The FES rock mass model – Part 2: Some examples

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SUMMARY

The mathematical background of the FES (Fissured, Elastic, Saturated) rock mass model was recently presented in *Dam Engineering* (Volume III, Issue 1). In this second part of the paper some examples of its use will be presented. First of all, the case history of the very special events at the Zeuzier dam in Switzerland, which were in fact the trigger of the development of the said model. A number of further possible uses of FES will also be shown.

Introduction

FES has proved to be a useful model to simulate the structural behaviour of a fissured, elastic, saturated rock mass. It leads to a nonlinear elastic relationship between strain and stress, taking additionally into account the influence of the interstitial so called neutral water pressure, which acts in the discontinuities of the rock mass.

As with all models in any field of technology, FES has also its limitations not to be overlooked. Particularly, it excludes any sliding along the discontinuities, as such sliding would change the fitting of the surfaces of the two rock blocks in contact and would modify the characteristic function of the closing of the fissures. This may even produce a dilatancy of the rock mass.

FES should therefore primarily be used to analyze limited changes of the stress-strain state starting from a situation of equilibrium.

A special field of interest is the study of the variations of the neutral water pressure and their influence of the stress-strain field. However, other types of problems can be delt with using this model.

As in any structural or rock mechanic analysis, the results of the computations have only a theoretical aspect unless the main parameters are confirmed by field and laboratory investigations or tests.

The knowledge of the following main parameters is required:

- the modulus of elasticity of the rock material (as well as its Poisson ratio)
- the frequency of the discontinuities, which is clearly related to the RQD values of the rock-mass, and
- the shape at the openings along the surfaces of the discontinuity. Indirectly, this shape may be obtained, for example, from loading tests on the rock-mass itself.

Back analysis of any change in the stress-strain field or in the pressure, porosity and permeability field occurring under a load in a more or less extended rock-mass may be used to define or approximate the above parameters in order to introduce them into the computation.

1 The Zeuzier case history

1.1 The events

The 156 m-high, double curvature arch dam of Zeuzier, near Sion in the south-western part of Switzerland, was built in the years 1954 to 1957 (see Figures 1 and 2). It behaved since then in a very satisfactory, mainly elastic way up to the end of the year 1978, when unexpected continuously increasing deflections towards upstream were detected.

The dam closes a deep gorge cut in a limestone anticline of the Malm-formation. Below the Malm and separated from it by a thin, but quite impervious layer of the Callovian-Oxford series, one finds the more pervious and thicker formation of the Dogger as shown in Figure 3. The rock formations below the Dogger are not of interest in this context.

Upstream of the rock ridge on the reservoir bottom, the relatively pervious Malm limestones are covered by impervious marl layers. The Malm formation crops out towards downstream and can therefore be considered as being drained by the valley and non saturated. In contrast the Dogger marly limestones form a confined aquifer; the Oxford series being its upper aquiclude.

During the Winter 1978/79 the upstreamwards displacements of the arch continued steadily while the water level was drawn down due to the normal operation of the powerplant. Figure 4 shows, as an example, the displacement of a point at the crest of the dam as detected by the pendulum line located in the valley axis. While the value on 31 October 1978 still remained inside the envelop of former years, the displacement on 6 December was clearly outside this limit; the movement was continuing quite fast after that date.

As soon as the snow had melted, a careful geodetic survey became possible in the early summer of 1979; it showed a clear settlement of the whole region around the dam, as well as a continuous narrowing of the valley. The order of completely emptying the reservoir was then given by the Swiss Federal Authorities.

Since summer 1976 an investigation adit for a possible future highway tunnel was under excavation at a lower elevation some distance from the dam. After having investigated a number of possible reasons for the settlements, the existence of a correlation between the movements of the dam and the quite important water inflows in the adit was suspected; at that moment the excavation works for the adit were stopped by order of the authorities.

The deformations at the dam site were continuing but progressively slowing down. It took nevertheless about 6 to 8 years for the settlements to get to a practical end. The total settlement at the dam site had reached finally about 13 cm, the narrowing of the valley about 7.5 cm and the deflection of the dam towards upstream approximately 12.5 cm. Figure 5 shows the settlements of two points at the abutments of the dam's crest.

The unreinforced concrete of the arch, as a brittle material, could obviously not resist such big deformations without cracking. Figure 6 shows the cracks, up to 15 mm wide, which formed along the downstream foundation line as well as the opening of the contraction joints between the concrete blocks at the upstream face.

After the settlements had stopped, the cracks in the dam body were grouted with epoxy resin and the reservoir was reimponded in six yearly stages. Since September 1988 the reservoir and the powerplant have been operating without any limitation; the behaviour of the dam being absolutely satisfactory. It shows only elastic deformations in the same magnitude as before the settlements had occurred.

The main problem was thus to explain the quite extraordinary events which occurred since autumn of 1978. It was however clear from the beginning that the thin arch dam only can react to the movements of the abutments but in no way resist them. The reason of the damages of the dam was therefore obviously to be found in the settlements of its rock foundation.

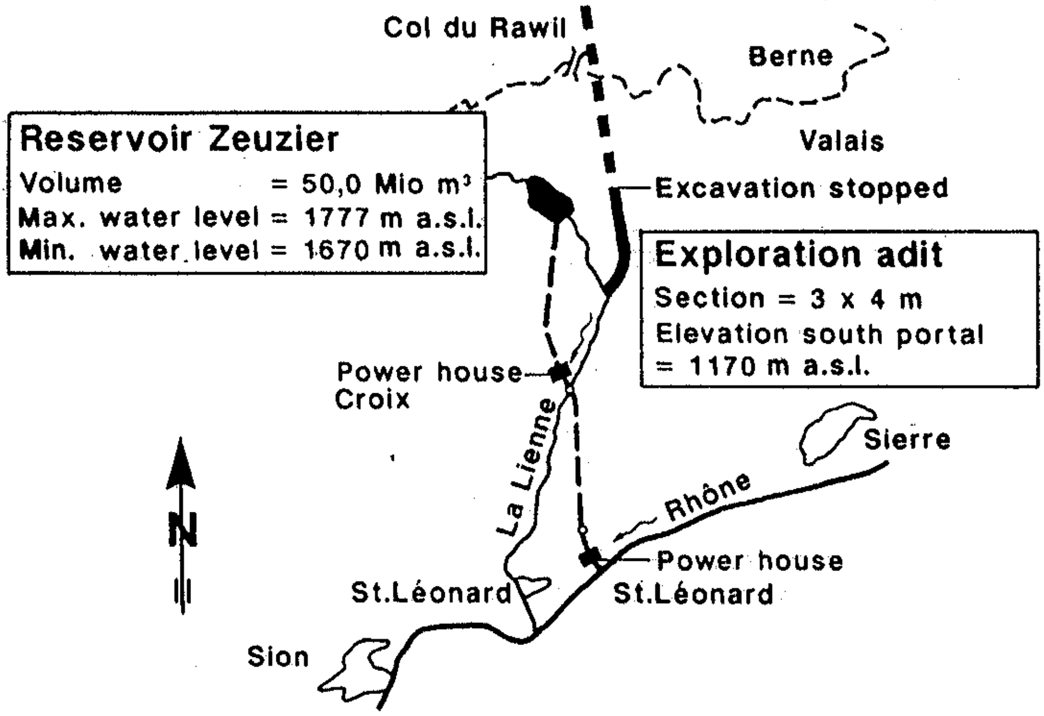

Figure 1. General situation of the powerplant La Lienne with reservoir Zeuzier and investigation adit for the planned Rawil highway tunnel

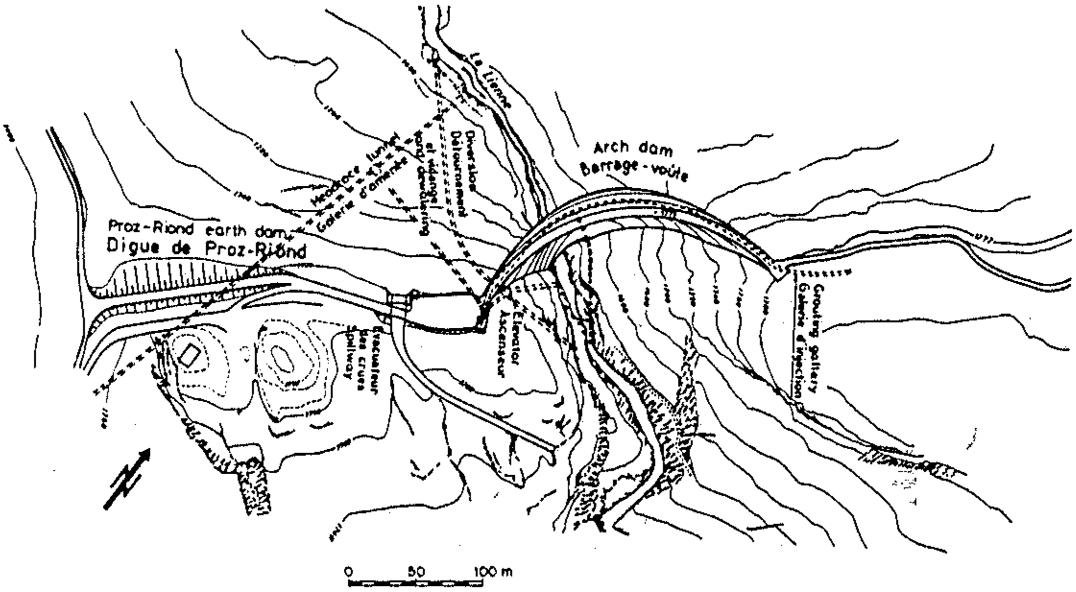

Figure 2. Plan view of the Zeuzier arch dam

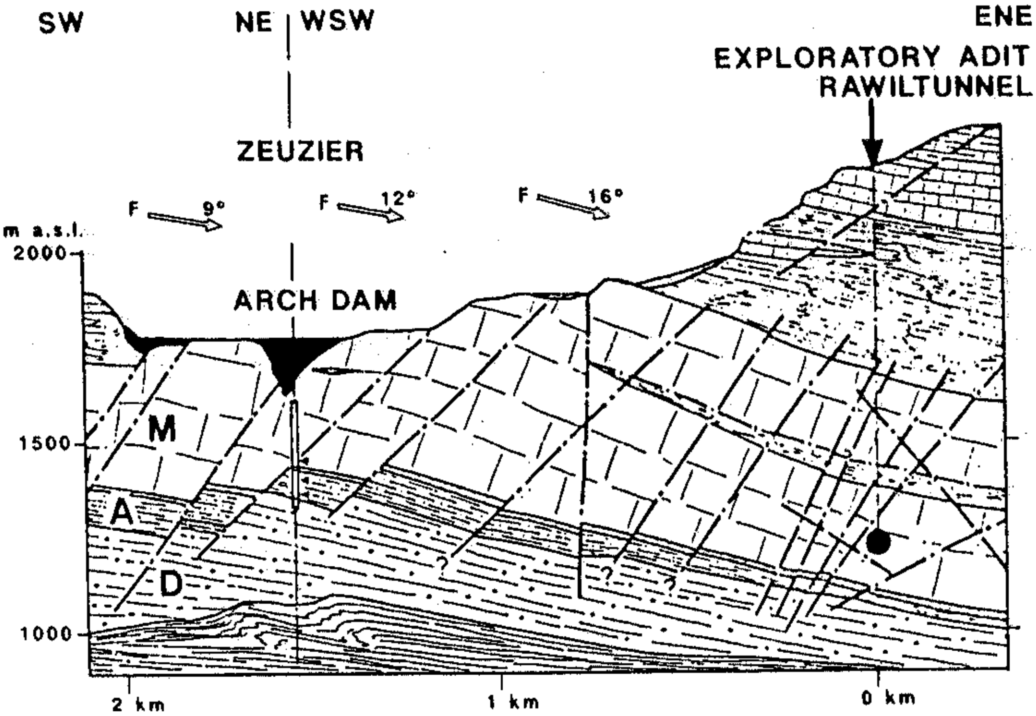

Figure 3. Geological profile through the rock bar of Zeuzier extended to the exploratory adit (by Dr T. Schneider). The dam is founded on the Malm limestone (M), below which is the Dogger formation (D). The two are separated by the Callovian-Oxford series (A)

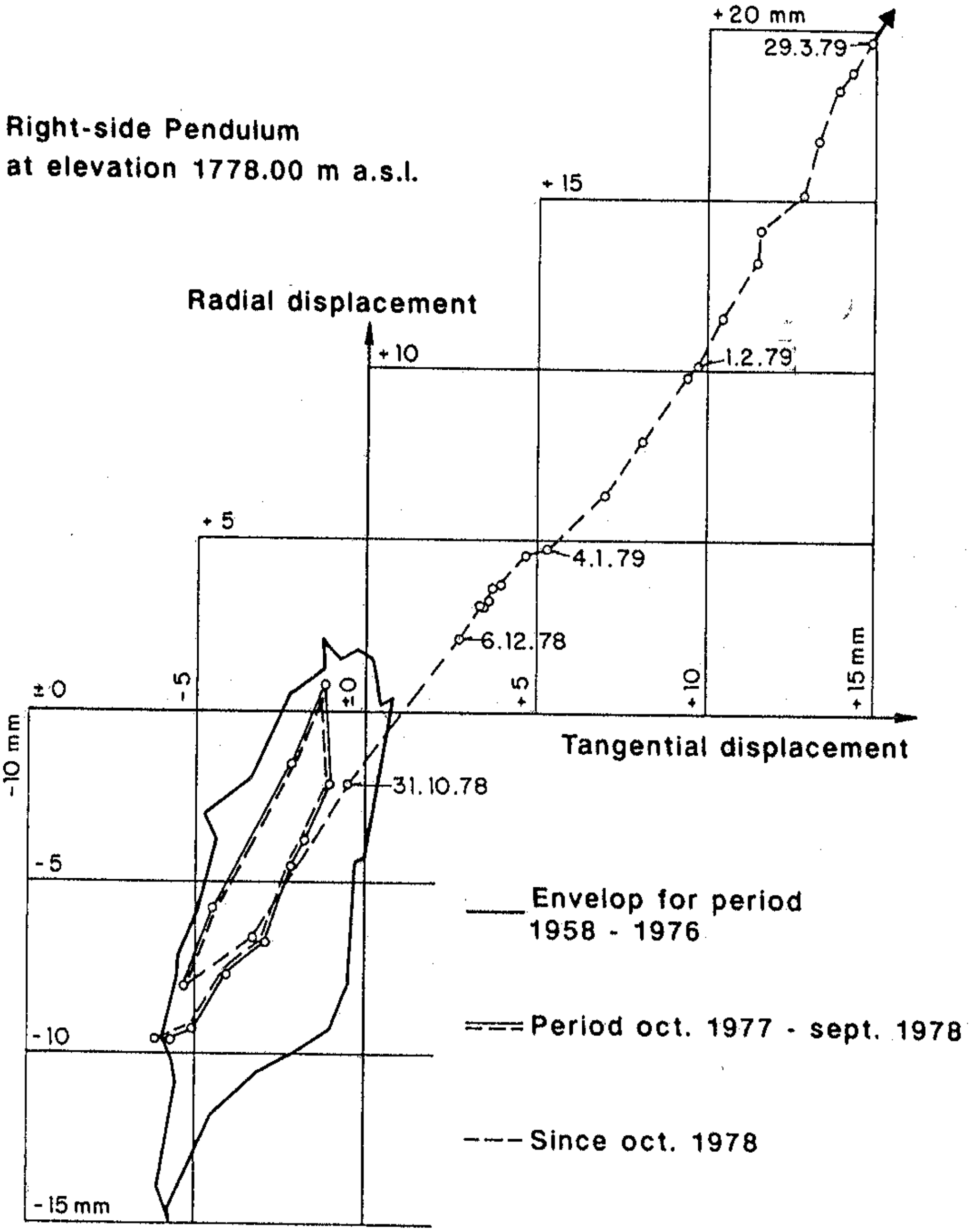

Figure 4. Zeuzier dam - Displacements shown by the right side pendulum at crest elevation before and during the events of the last months of 1978 and the first of 1979 (by O. Gicot)

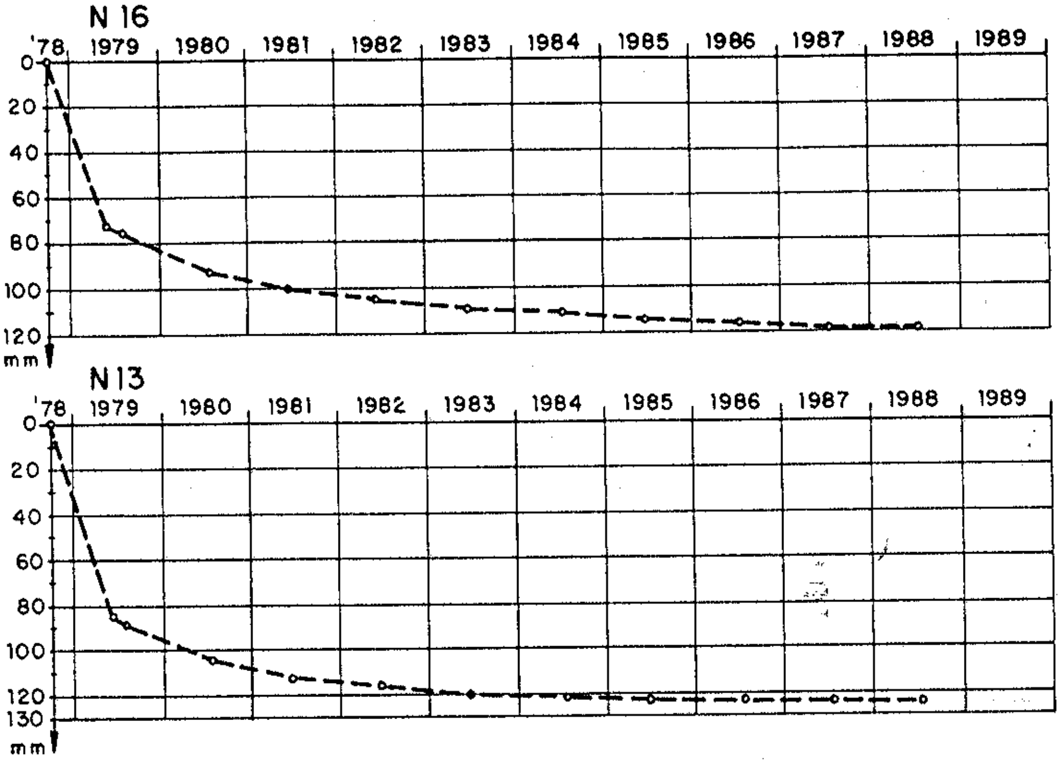

Figure 5. Zeuzier dam - Settlements of the points N13 (right valley side) and N16 (left) at the abutments of the crest arch

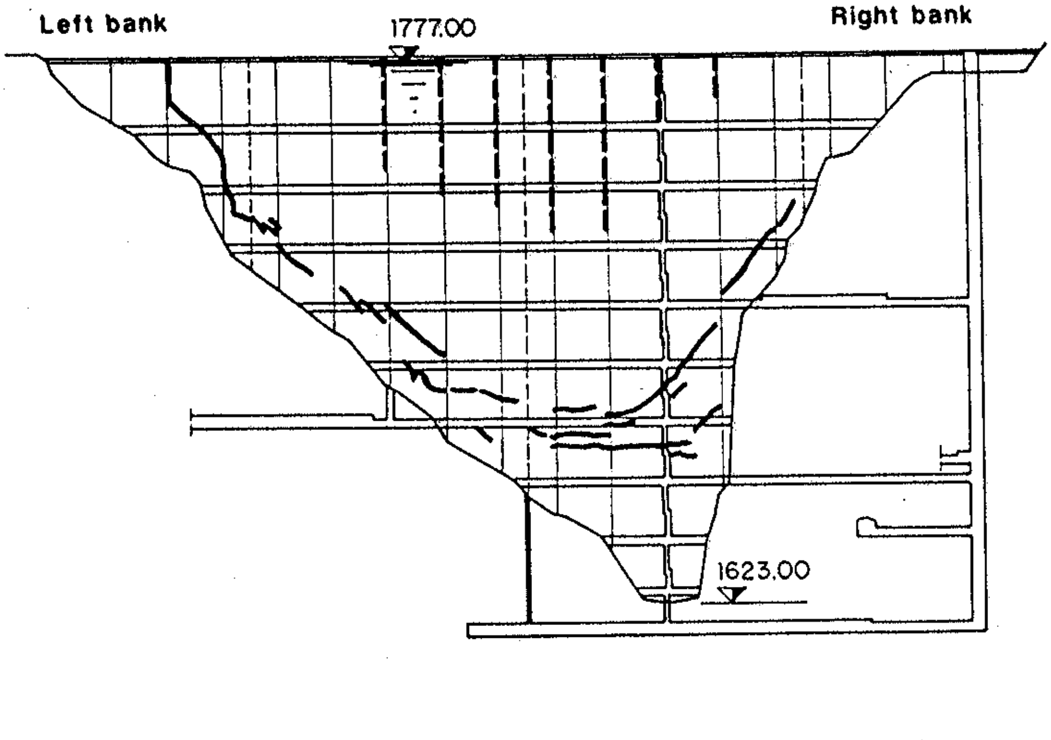

Figure 6. Zeuzier dam - Main cracks on December 1980 (opening above 1 mm); upstream face cracks = dashed lines; downstream face = solid lines

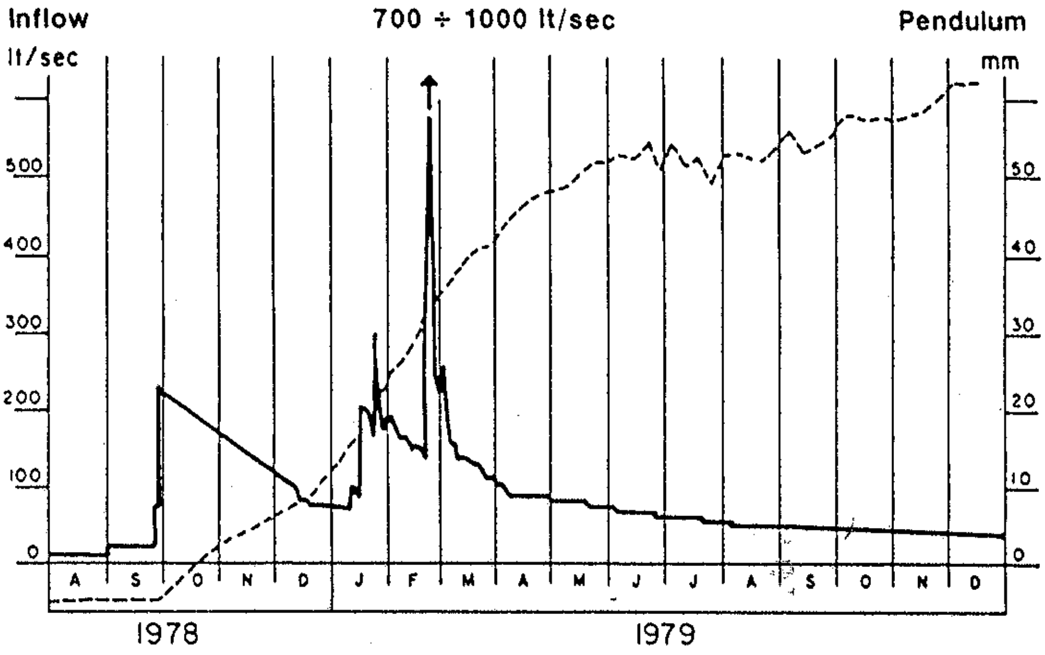

Figure 7. Zeuzier dam - Inflow in the exploratory adit (solid line) and displacement towards upstream shown by the central pendulum at crest elevation

A number of hypothesis were proposed to explain the movements at the foundation. So, for example, some tectonic phenomena were invoked which could have caused a shearing of the rock and a sliding along faults. However this explanation had to be ruled out quite soon, as no exceptional seismic event had occurred during that period, nor any observation or any geodetic survey of the region could support such a hypothesis. Also a sliding of a rock mass along any fault on the valley flanks against the arch had to be discarded.

The reason of the settlements and the deformation of the foundation could therefore be only one of hydrogeological nature as it will be shown later.

Unfortunately, the excavation of the mentioned exploratory adit was not properly monitored except for the flow of water entering it. No measurements of the ground water pressures, nor of the piezometric levels, or of the settlements at the terrain surface, were made during the excavation works. Only the data from the normal monitoring of the dam are available without any interruption since the time of construction. Additional instruments could be installed in the dam only a few months after the main event occurred in September 1978. For reasons of costs, deep bore-holes in the rock foundation were discarded.

For these reasons it was necessary to make some assumptions about the properties of the underlaying rock mass; these assumptions turned out to be very convincing or appeared at least to be quite reasonable.

1.2 The hydrogeological explanation

As already mentioned, the Dogger formation represented a confined, pressurized aquifer. On Figure 3, it may be seen how the exploratory adit was driving quite near to it. As the main inflows in the adit occurred at the crossing of open vertical faults, it is very likely that a connection with the aquifer existed along these surfaces of discontinuity and that a way for the water to escape was opened by the excavation. Figure 7 shows the variation of the outflow from the adit with time.

During the excavation of the adit peaks of inflow took place, in particular at the end of September

1978, January 1979 and February 1979. After the works in the adit were stopped in March 1979 a steady decrease of the flow was observed, from these peaks down to the present value of about 18 l/s. This fact is a clear consequence of the progressive emptying of the confined aquifer. Indeed, no influence of rain or snowfall, nor that of the reservoir level could be related to the rate of water inflow in the adit. Also the chemical analysis showed a deep origin of the water. Additionally the tritium analysis proved this water to be relatively old. The present outflow of 18 l/s is thought also to correspond approximately to the inflow from deeper regions which had fed and still feeds the aquifer. In particular, no connection with the reservoir appeared to exist. As shown by the chemical analysis, and supported by the already mentioned drainage effect of the valley, the aquifer cannot have been included in the Malm formation; it could therefore only be located in the Dogger formation.

It remains now to explain, how the drawdown of this aquifer by the adit had caused the very important settlements observed in the rock mass at the dam site.

1.3 Using the FES Model

Settlement in soils caused by the lowering of the water table is a well-known phenomenon. Also, settlements in oil fields are quite frequent. Settlements of such importance as in Zeuzier, resulting from the drainage of a very stiff mass of rock are not usual at least in the field of civil engineering, and few precedents are known.

At that time, to our knowledge, no mathematical tool existed which could have been used to analyse the phenomenum. This situation was in fact the actual stimulus to develop FES.

At the beginning, three particularities of the settlements observed were quite puzzling:

- the considerable amount of the settlement, of about 13 cm, which had occurred in such an excellent rock mass;
- the quasi-simultaneous water inflows in the adit and the beginning of the settlements (Figure 7) below the dam, in spite of the great distance of about 1.5 km between the two, and
- the fact that the settlements were more important at the dam axis than above the adit, which was suspected to be the cause of the settlements.

However, after some trials and adjustments, it was finally possible to show that the rock mechanic computations carried out on the basis of FES were able to explain all the facts observed, including the three mentioned above, as well as any value measured by instruments or by geodetic survey; all this in the frame of the precision normally required and obtained in technical analysis of this kind.

The most important parameters of the problem are, of course, the mechanical properties of the rocks in the Dogger formation. The following appeared to be the most probable values:

- Module of elasticity of the rock matter: 22.5 GPa
- Average distance of the discontinuities (thickness of the rock layers): 45 mm
- Average length of the undulations of the discontinuities: 100 mm
- Height of the undulations of the opening of the discontinuities (unloaded): 0.4 mm

Additionally, the piezometric level of the pressure in the aquifer was assumed to be at the beginning on elevation: 1600 masl; the elevation of the adit being: 1230 masl. The outlet orifice from the aquifer into the exploratory adit was to be opened in stages at different dates according to the crossing of the various faults by the excavation.

With the above values, the FES Model represented in Figure 8 was obtained. The total joint closure takes place at

- Stress: $\sigma_0 = 55$ MPa and
- Strain: $\varepsilon_0 = 9.42\ {}^0/_{00}$

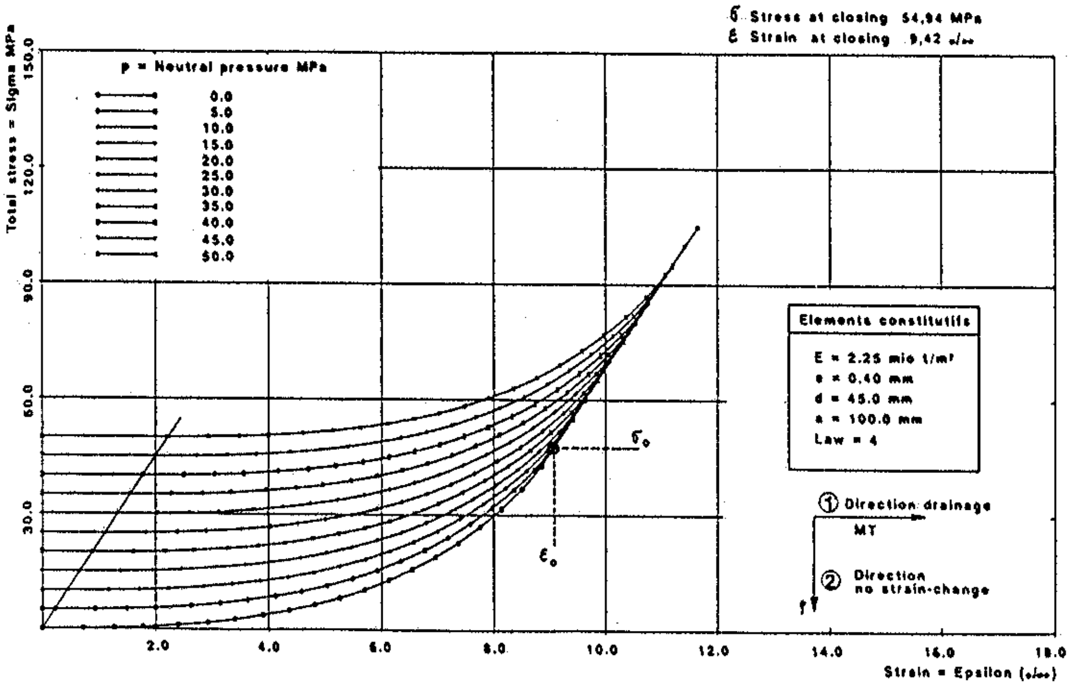

Figure 8. Numerically computed FES model for the Dogger formation at Zeuzier

The permeability of one set of joints as a function of the effective stress is best represented by:

$$k = k_0 \left(1 - \frac{\sigma_e}{\sigma_0}\right)^{m_1}$$

where, $k_0 = 2.5 \cdot 10^{-4}$ m/s and $m_1 = 10$.

The porosity of a set of joints in function of the effective stress is given by:

$$n = n_0 \cdot e^{-m_2 \cdot \sigma_e / \sigma_0}$$

where, $n_0 = 6.5\ ^0/_{00}$ and $m_2 = 5.2$.

The results, both for permeability and porosity, are shown in Figure 9 in the assumption that three sets of orthogonal joints as well as a secondary porosity of $7.5\ ^0/_{00}$ exist and that the ratio of horizontal to vertical total stresses in the original situation is 0.5. On the basis of these first results a frame for the computation was set up.

Figure 10 shows the extension in plan of the rock body studied. Obviously, some approximation is involved in the definition of its lateral boundaries. Figure 11 gives the cross section of the valley with the elements used for the computation. Additionally any 'column' of this scheme is subdivided into a number of elements.

The FES Model just described applies to the Dogger formation. At the beginning this mass was completely saturated; with time the water pressure decreases and the upper part of the Dogger 'anticline' starts to drain and behaves practically as a dry rock. No additional settlement has any longer to be expected to be caused by this drained out mass. It will simply deform elastically with the modulus of elasticity which corresponds to its actual locally variable state of stress. The same type of deformations takes place in the Malm limestone which is considered from the beginning to be unsaturated. Consequently, this rock mass settles practically as a block and in fact simply transmits to the ground surface the settlements of the upper boundary of the Dogger, spreading them across the valley because of its elastic rigidity.

The computation takes into account the coupling of the flow of water with the changes of the interstitial water pressure and the instantaneous level of the water table as well as with the changes in stress and strain and consequently with the changes in permeability and porosity of the rock mass. The computation proceeds obviously in short time increments, in order to simulate the progressive emptying of the confined aquifer, the variable water flow rates and the ongoing settlements. Of great importance is the fact that up to about 100 days after the first important water inflow in the adit, the aquifer was entirely pressurised. From this date on, a free water table formed in the upper part of the Dogger rock mass and progressively extended towards both valley sides.

This divide, or change in configuration of the problem, has a particular significance as before it the settlements were very fast; after it they slowed down very sharply. In the first period, being the aquifer confined, any change in the water pressure spreads thus very quickly from the adit into the entire aquifer and therefore causes almost instantaneous settlements all across the valley. As soon as a free water table has formed, the reaction is strongly damped and slow.

The main results of the analysis carried out are plotted in Figures 12 to 15 which are self explanatory and show the very satisfactory correspondence of the results obtained by computation with the geodetic measurements and with the instrument readings.

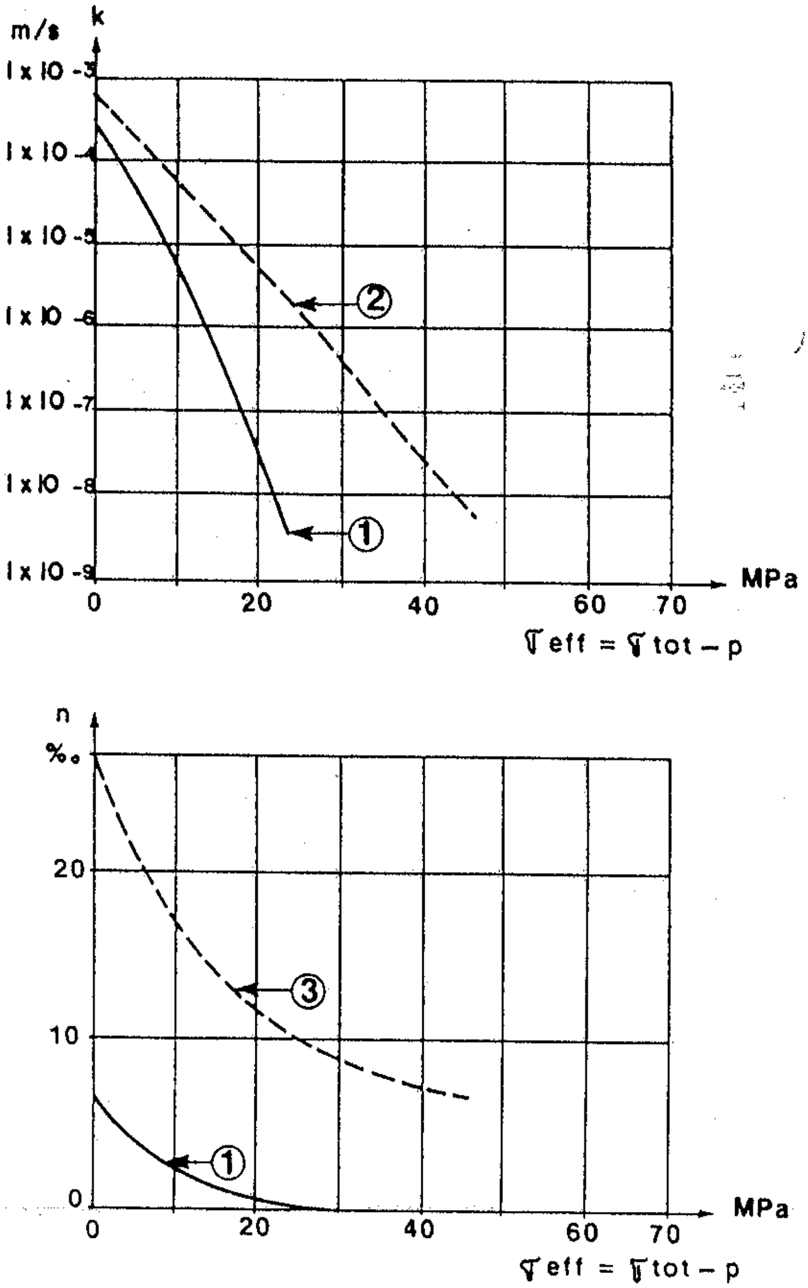

Figure 9. The Dogger formation at Zeuzier - The FESA model; Permeability and porosity versus effective stress, where: k = Permeability; n = Porosity; (1) = Single joint system; (2) = Three joint systems (ration of lateral pressure = 0.5); (3) = As (2) including second order porosity

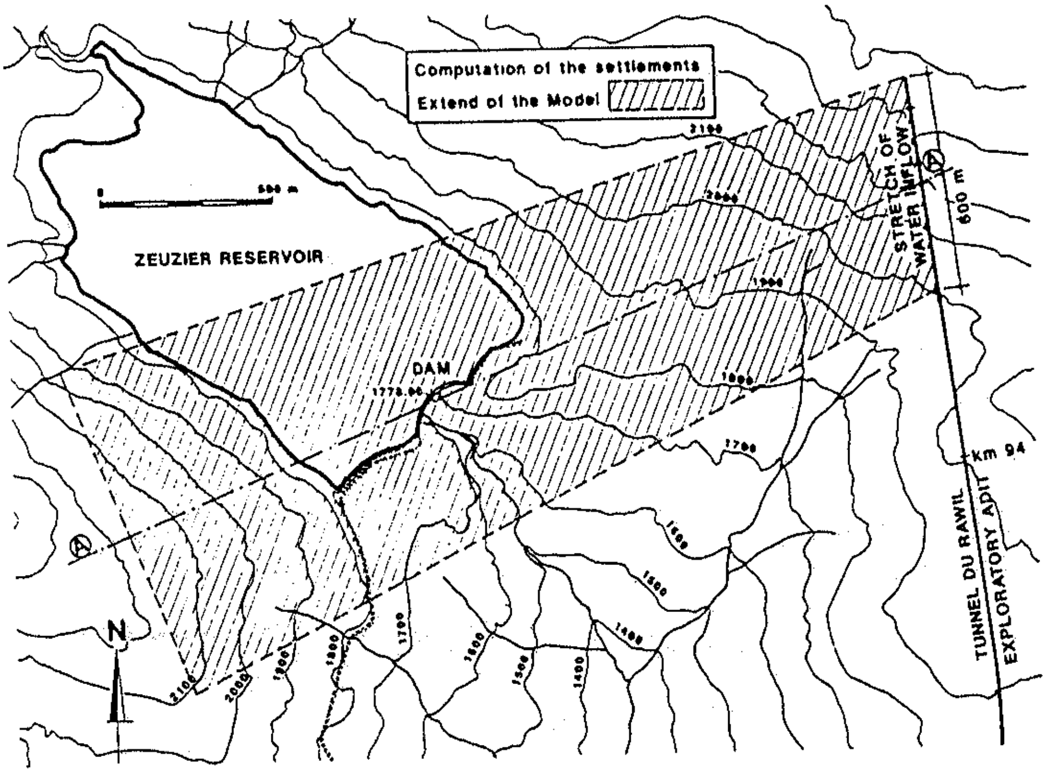

Figure 10. Zeuzier dam - Plan view of the extent of the model used to compute the settlements

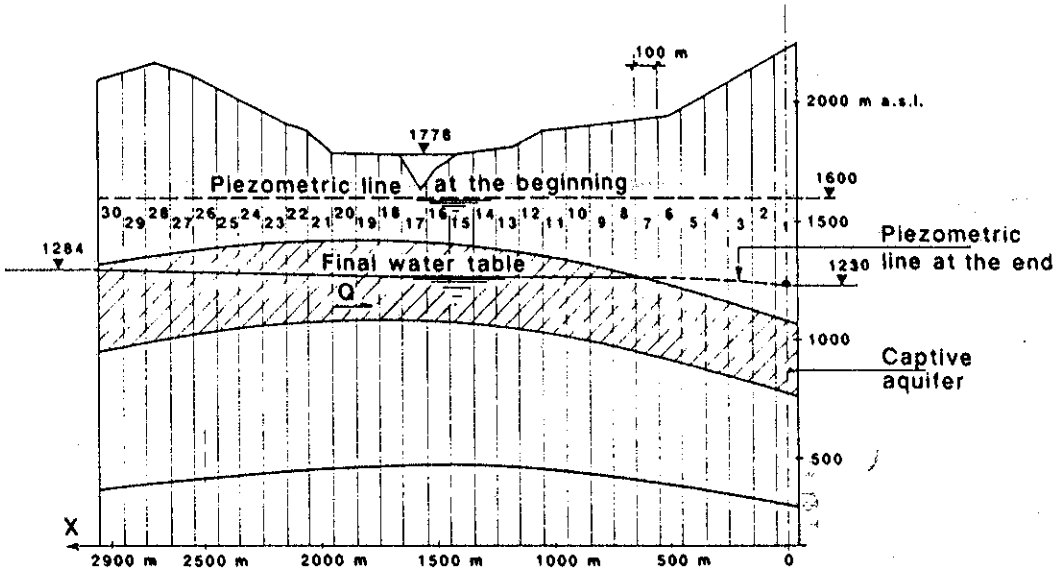

Figure 11. Zeuzier dam - Section A-A (see Figure 10) - Model to compute the settlements

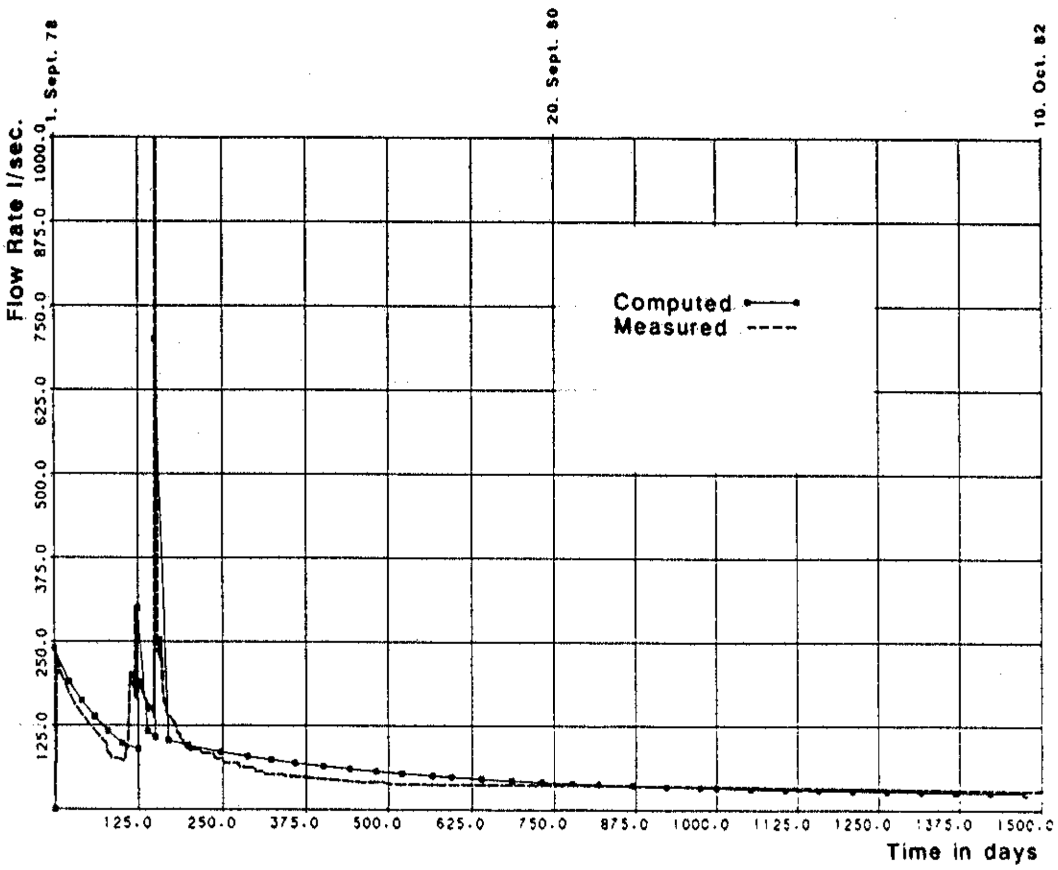

Figure 12. Zeuzier dam - Rate of inflow in the exploratory adit

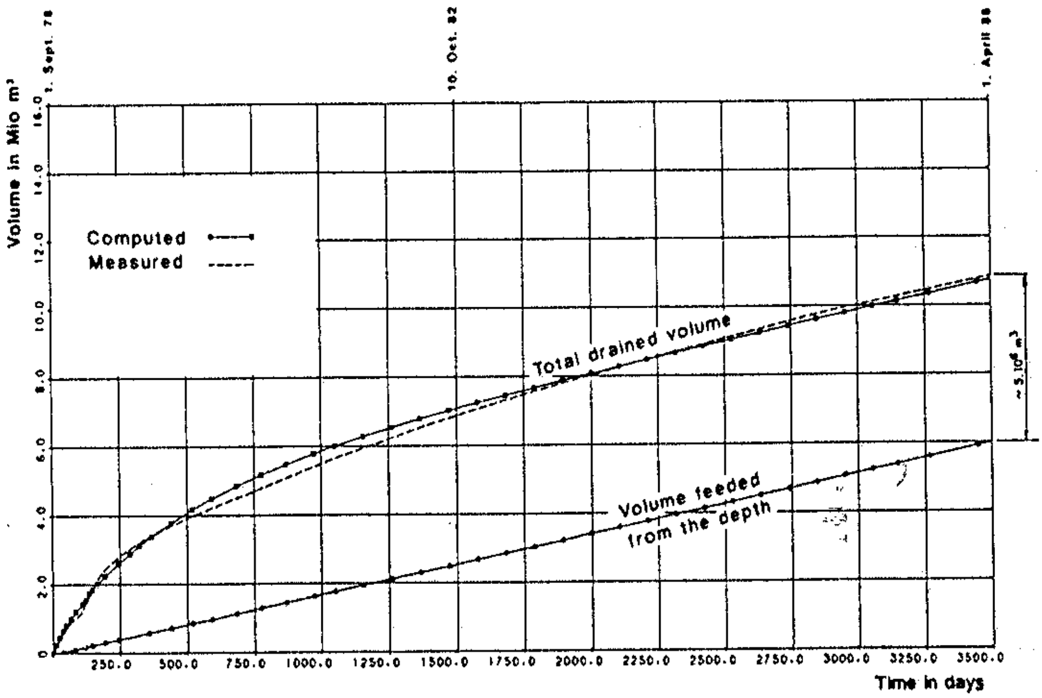

Figure 13. Zeuzier dam - Volume drained by the exploratory adit versus time

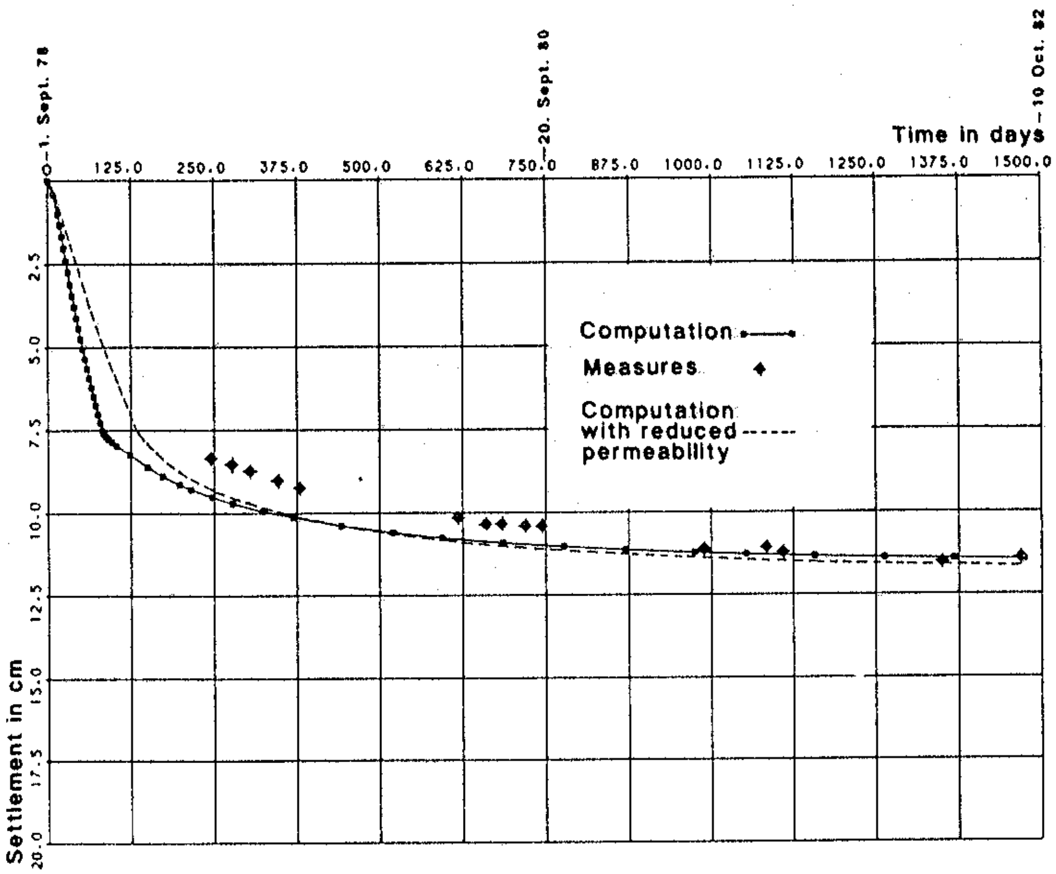

Figure 14. Zeuzier dam - Settlement below the dam versus time

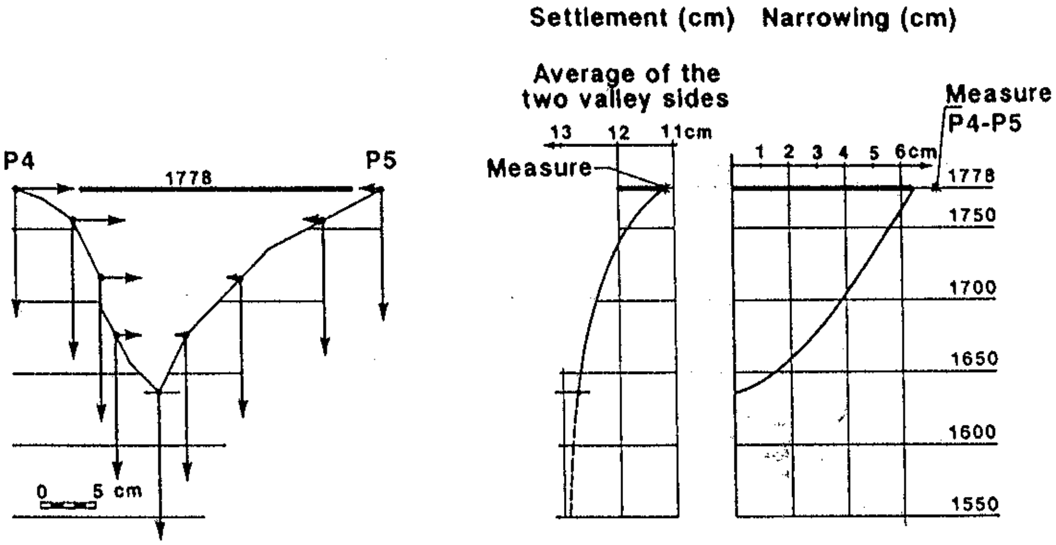

Figure 15. Zeuzier dam - Settlements and narrowing of the valley; the final situation

2 Using FES at Kölnbrein

The Kölnbrein arch dam in Austria is presently being strengthened after it had suffered extensive cracking in its lower part at the valley centre. The design of the strengthening includes an important concrete arch-gravity support structure, downstream of the arch. Between this structure and the dam itself, about 600 neoprene pads will transfer an important share of the hydrostatic load from the arch dam to the supporting block. The resulting load transmission is clearly influenced by the rigidity, or the deformability, of both the dam and the support block.

The deformability of the two includes the elastic as well as the non elastic, non reversible behaviour of the structures; for both structures it is additionally a function of the behaviour of the concrete mass as well as of the foundation rock.

As the arch dam was already loaded a number of times in its life, no important non reversible plastic deformations of the rock foundation need be expected in the future. The situation is substantially different for the newly built downstream structure, which will be loaded for the first time at the reimpounding of the reservoir and may therefore show some non elastic deformations of the foundation rock.

The ground below the support block is formed by different rock types: massive granitic gneiss, banked gneiss and shales of various types. For each one of these types an elasto-plastic computation was carried out. As an example, the results for shale No. 3 were shown in the first part of this paper[1].

Figure 16 indicates, for the various rocks, the amount of non-reversible strain to be expected at the first unloading versus the maximum compressive stress which was applied to the fissured, dry rock mass.

As the actual stresses on the rock foundation below the structure will be quite low, the analysis shows that no important non reversible deformations of the structure due to the foundation are to be expected. It was therefore not necessary to take into account such an effect in the lay-out of any of the supporting pads between dam arch and the downstream structure.

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[1] See Figure 12, p68, Issue 1, Volume III.

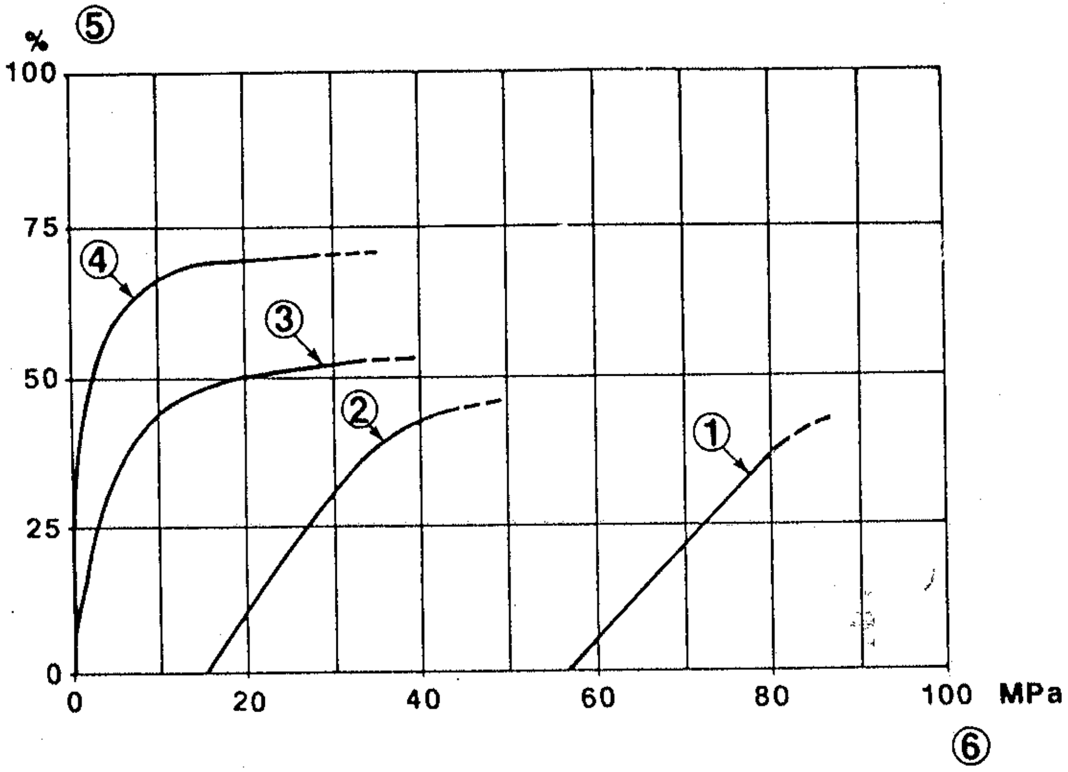

Figure 16. Kölnbrein dam - Foundation of the downstream support block. Non reversible compressive strain of the rock mass from initial state: (1) = Massive gneis (granite); (2) = Banked gneis; (3) = Shale No. 3; (4) = Shale No. 4; (5) = Non reversible portion of the total strain; (6) = Maximum compressive stress

3 Sliding

As already stated, the FES Model applies only as long as no sliding along the discontinuities takes place. It is therefore quite important to know where these limits are in any single case and at any point of the considered rock mass. It may be recalled from part one of this paper that the limit for sliding is given by the friction angle and by σ_c, the rock to rock contact stress[2]. Additionally some cohesion can apply in single cases.

The FES stress-strain diagram, as for example the one of Figure 8, can be completed to show not only the total stresses and the neutral water pressures but also the effective as well as the contact stresses.

Figure 17 gives the guidelines for reading Figure 18. To any point P of the stress-strain field the following values can be obtained:

- σ_t = total stress
- p = interstitial water pressure
- σ_e = effective stress $(\sigma_t - p)$
- σ_c = contact stress = $\sigma_e + \alpha \cdot p$
- α = degree of closing (contact surface)

Figure 19 shows a different manner of presenting the same relationships.

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[2] See Figure 1, 3a and Equation (1) of Part one of this paper: Issue 1, Volume III.

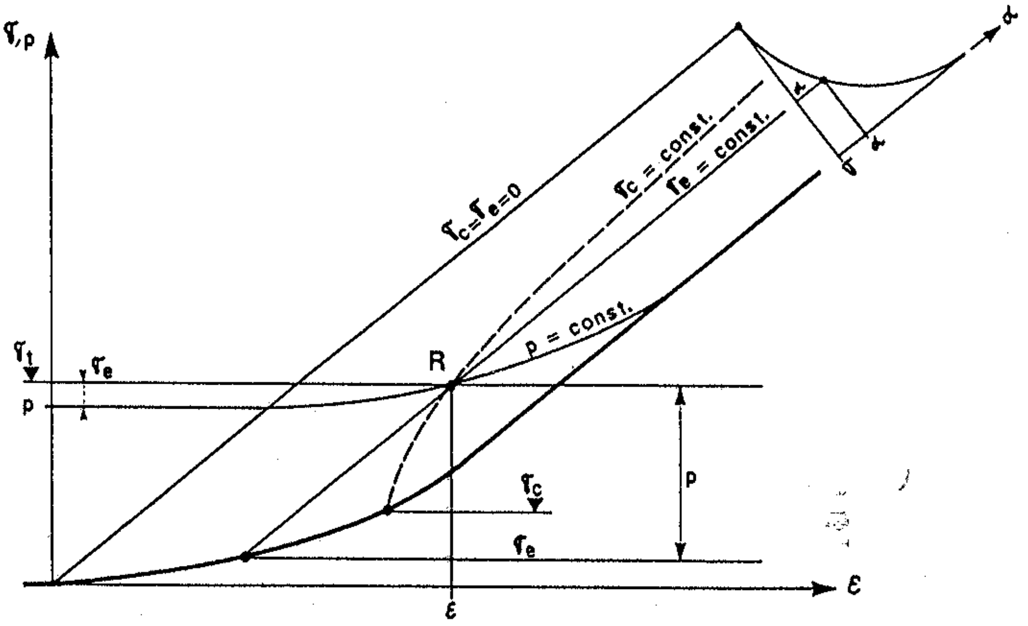

Figure 17. The FES model - guidelines for Figure 18; at any point R belongs to: - Degree of closure α; - Strain ε; - Total stress σ_t; - Neutral pressure p; - Effective stress σ_e; - Contact stress σ_c

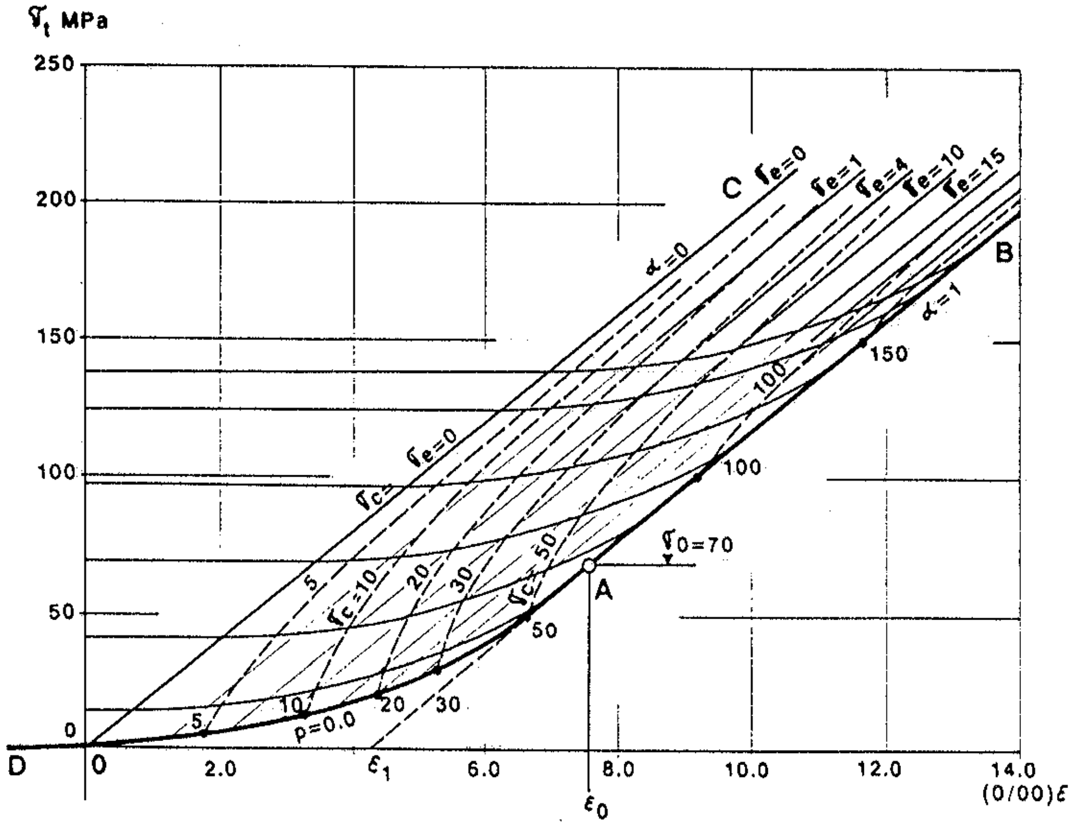

Figure 18. The FES model for a rock mass: ε = Strain; σ_t = Total stress; p = Neutral water pressure; α = Degree of closure [$\alpha = 1$ Fissures completely closed, $\alpha = 0$ Fissures completely open (first contact)]; $A(\varepsilon_0, \sigma_0)$ = Point of total closure at nil water pressure; σ_e = Effective stress; σ_c = Contact stress

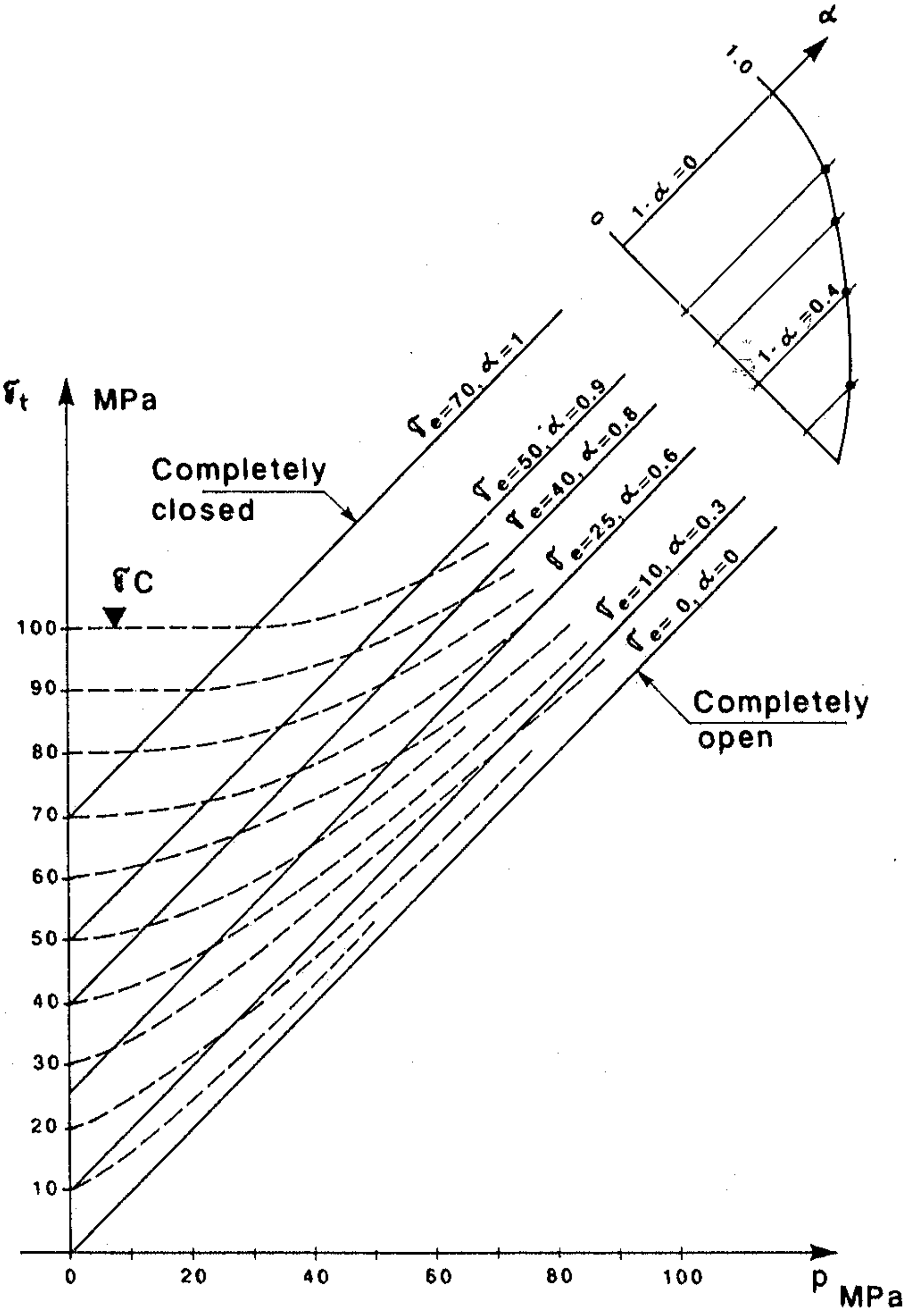

Figure 19. The FES model - Relationship between the different stresses, neutral pressure and strain as in Figure 18

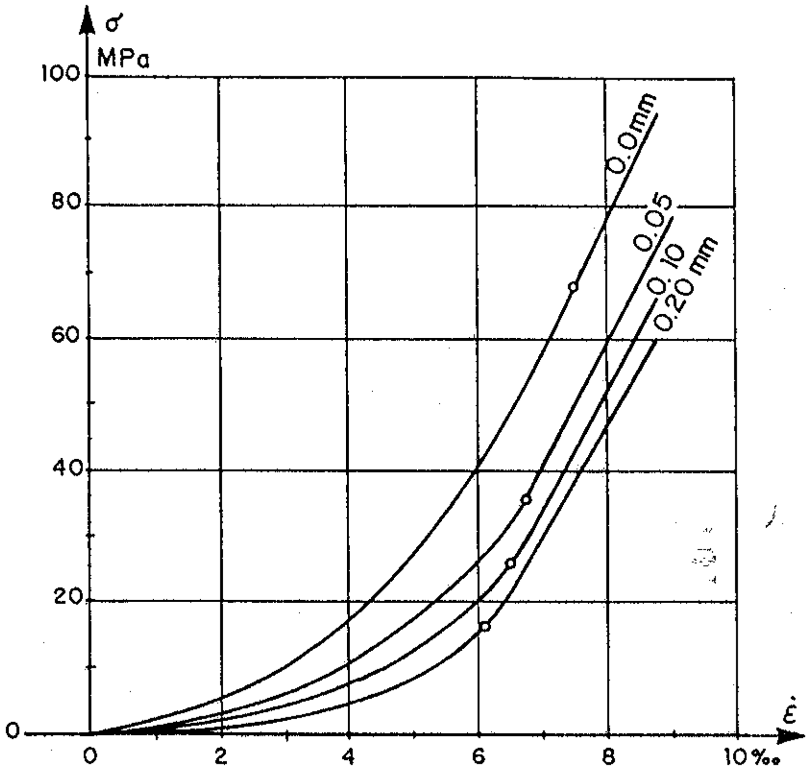

Figure 20. Influence of weathering of the surfaces of the rock blocks (example for dry rock); thickness of the weathered layer from 0 to 0.20 mm - modulus of elasticity reduced 10 times

4 Weathering

In order to solve certain engineering problems, it may be of interest to know the influence of some weathering of the surfaces of the blocks on the behaviour of a given rock mass.

Figure 20 indicates the influence of the softening of a thin layer of rock, or of a coating, along the surfaces of the single blocks. In this thin layer, the modulus of elasticity of the rock is reduced, in this example, by 10 times. The stress-strain relationship is shown for different thicknesses of the soft coating. It may be observed how significantly the shape of the stress-strain relationship varies.

5 Grouting

The grouting process is not a stable one in the sense that the grout always tends to concentrate in a single joint while the nearby rock mass is just compressed but not grouted because the corresponding fissures tend to close so to impede the penetration of the grout. For both the joint actually grouted and the two adjacent rock masses, the FES Model may be used as shown at Figure 21.

An effective grouting requires high pressures and a stable mix, that is a mix with an adequate cohesion. This implies generally a complete opening of the discontinuity or the joint grouted, so that no contact will exist any longer between the two blocks. This state corresponds to a point (A) to the left and above of the line OC in the FES Plot (see Figure 8). The total stress σ_t equals the grouting pressure. The representative point (B) for the nearby rock mass, in contrast, will lie on the right and below the said line, but at the same total stress level, for equilibrium. At this point a modulus of compressivity will be defined, depending on the actual, interstitial water pressure. The initial state of stress (I) in the rock mass will therefore split in two. The characteristic line of the widening of the joint, and consequently the grout penetration itself, can therefore be computed by taking into account the deformation of the adjacent rock mass with the corresponding modulus of deformability.

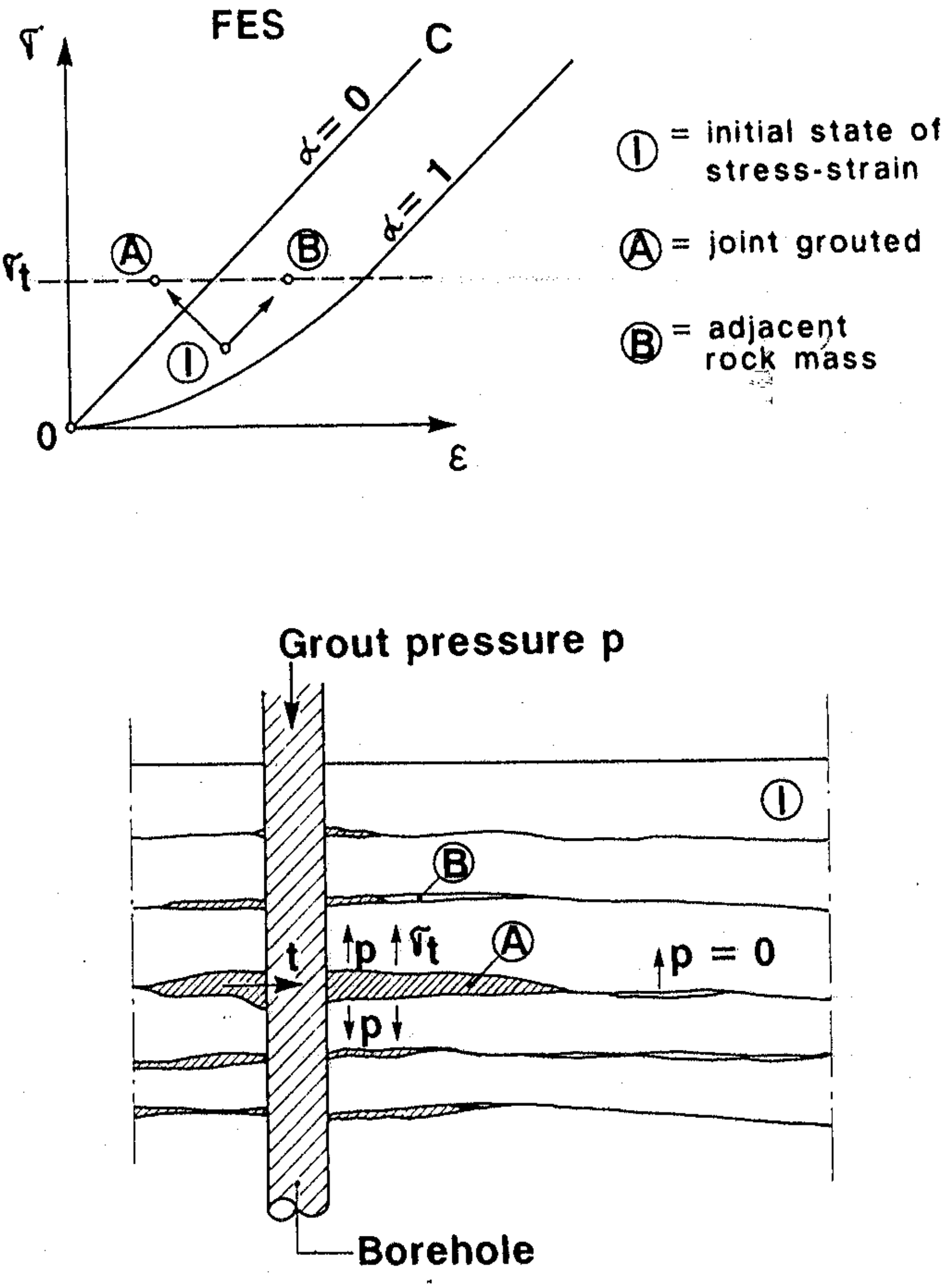

Figure 21. Penetration of grout in the joints. The total stress in the rock mass is the same but the openings of the joints are different due to different fall of the grouting pressure along the crack surfaces

Considerations on the basic idea of FES have contributed to develop new concepts for the grouting procedure which have already proved to be very useful at a number of dam sites.

Conclusion

A number of examples of the use of the FES Model in rock mechanics have been described. More applications are likely to take place in the future.

As usual with any of such mathematical models, back analysis of actual cases as well as field tests are highly desirable to confirm the adequacy of the model in order to solve practical engineering problems and possibly also to put in evidence additional factors which should be taken into account in the developments to come.

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