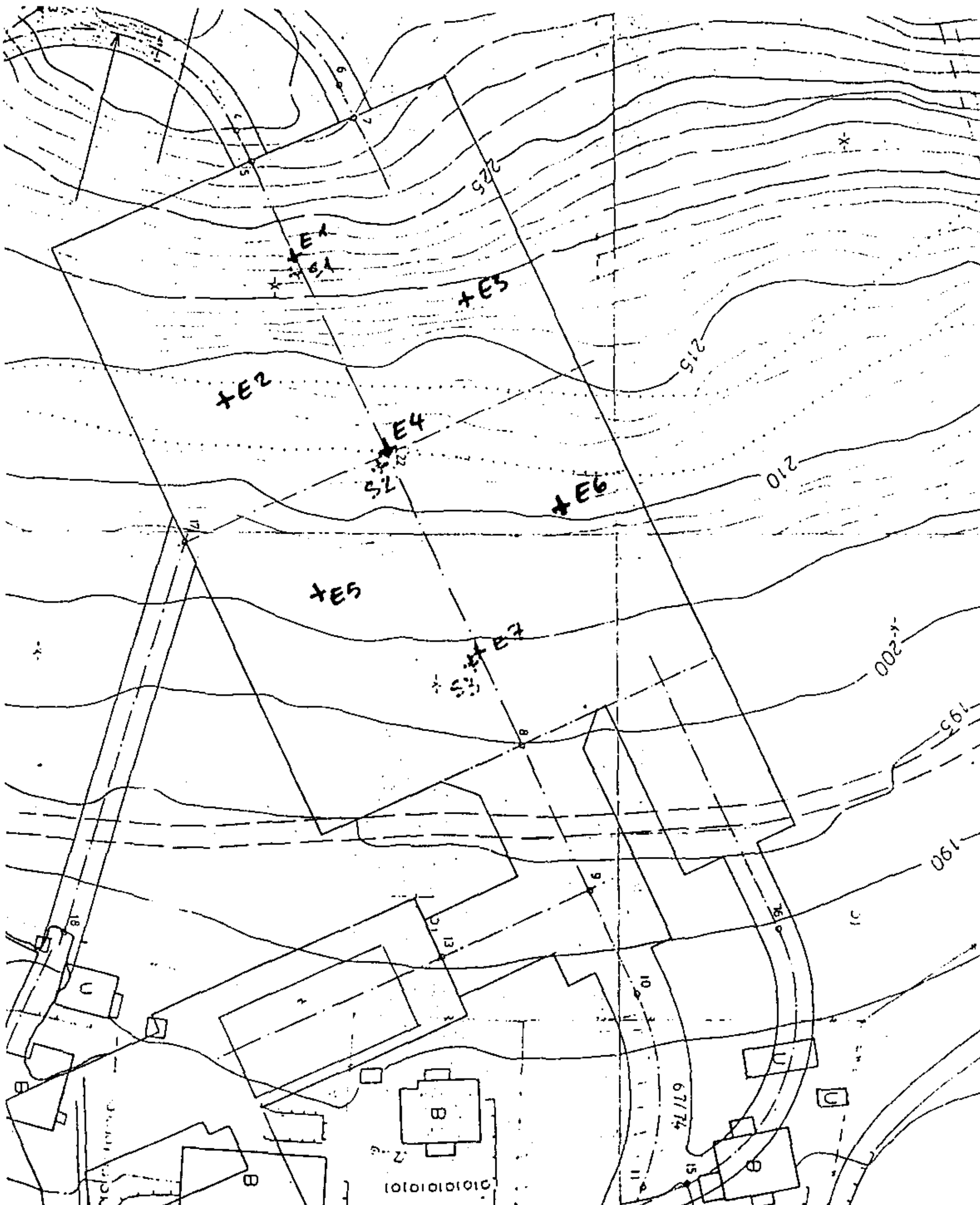


Fig. 7.



EXTENSOMETER

Fig. 8.



LOCATION OF EXTENSOMETERS

Fig. 9

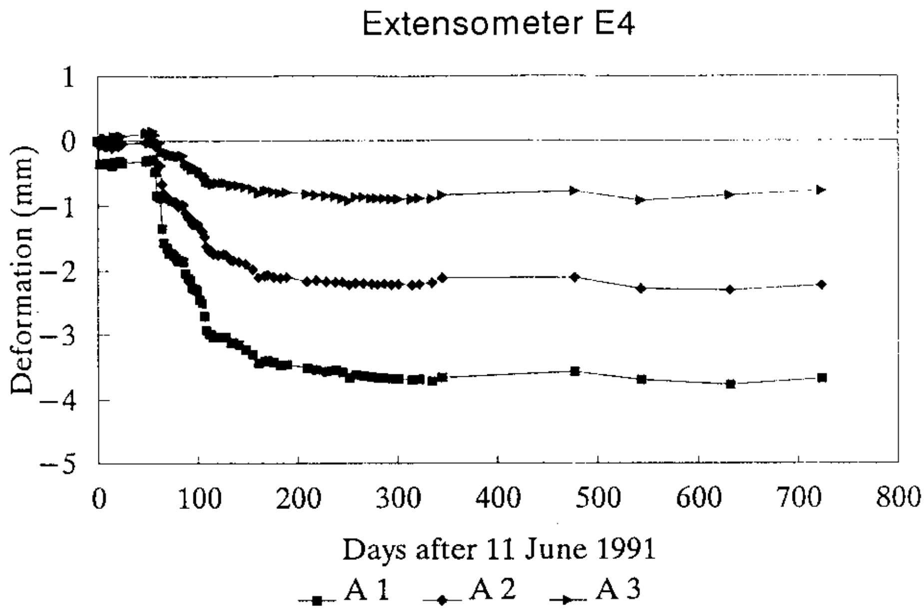

Extensometer E4
Deformation (mm)
1
0
−1
−2
−3
−4
−5
0
100
200
300
400
500
600
700
800
Days after 11 June 1991
A 1
A 2
A 3

Fig. 10

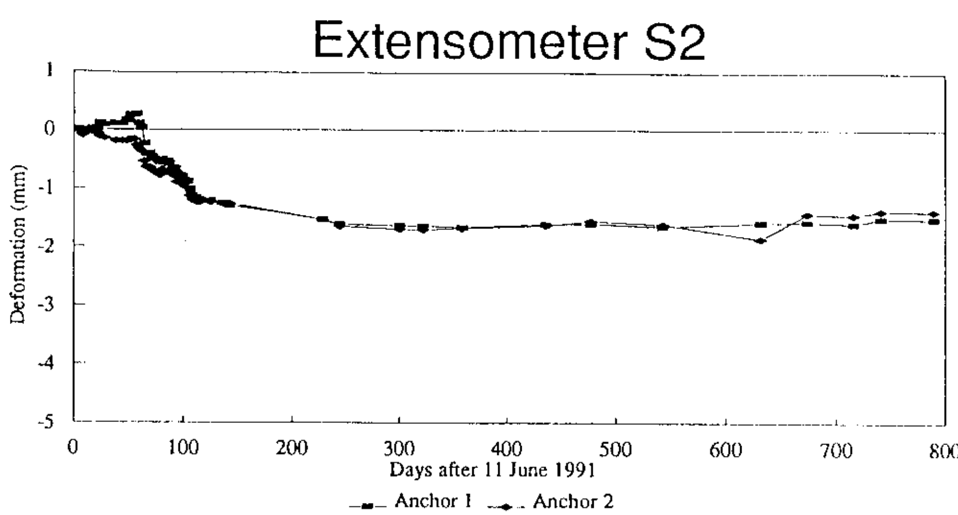

Extensometer S2
Deformation (mm)
1
0
-1
-2
-3
-4
-5
0
100
200
300
400
500
600
700
800
Days after 11 June 1991
Anchor 1
Anchor 2

Fig. 11