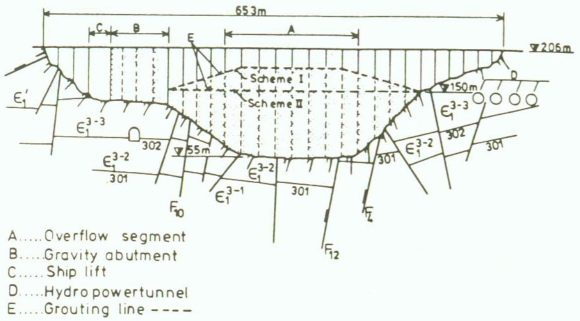
26 — GE Xiurun *(China)* (*)

THREE DIMENSIONAL NONLINEAR FE ANALYSIS OF GEHEYAN GRAVITY ARCH DAM ON COMPLEX ROCK FOUNDATION

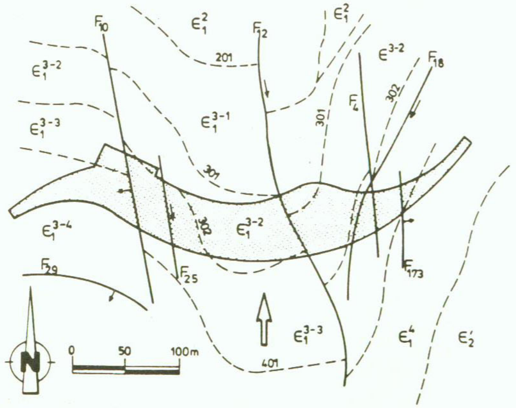
I would like to take the occasion to give you a brief introduction about the Geheyan gravity arch dam being constructed in China, height of which is 151 m, and the length of the crest is 653 m (Fig. 1 *a*). The installed capacity of the power station is 1 200 MW.

The design and construction of Geheyan Dam is very complicated because it is founded on the complex bedrock, existing a lot of weak intercalations with large scales and several large faults with strong karst erosion (Fig. 1 *b*).



Sketch of the upstream of the Geheyan gravity-arch dam

Fig. 1 *a*



Geological-Tectonical sketch of the dam site

Fig. 1 *b*

(*) Contribution prepared by Ge Xiurun, Gu Xianrong, Zeng Hung and Wang Chunying (China).

The exposed layers are marine facies carbonatite and fragmental rock from Cambrians. The limestone and shale are overlapped. The direction of the dip lays to the upstream and dip is about 25-30 degrees.

Considering the nonsymmetry of the valley shape, the complicated geological conditions and the unfavourable rock properties, particularly on the upper part of the left bank, the designer, the " Planning Office of the Yangtze River Basin of the Water Conservancy Ministry of China ", proposed a series of measures in order to improve the working state of the dam.

1. A large concrete gravity abutment on the left bank.

2. The dam body is divided into 30 dam sections during construction, afterwards only the lower parts of dam sections will be integrated by grouting. By means of this measure, the type of the Geheyan Dam is peculiar. The upper part of the dam works like a gravity dam which is founded on the lower part of dam body being below the grouting elevation and working as a real gravity arch dam.

3. The gradual elevating of the grouting level of the dam body reaches as high as + 18 m on the central part of the dam. This scheme adopted in design is called as structure scheme I (Fig. 1 *a*). The scheme II is with the grouting elevation of 150 m on the central part.

4. Shearing tunnel plugs will be constructed for some parts of the weak intercalations 301, 302.

According to the requirement of the designer we made 3 D FE Nonlinear Analysis for Geheyan Gravity Arch Dam, the purposes of calculation analysis are as follows :

1. To check the stress and deformation states of the dam body under different loading cases.

2. To give out the information of the working states of the rock foundation, particularly of the main faults and weak intercalations.

3. To evaluate the degree and extent of the influence of the no-tension character of the main fault upon the stress distribution of the dam body.

4. To make the comparison between the structures of dam body of scheme I and scheme II and to give out the assessments.

The numerical model should involve the whole dam body with large volume of rock foundation including major weak intercalations and faults (Fig. 2 *a*).

Therefore, the model consists of :

— 1 428 isoparametric elements (8-20 nodes);

— 417 joint isoparametric elements (8-16 nodes).

The total number of nodes is 3 280.

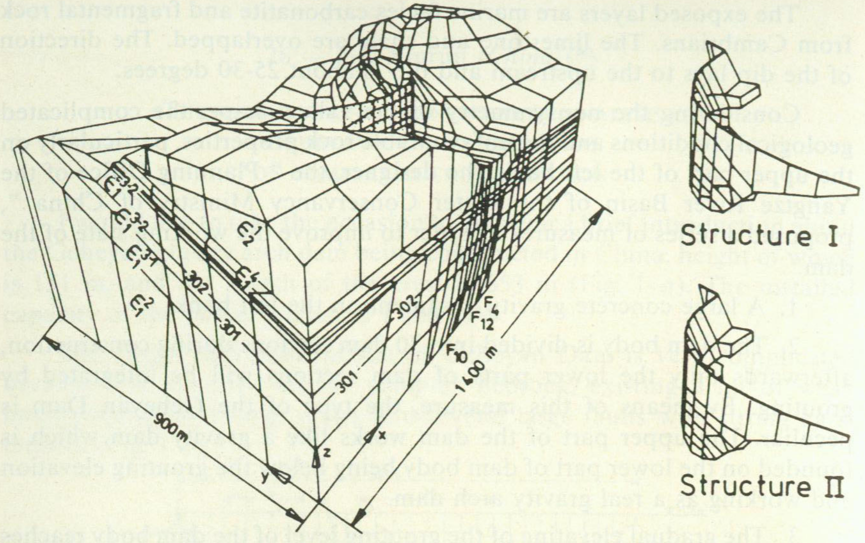


Fig. 2a : Numerical models for Geheyan dam and its foundation

Fig. 2 *a*

All the mechanical parameters of rock and weak intercalations used in the calculations are obtained from the large scale tests *in situ.*

The loading factors considered in the calculation include :

1. Dead weight of gravity arch dam and structure.
2. Static water pressure.
3. Silt pressure of reservoir.

The temperature loading is quite important for arch dam.

Two basic temperature variation cases called as temperature rising and temperature descending are considered in our calculation.

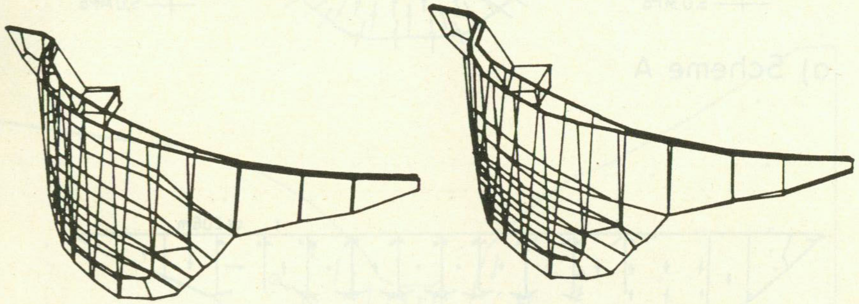
5. Permeasive pressure of ground water.

We have made a lot of calculation schemes considering different combinations of loadings and structures of dam. Here we only give out three of them called Schemes A, B and C. Calculation Scheme A is the case for dam body of structure Scheme I with the temperature descending and other loading above mentioned. The calculation Scheme C like Scheme A but with the no-tension analysis of main fault of rock foundation. Calculation Scheme B is the case for dam body of structure Scheme II with the same loading condition like Scheme A.

The 3 D FE nonlinear analysis program — JRNA 3 written by our institute has been used for the calculation of the Geheyan project. Using the special written program the calculation of the Geheyan dam has been completed on the personal microcomputer PC/AT with DSI-780 accelerate board.

Now some words about the results for Schemes A, B and C, particularly about the influence of grouting elevation upon the dam behaviours.

The dam body is dominated by horizontal displacement orienting towards downstream, the maximum displacement occurs at the top of the crown section. Fig. 2 *b* shows that it is about 7.8 cm for Scheme A and 9.7 cm for Scheme B.



Scheme A Scheme B

Fig.2b: Deformation diagrams of the dam body

Fig. 2 *b*

Fig. 3 *a* shows that the stress distribution of Scheme A is much better than that of Scheme B. The whole dam body is dominated with compression stress. Tensile stress only occurs at the dam heel, the value is generally about 2.0 MPa within a very limited region. The compression stress at both abutments of the Geheyan gravity arch dam generally exceeds 4.0 MPa, the maximum could as many as 5.7 MPa.

It can be seen that Scheme A which elevates the grouting elevation at the center of the dam could also play a better role in limiting the deflection of the dam body.

The axial thrust, radial shear and vertical shear forces borne by the concrete gravity abutment are slightly larger in Scheme A than those in Scheme B. However, the resultant direction of Scheme A tends much more into the foundation and rock mass of hills. That is, the bearing direction for concrete abutment in Scheme A is more reasonable.

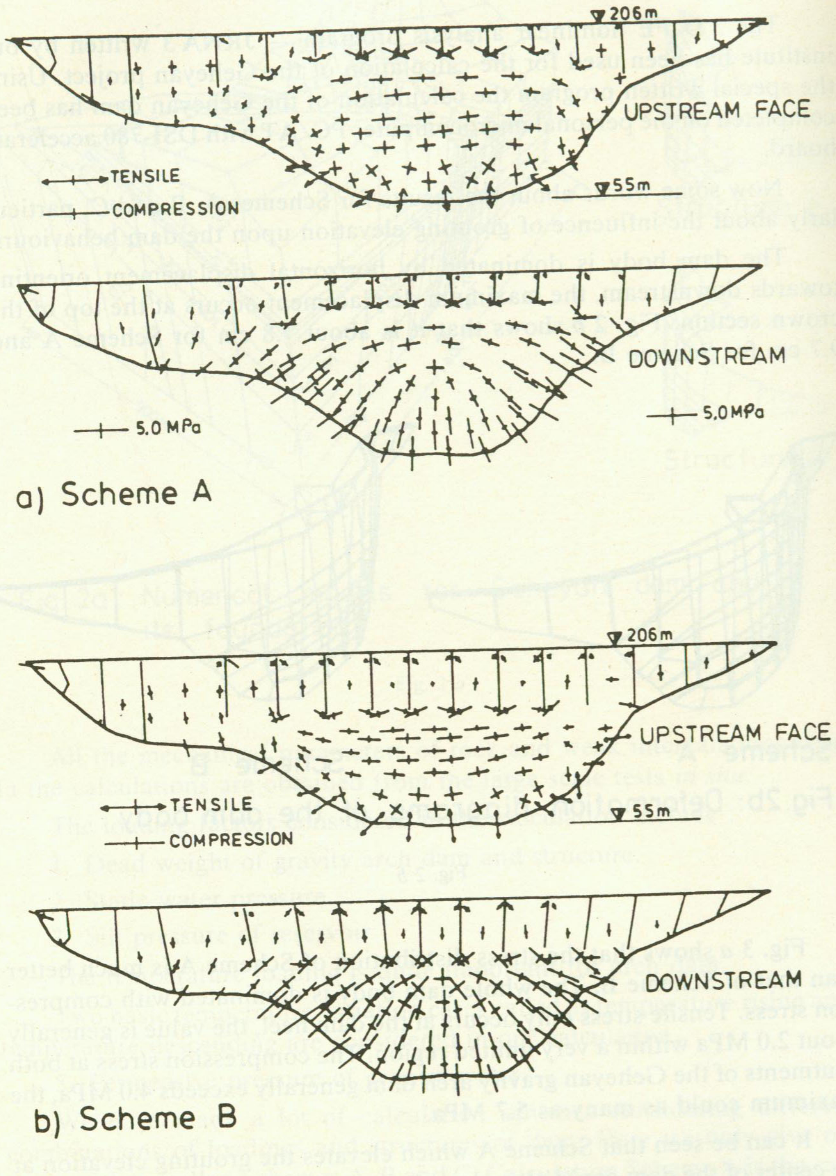


Fig. 3: Principal stresses on the upstream and downstream damfaces for scheme A and for scheme B

Fig. 3

Generally, Scheme A is obviously superior to Scheme B by its improvement of dam body stress, limitation of deflection and rationality of the bearing direction of the gravity abutment. That is to say, the grouting elevation for dam body of Scheme I is much more reasonable.

Now some words about the displacement and stress distribution of weak intercalations and faults.

Fig. 4 shows the relative displacement diagram of 301 weak intercalation between its top and bottom planes.

The diameter of circle represents the amount of vertical relative displacement, and the arrow represents the direction and value of shear relative displacement. Here you can see more concentration on the central

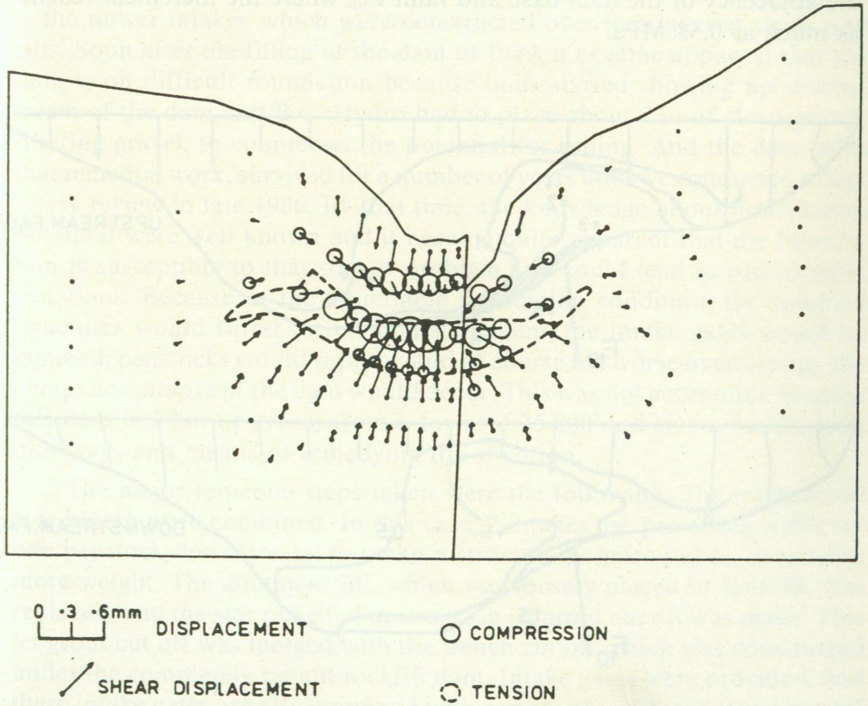


Fig. 4: Relative displacements between the top and bottom planes of the weak intercalation (301).

Fig. 4

part, and the shear displacements are oriented towards to the centre. So it is quite different with the concept of sliding of a rigid body on a plane.

The normal tensile of fault F_{10} appears in the joining locations of the dam body generally less than 0.05 MPa, the maximum being 0.26 MPa.

The combination of F_{10} and weak intercalation 302 plays a critical role in the stability of the dam abutment on the left bank. Therefore, it is of typical significance to select F_{10} as our no-tension analysis to show its influence upon the stress distribution of the dam body.

By contrasting analysis of the elastic and nonlinear schemes (calculation Scheme A and C), we find that the influence of no-tension analysis of fault F_{10} is most significant on the dam body stress (Fig. 5). The stress increments obtained by the stress transfer process of nonlinear calculation are focused on the locations at the left abutment of the dam, especially on the adjacency of the dam base and fault F_{10}, where the increment reaches as much as 0.58 MPa.

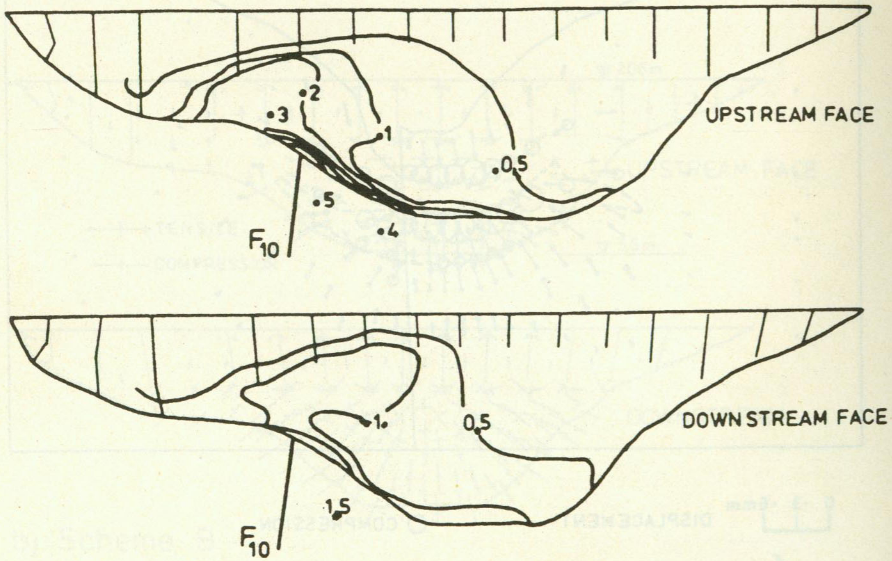


Fig. 5: Isolines for stress increment of σ_1 on the upstream and downstream faces due to no-tension of F_{10}.

Fig. 5

Conclusions

The results of the three dimensional finite element analysis for the gravity arch dam and the complex rock foundation has already been adopted and applied to the optimal design of the dam body as well as to the guidance of construction. This study is also the first successful case in our country to employ the personal microcomputer in 3 D FE analysis of such a large scale geotechnical engineering problem.

27 — T. PATAKY *(Canada)*

I am going to talk about the John Hart Dam on the west coast of Vancouver Island in Canada. The portion of the dam which is of interest is the power intakes which were constructed over interlayered sands and silts. Soon after the filling of the dam in 1949, it became apparent that the dam is on difficult foundation because boils started showing up downstream of the dam and B.C. Hydro had to place about 2 m of clean gravel, filtering gravel, to counteract the potential for piping. And the dam, with that remedial work, survived for a number of years until we conducted a dam safety review in late 1986. By that time, the knowledge about liquefaction potential were well known and it became quite apparent that the foundation is susceptible to that sort of problem and could lead to catastrophic situations. Because of the liquefiable foundation condition, the concrete structures would suffer differential settlement, the intake gates would be jammed, penstocks would rupture, and of course the worse overturning, the complete collapse of the dam would occur. This was not acceptable because this dam is 2 km upstream from a town of 35 000 and so we had to look into ways and means of remedying the situation.

The major remedial steps taken were the following. The intakes and wye blocks were combined. In this case, 2 intakes are providing water for one penstock downstream, to make it structurally more stable, to provide more weight. The abutment fill, which was loosely placed in 1946-48, was replaced, and the star remedial measures, a jet grout cut off was made. This jet grout cut off was merged with the trench cut off which was constructed under the completely rebuilt rockfill dam. Intake gates were provided, and these intake gates are also equipped with earthquake-induced flow shut-off devices.

As you can see (Fig. 1) the right abutment and the left abutment were replaced with compacted fill, rather a tricky operation. During all this time we had the reservoir off, which was a couple of metres below the top of this section here. In the meantime they've completed the cut off trenches