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MONITORING THE PERFORMANCE OF A STEEP ROCK SLOPE EXCAVATION

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ABSTRACT

Excavations in a rock mass determine deformations which depend on the nature of the foundations, the local geostructural and geomechanical conditions, the natural slope morphology, geometry of the excavations and time-dependent factors.

This paper focuses attention on a recent experience of a 60 m high subvertical excavation undertaken during costruction of an independent spillway for a gravity dam in Northern Italy where a highly fractured limestone rock mass characterised by a pronounced overturned fold presents very different conditions on the two slopes of the valley.

The complex monitoring system installed permitted control of rock mass behaviour and of original design assumptions. Information gained was used for re-evaluation of local reinforcement required to attain acceptable conditions of safety.

1. INTRODUCTION

Design of large excavations in rock masses requires in depth knowledge of their geological history and mechanical properties.

Present day investigation technology allows determination of the local shear strength and elastic parameters of foundations. Modern computers together with sophisticated powerful software can process the data and give an important contribution to stability assessment.

However, the complexity of rock masses and the necessity, in most practical cases, of limiting the number of tests for economic reasons, makes it extremely useful to control rock mass behaviour and design assumptions by monitoring excavations while they proceed.

The case examined in this paper refers to the design and subsequent excavation of a large rock volume in a steep slope. The implementation of a complex monitoring system allowed systematic control of rock mass deformations behind the excavations.

Knowledge of the deformations was used to:

1. Study elastic behaviour of the rock mass in relation to the local structural and mechanical conditions.
2. Check design assumptions for long term safety factor calculations.
3. Design extra rock reinforcement in sectors where original design assumptions were too optimistic.

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2. GEOGRAPHICAL AND GEOLOGICAL SETTING

The study refers to the excavations for the construction of the Ravedis gravity dam on the river Cellina in the province of Pordenone in Northern Italy.

The dam (Fig. 1) is presently under construction at the foot of the Southern end of the "Prealpi Carniche", a short distance upstream of the Veneto - Friuli plain which accompanies the river to the Adriatic sea north of Venice.

In the site area the rock outcrops are reef facies limestones. In geological times these have been subjected to compressive tectonic forces which have generated structures with very marked folds and faults. The fold axes are directed NE-SW and their direction S-SE. The faults are generally strike-slipping and normal to the fold axes.

At the dam site these structures are very evident. The rocks in the area are of the Jurassic-Cretacic carbonatic series. The limestone strata are regular and vary in thickness between a few centimetres and 2-3 metres. Silty to clayey levels sometimes separate the strata. The intense tectonic activity has caused a high degree of fracturing with spacing of the discontinuities of the order of a centimetre.

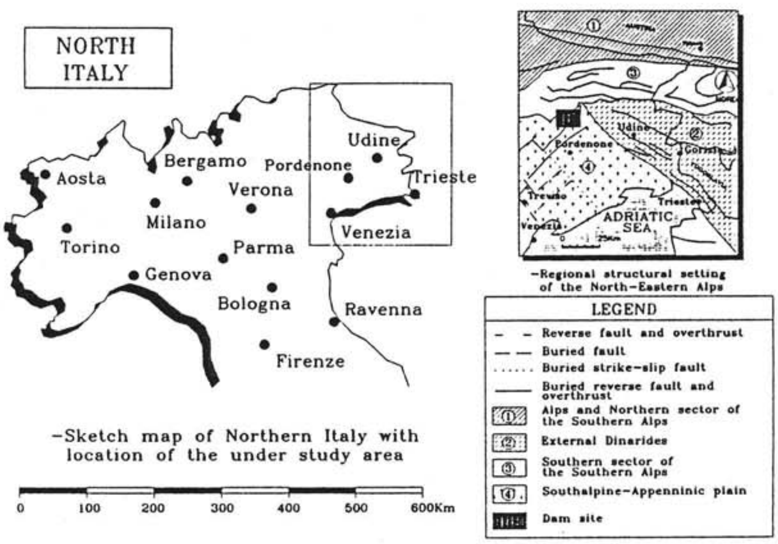

Fig. 1 - Geographical and geological setting

3. GEOSTRUCTURAL AND GEOMECHANICAL INVESTIGATIONS

Numerous geostructural and geomechanical investigations were carried out in the site area (Fig. 2).
These were mainly:

a) geostructural and geomechanical surveys,
b) in-situ shear tests on large cubic rock blocks (50x50x50 cm),
c) deformability tests with cylindrical jack (plate bearing).

The surveys (a) served to collect data on geometry of strata and of other discontinuities (joints, faults) and on rock mechanical features (spacing, opening and roughness of joints, etc.).

Analysis of the data led to the determination of a structural model with an asymmetric overturned anticline (Fig. 3) with NE-SW axis and SE direction (Fig. 2).

Interpretation of the available geostructural and geomechanical data gave knowledge of the principal structural features of the fold which differed according to whether the rock mass was compressed or decompressed (Fig. 3).

The shear tests on the cubic blocks of rock (b) brought to determination of the strength parameters. A total of 10 tests were carried out, of which 8 on the right slope of the valley and 2 on the left. A greater number of tests were performed on the right slope because more critical geomechanical conditions are present on this side (where the foundations are decompressed - see Fig. 3) as resulted even more evident after observation

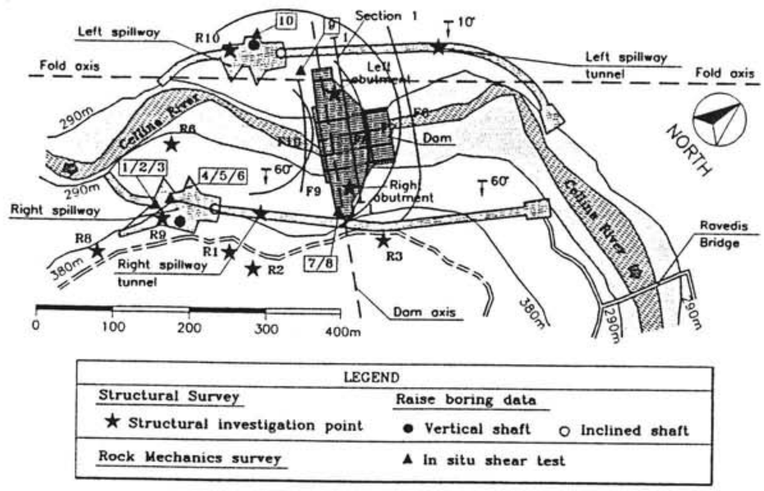

Fig. 2 - General Layout

of displacements.

Shear test data were processed in order to reconstruct the shear strength curves of the rock masses (Fig. 4) according to the Hoek & Brown failure criterion (Hoek & Brown [1]).

Elastic properties were determined through pressure tests (c) with a cylindrical jack in the exploratory tunnels along the dam axis.

The results obtained were as follows:

E_a (right abutment) = 1820 MPa
E_a (left abutment) = 3150 MPa
where
E_a = average elastic modulus of rock foundations.

Examination of results from investigations a), b), c) together with information obtained through geophysical surveys and analysis of large diameter drilling data (with raise borer) showed that:

1) the rock masses are generally very fractured and their strength and elastic modulus is not high;
2) due to the different structural conditions, the strength and elastic parameters of the foundations are much higher on the left side of the valley than on the right;
3) the rock masses on the right bank give evident signs of decompression caused mainly by strata dip which favours relaxation through flexural toppling.

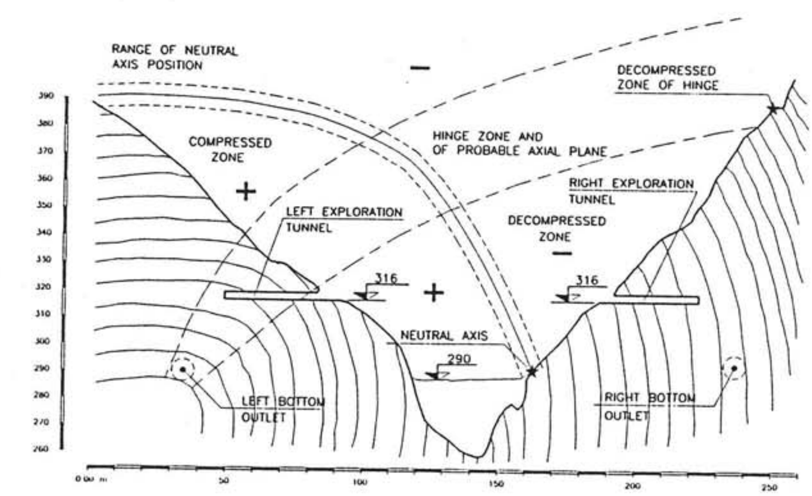

Fig. 3 - Possible interpretation of the parallel flexural fold - cross section in axis of dam

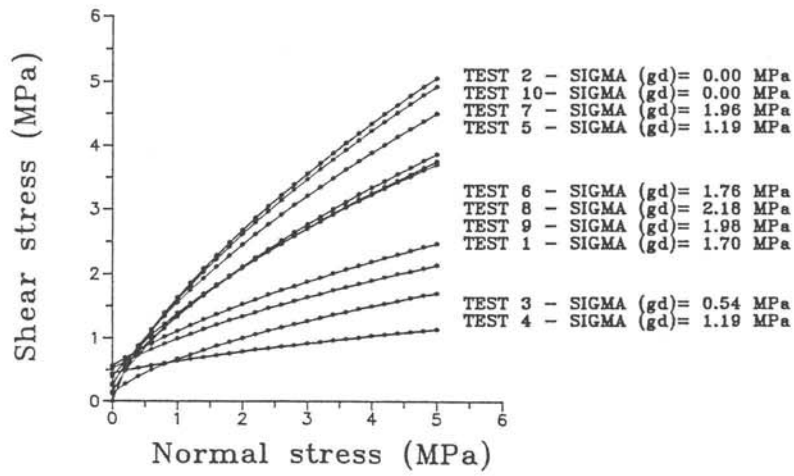

Fig. 4 - In situ shear test rock mass strength

4. MONITORING EXCAVATIONS OF THE RIGHT SPILLWAY

The spillway on the right slope is a gateless W-shaped weir, independent from the dam, connected laterally to the spillway channel which leads to the discharge tunnel (Fig. 5).

Construction of the weir and spillway channel required removal of large rock volumes between el. 400 m a.s.l. above the Old Cellina Road and el. 326 m a.s.l. where the concrete structure was to be founded. The natural slope of the valley in this area was about 43° above horizontal.

Excavation was designed so as to leave a systematically dented profile formed by a succession of 10-12 m high vertical rock walls and 3 m wide horizontal berms (see Fig. 6). The average slope after excavation was about 68° above horizontal (Photo 1).

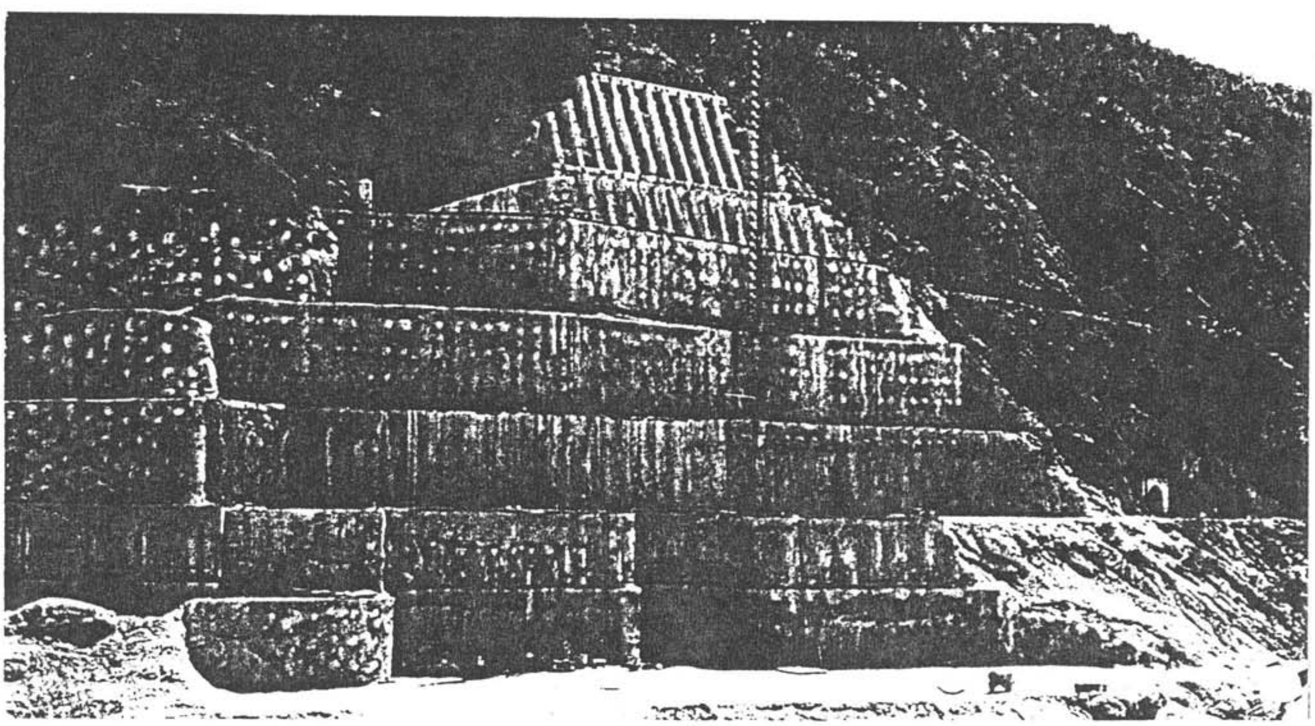

Photo 1 - General view of excavations for construction of the right spillway

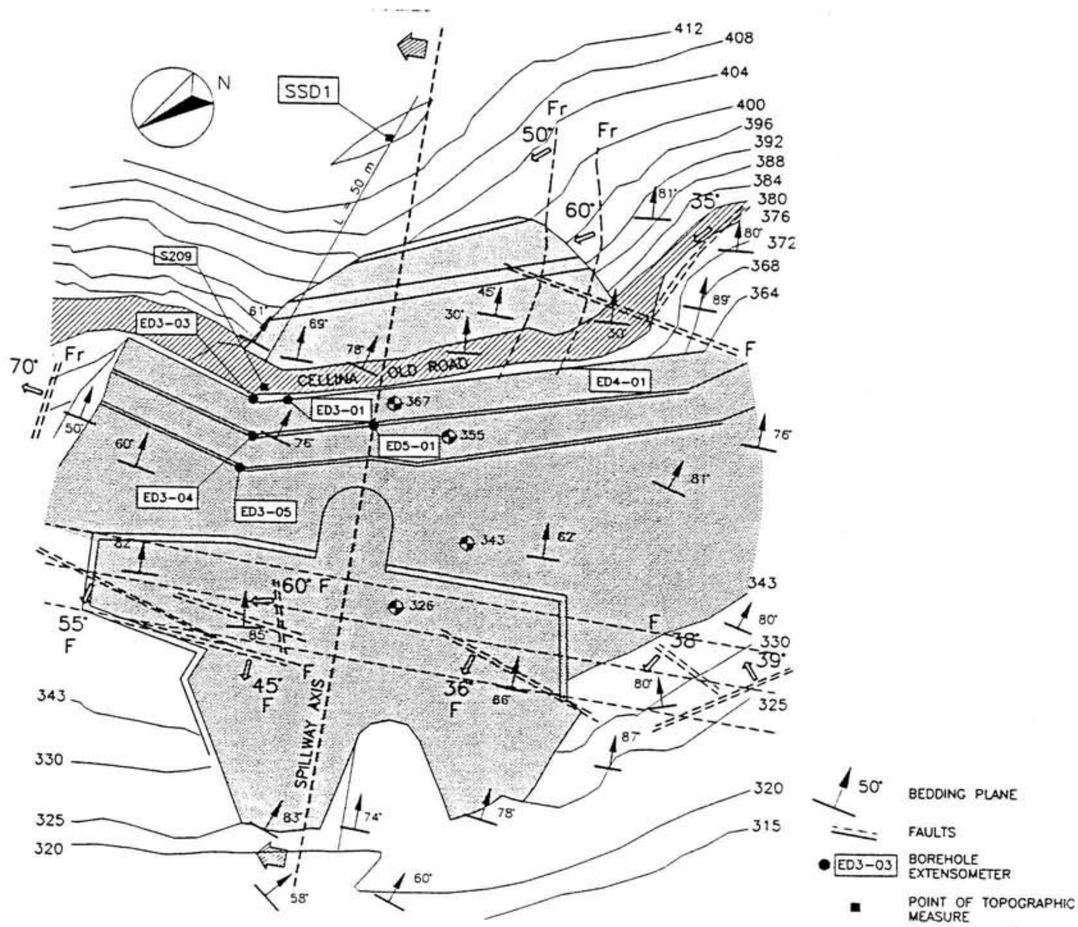

Fig. 5 - Right spillway - geostructural and excavation map

Operation proceeded from above, starting from the Old Cellina Road at el. 380 m a.s.l., with a step-by-step repeated procedure consisting of excavation to a horizontal surface at 3-4 m depth followed by rock reinforcement with post tensioned anchor rods to contrast deformations caused by variations of the state of stress.

The following assumptions were made for initial slope stability assessments with the method of slices (Lunardi et al. [2]):

1) homogeneous, isotropic and continuous rock mass, with discontinuities tangent to the slip surface;
2) seismic pseudo-static forces in accordance with Italian standards (1st category classification);
3) rock reinforcement simulated with improved mechanical properties in the zones involved (Fig. 6).

The minimum acceptable safety factor was 1.3 (in accordance with Italian standards).

In order to control deformations of the rock mass both during construction and during operation of the reservoir, both external (optical) and internal (extensometer) monitoring tools were used:

Surface Control (optical system)

Periodic measurements of distances between pre-established points (Fig. 7) were made with high-precision topographical instruments. The points controlled were:

- S207, el. 327.96 m a.s.l., right abutment of dam
- S209, el. 381.25 m a.s.l., right spillway
- SSD1, el. 408.00 m a.s.l., right spillway
- S208, el. 363.55 m a.s.l., left abutment of dam
- S210, el. 378.32 m a.s.l., left spillway
- S202, el. 327.96 m a.s.l., left slope upstream of structures

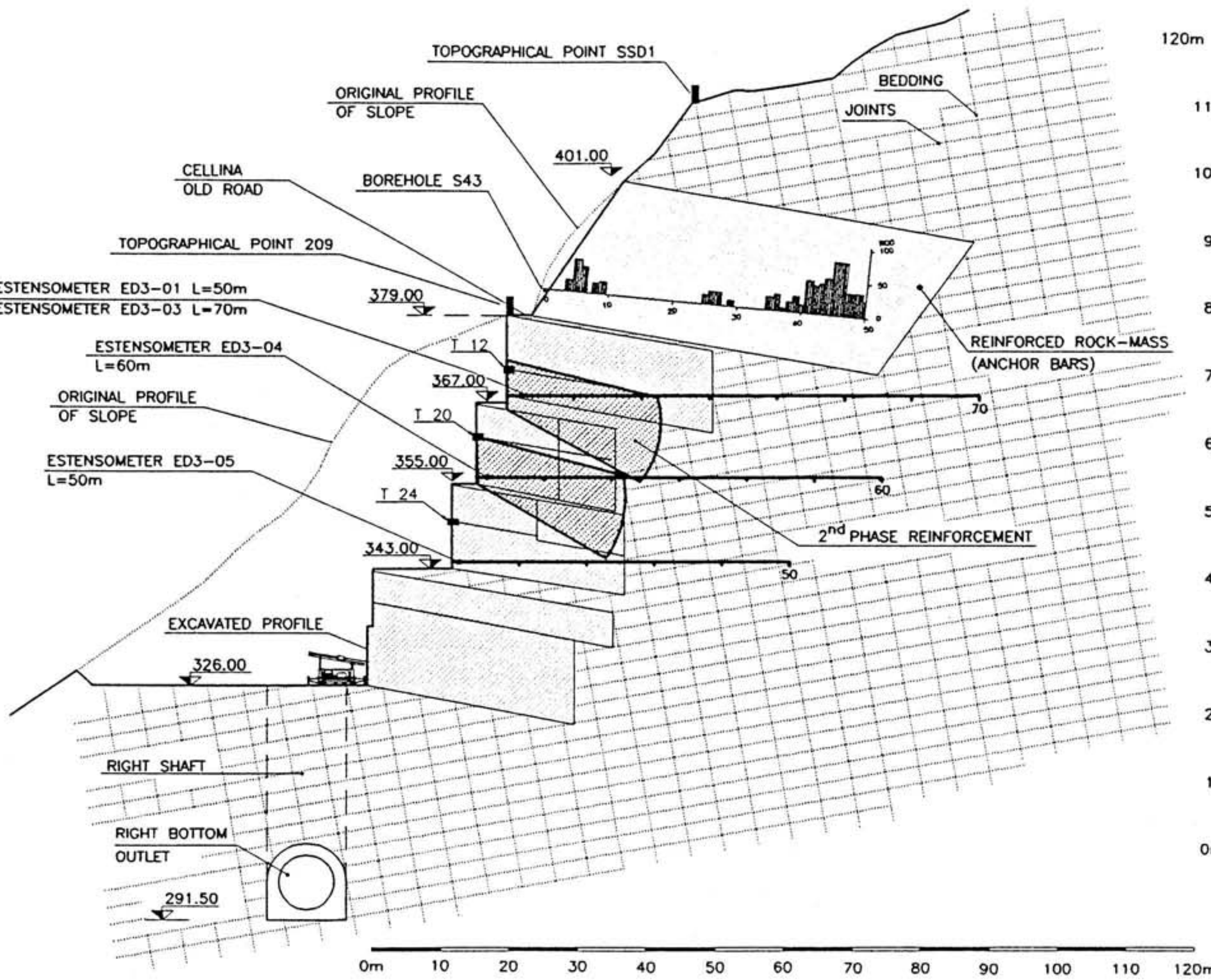

Fig. 6 - Cross - section in axis of the spillway

Internal Control (extensometers)

In depth deformations were controlled after installation of 6 multi-base extensometers connected to the main data processing unit:

- ED3-01, el. 367 m a.s.l., L = 50 m, 5 bases
- ED3-03, el. 367 m a.s.l., L = 60 m, 6 bases
- ED3-04, el. 355 m a.s.l., L = 60 m, 6 bases
- ED3-05, el. 343 m a.s.l., L = 50 m, 5 bases
- ED4-01, el. 367 m a.s.l., L = 50 m, 5 bases
- ED5-01, el. 355 m a.s.l., L = 70 m, 6 bases

The extensometers were installed in subsequently grouted 50-70 m long sub-horizontal boreholes.

Extensometer heads were located on the excavated rock wall surface.

The systems included:

- invar rods with anchor base at extremity
- sensors with analogic signalling
- analogic to digital conversion unit
- data processing unit.

Recording of surface displacements and internal deformations started in June 1989. In this paper, rock movements until June 1992 will be considered. This period covers approximately the duration of excavations (February 1989 - June 1991). Deformations which took place later are negligible.

Four different phases can be distinguished:

- PHASE 1: from the beginning of works (February '89 until June '89 on the Fig. 8) until excavations were suspended at el. 349 m a.s.l. due to excessive displacements (April '90);

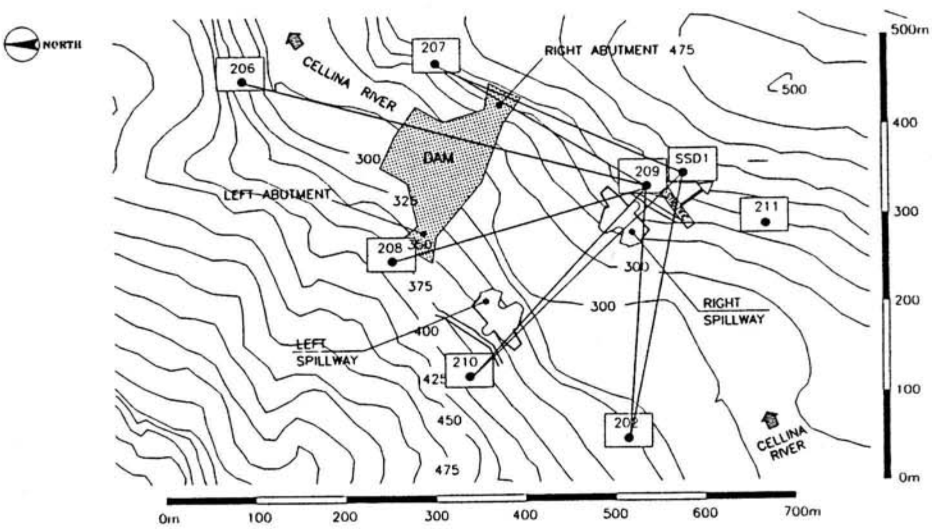

Fig. 7 - Topographical network

- PHASE 2: during suspension of excavations (April '90 until January '91), including the period while extra rock reinforcement was being implemented between el. 370 and 355 m a.s.l. (October '90 until January '91);
- PHASE 3: period of completion of excavations from el, 349 m a.s.l. to final el. 326 m a.s.l. (February '91 until June '91);
- PHASE 4: one year period (June '91 - June '92) immediately after excavations were completed.
- The recorded movements are illustrated in Figs. 8 and 9.

Diagrams a) and b) of Fig. 8 show the displacements measured with the optical topographical instruments from points 202 / 207 / 210 to point SSD1 and from point 209 to points 202 / 207 / 208 / 210.

Diagrams c) and d) of Fig. 8 and a), b), c), d) of Fig. 9 show the displacements recorded with each base of the 6 extensometers.

The diagrams of Fig. 8 (optical instruments and extensometers ED3-01 and ED4-01) cover almost the entire period of the four phases. Those of Fig. 9 refer to instruments installed at levels reached by the excavations in phase 3 and, therefore, cover only the end period.

Examination of development of the deformations (Table 1) brings to the following considerations:

- PHASE 1: rapid deformations caused by excavations. Larger deformations were recorded near the spillway axis as shown by the different values recorded at ED3-01 (~ 5 mm) and at ED4-01 (~ 0.2 mm); optical measurements (diagram 8a) confirm the value recorded and the direction of movement (normal to the excavated wall and to rock strata bedding). The larger deformations observed at ED3-01 are also due to the morphology of the excavations, which are externally convex (Fig. 6). Initially the rock wall displacement at ED3-01 progressed at the rate of 1 mm/month and reached ~ 2 mm/month towards the end of this phase; the displacement was found to be directly correlated to the depth of excavation (Fig. 10): 1 mm displacement for every 3 m excavation.
- PHASE 2: suspension of excavations greatly decreased rate of displacement. Extensometer ED3-01 recorded a surface displacement of ~ 1.6 mm and ED4-01 of 0.6 mm. Optical measurements show a continuation of displacements which, however, come within the range of error of the instruments. In this phase surface displacements are roughly 0.1 mm/month and tend to decrease.
- PHASE 3: with excavations proceeding, new deformations develop but are less important than in phase 1 for two reasons: geometry of excavation surface and greater distance (downwards) of volumes removed. Surface displacement ad ED3-01 is of about 3 mm and at ED4-01 of 0.6 mm. The other extensometers installed during this phase at different elevations show varying values of displacement (maximum of 3.1 mm at ED3-05). The average rate of displacement recorded at ED3-01 is 0.6 mm/month and 0.2 mm for every 3 m (in depth) excavated (Fig. 10), 5 times less than in phase 1; the rate of displacement of the extensometers closer to the excavation works (e.g. ED3-05 at el. 343 m a.s.l.) is greater than 1 mm/month.

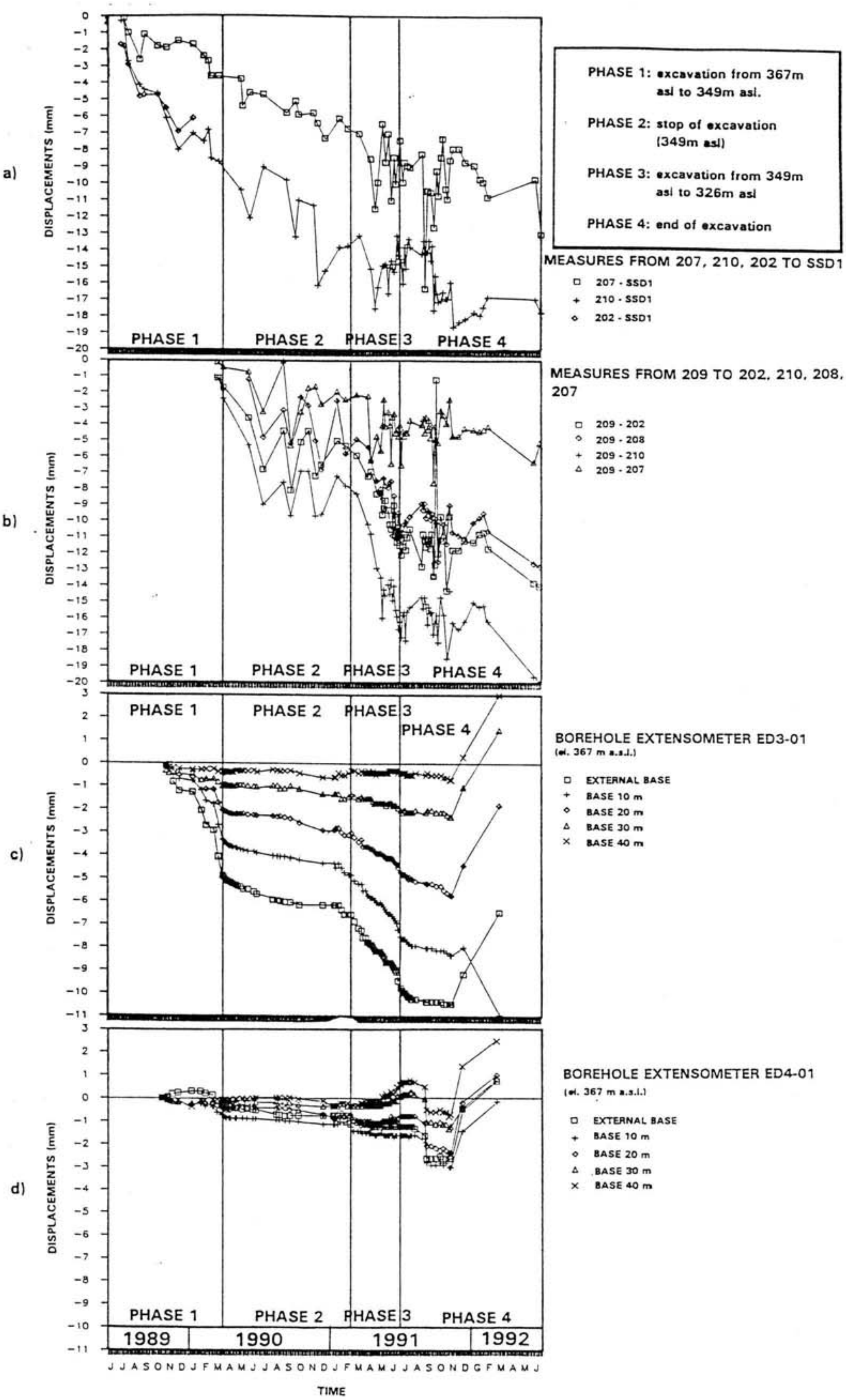

Fig. 8 - Time-displacement diagrams

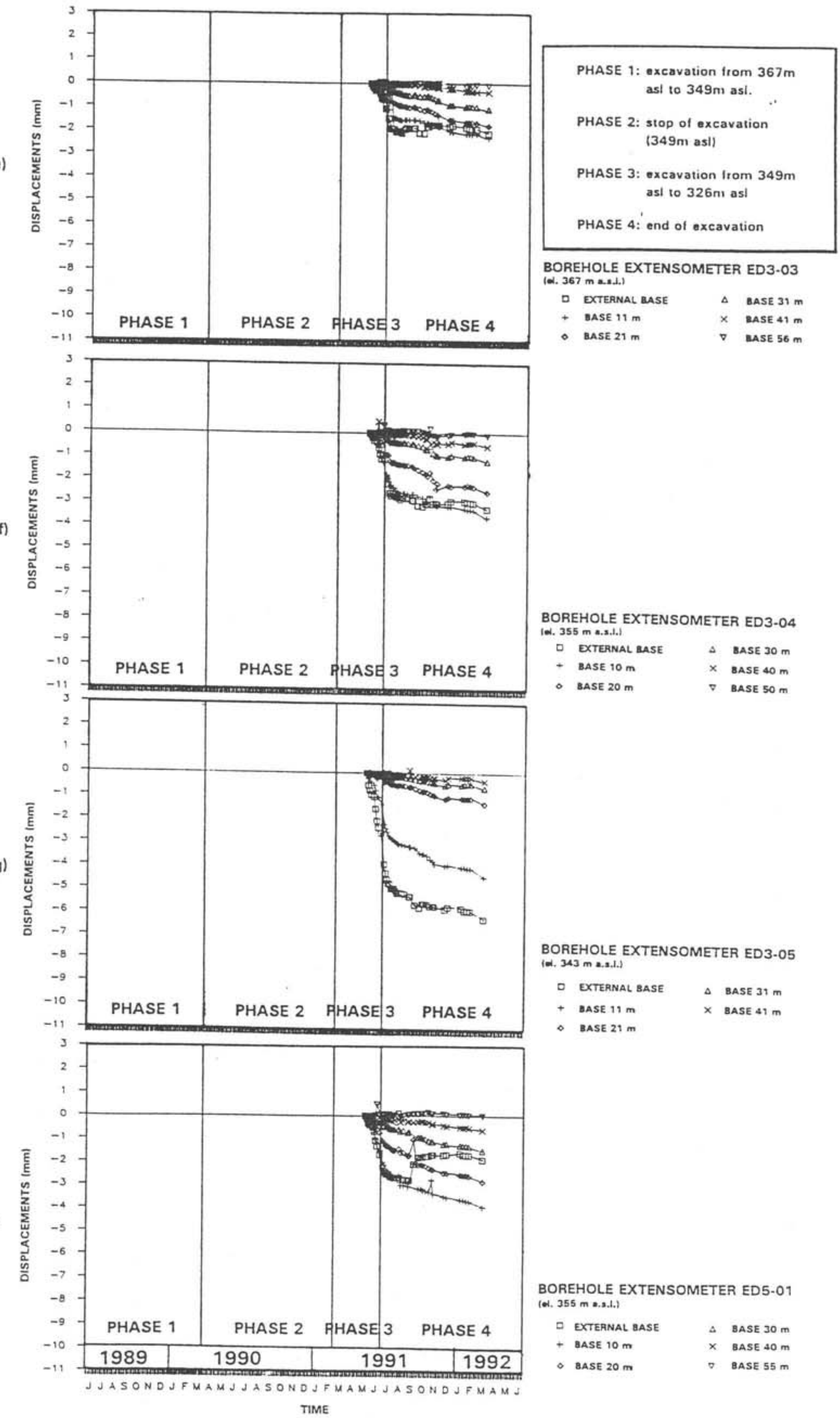

Fig. 9 - Time-displacement diagrams

Phase	Time	Works	Topographical control							Extensometers					
			209 to				210 207 202			ED3-01	ED3-03	ED3-04	ED3-05	ED4-01	ED5-01
			202	210	208	207	to SSD1			367m	367m	355m	343m	367m	355m
1	june89-march90	excavation: 367-349m	1.7	2.5	1.2	0.5	9.2	3.6	6.1	5,00	/	/	/	0,20	/
2	march90-february91	stop excavation (349m)	3.6	5.2	4.2	1.7	4.7	3.1	/	1,60	/	/	/	0,60	/
3	february91-june91	excavation: 349-326m	5.7	8.5	3.9	2.1	0.7	1.8	/	3,10	1,10	1,20	3,10	0,60	1,90
4	june91-june92	end of excavation	2.7	3.2	3.2	0.7	3.4	4.6	/	0,9 / -3,2	1,00	2,00	3,20	1,3 / -2	2,00
		overall displacement:	13.7	19.4	12.5	5.0	18	13.1	/	10,6 / 6,5	2,10	3,20	6,30	2,7 / -0,6	3,90

Table 1

- PHASE 4: monitoring movements in the one-year period after completion of excavations showed a pronounced and progressive slow-down of displacements, although varying in absolute value at the different extensometers. The anomalous values recorded at ED3-01 and ED4-01 between November '91 and March '92 can be attributed to electric problems with the transducers.

The more interesting and complete information was obtained from extensometer ED3-01 and from the optical measurements. Displacements could be correlated very well with depth of excavation as can be seen in Fig. 10. The optical measurements show greater absolute values than extensometer recordings for two reasons: observations began at an earlier date and, also, these take into account the (less important) movements which were taking place on the left side of the valley during excavations of the left spillway. The lower gradient observed with the extensometer in phase 3 is very evident and is explained above all by the changing geometry (from convex outwards to concave - see Fig. 5).

As mentioned, in April 1990 excavations were suspended after observation of the deformations. Shear tests had shown that failure occurred on the test blocks after displacements of some millimeters. Optical measurements during phase 1 (points 202, 210) showed displacements of the order of 1 centimetre and, therefore, high in relation to the stress-strain failure curve of the rock mass in this area as determined during investigations. The critical conditions of the slope had previously been shown by the slide of a certain volume of highly fractured rock (as proved by RQD values) between el. 400 m a.s.l. and 380 m a.s.l. during excavations.

The above considerations conducted to the decision of suspending excavations in order to re-evaluate the long term slope stability conditions. Six new-shears tests were conducted in this particular area during suspension of excavations to collect the necessary information and verify the local shear strength.

A finite element analysis with an elastic-plastic model (mesh of which is illustrated in Fig. 11) determined the more critical sectors within the rock mass. Critical sectors with negative stresses were found in the zone between elevations 370 and 355, not far from the foot of the excavations which had taken place up to the date of suspension.

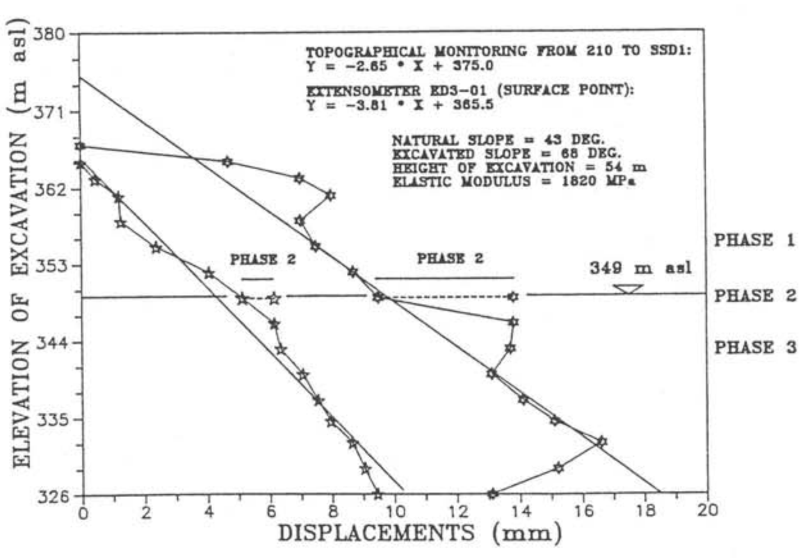

Fig. 10 - Rate of displacements during excavations

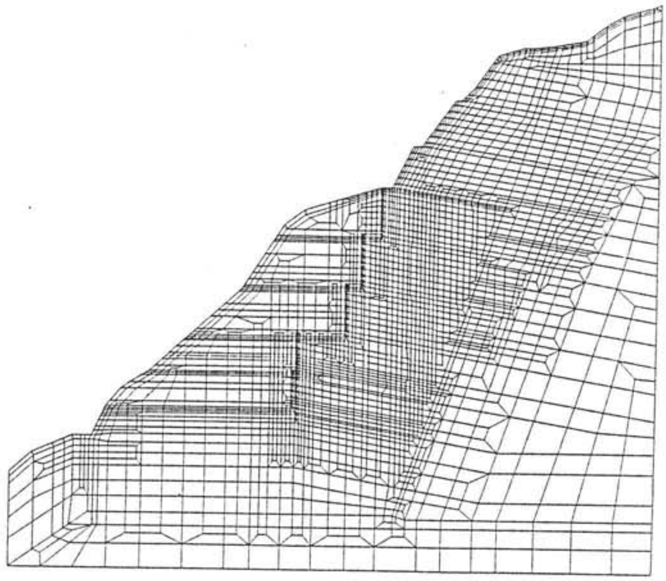

Fig. 11 - Finite element mesh for modelling excavation of right spillway

Actual displacement confirmed the elastic modulus determined in the previous tests.

On the basis of the finite element calculations, extra rock reinforcement (see Fig. 6) was designed and implemented so as to re-establish conditions with acceptable safety factor of 1.3 which was cross checked with the method of slices.

5. CONCLUSIONS

Optical measurements of displacements and use of extensometers to monitor internal deformations are without doubt important tools for controlling excavations in rock.

In the case of the right spillway channel of Ravedis dam, monitoring of deformations permitted to:

a) record development of surface displacements starting from beginning of excavation works (topographical control);
b) record development of deformations within the rock mass after installations of extensometers (internal instrumentation);
c) analyse the influence of strata bedding and observe that principal direction of movement is perpendicular;
d) determine sectors which are in a more critical stress-strain condition;
e) record time related phenomena (creep) typical of "flexural-toppling" conditions.

The information gained led to important decisions involving design requirements for safe continuation of excavations.

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