Fourth Australia - New Zealand
Conference on Geomechanics,
Perth, 14-18 May 1984

The Determination of Stability of Arch Dam Abutment Using Finite Element Method and Geomechanical Models

CHOW WEI YUAN
Associate Professor of Hydraulic Engineering, Vice-Director of Hydraulic Engineering, Tsinghua University, Beijing, China
YANG RUO QIONG
Assistant Professor of Hydraulic Engineering, Tsinghua University, Beijing, China
and
LUO GUANG FU
Geological Engineer, Tsinghua University, Beijing China

SUMMARY The determination of arch dam abutments by means of the finite element method and geomechanical models is described. Geomechanical models have been performed in our laboratory. The stability factors of ER-arch dam, obtained by different approaches are in good agreement.

1 INTRODUCTION

In our country, a number of large arch dams have been constructed, some of them are higher than 100m. The problem of stability is acquiring greater importance in the design of arch dams, since the dams are now higher and wider, and the quality of these foundations is becoming more deficient in these projects. But the problem of stability of arch dams are still far from the sufficient knowledge of engineers, because it is extremely complicated to analyse the mechanism of rock abutments.

It is well known that the rigid block method is a conventional and traditional method in calculating the stability factor of arch dam abutment. By this approach, the problem of stability is reduced to a statically determinate one that along the boundary surfaces of sliding rock block, the state of stress by introducing rigid block hypothesis becoms determinate. The main drawback of such an approach consists of the difficulty of determining the qualitative influence of these hypothesis on the S.F. obtaind. For large dams especially under complex geological conditions, it is deficent to use this approach as the main way in evaluating its S. F. of abutment. The deficiency lies in the fact that the established rupture may not correspond to planes or surfaces of least resistance in the ground and may not be oriented in the worst direction with repect the acting forces. In recent years two appoaches have been utilized in our analyses: Geomechanical model tests and F.E.M. Great progress have been made in analysing stability of arch dam abutments by means of them. This paper will briefly examine these two approaches in analysing the stability of ER-dam.

2 GEOMECHANICAL MODEL TESTS

The structure of a large double curvature arch dam, ER-dam, of 245m high has been studied in our laboratory. As shown in Fig 1, the ER-dam site is located on the synite and basalt rocks, there exists a large area of bigger initial stresses. On the left bank the rock masses are rather intact. No large faults were discovered, only several sets of joints cross the rock masses, On the right bank, the rock masses are mainly composed of basalt. The rock masses are cut by three sets of joints and a fault F20. A belt of weak rock zone, uralitized basalt was found not far from the right arch dam base. One conventional model test of 3-dimension and seven rupture geomechanical tests of 2-dimension have been performed in our laboratory. The 3-d conventional model was utilized to investigate the dam behavior under normal loads. In this paper, the rupture tests were given specific attention.

TABLE I

GEOLOGICAL CHARACTERISTICS OF DIFFERENT DISTRICTS

Geological zone	Strike, dip angles	Ratio of discontinuity of joints	Parameters of strength	
			f	c
Basalt P_2	N49E, NW 72°	50%	0.8	8
Uralitarized basalt P_3	NW75° NE55°	10%	0.45	3
Minicrystal Basalt P_1	N45E NW64°	50%	1.0	18
Synite c	N64E NW66°	65%	1.2	25
Load released Lr		100%	0.7	4

In these models different geological conditions including deformability, characteristics of the foundation rocks and the main geological disturbances were reproduced. The models were made of heavy casting plaster, so that $C_\gamma = 1$. The material can be obtained by using BaO powder mixed with gypsum. The model was made of about 3500 little rectangular parallelepipedal blocks capable of reproducing, when placed close to one another, the fracture planes, the stratifications of the rock, as shown in Fig 1. The blocks were divided into four groups: 4 x 4 x 7cm., 3 x 5 x 8cm., 4 x 4 x 4cm., 5 x 5 x 5cm., according to their geometric scale, different rock zones can be made of them.

As shown in Tab I, in the models have reproducced 5 different geological zones their mechanical parameters are given there.

The following similarity criteria should be satisfied. At first, suppose $C_\gamma=1$, $C_\varepsilon=1$, $C_f=1$ then $C_c= C_E= C_\sigma= C_L$.

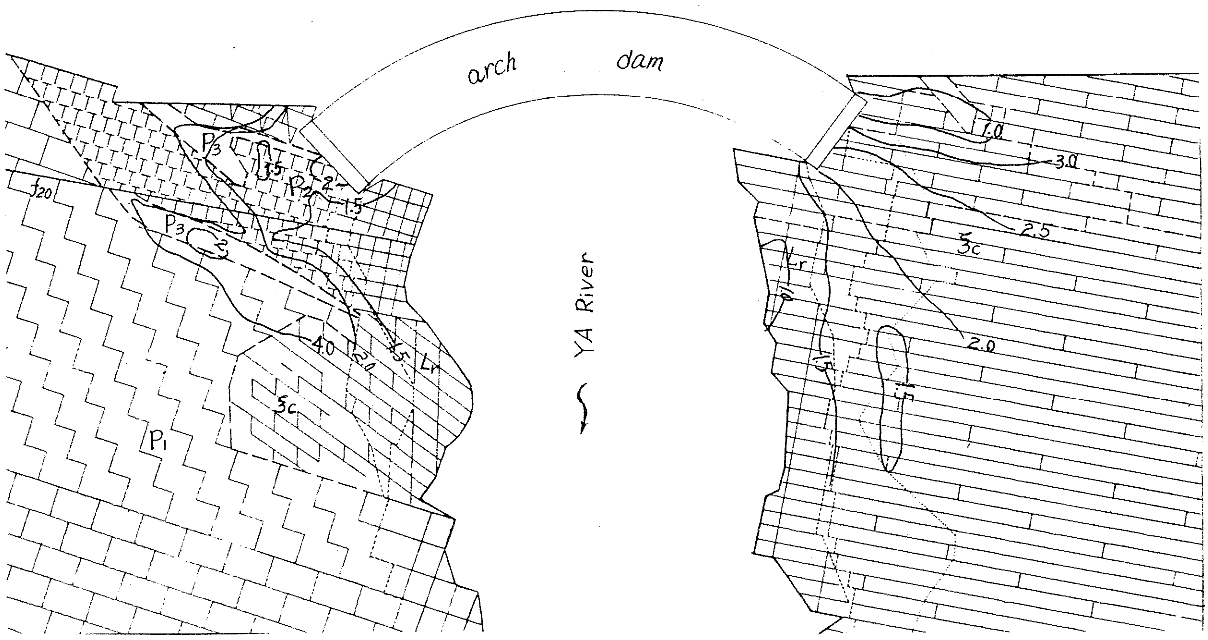

Figure 1
Schematic geomechanical sketch of dam foundation and isolines of safety factor

C_γ—— the ratio of density between prototype and models
C_ε—— the ratio of strain
C_f—— the ratio of coefficent of friction
C_c—— the ratio of cohesion
C_E—— the ratio of elasticity
C_σ—— the ratio of stress
C_L—— the geometric scale ratio

In Fig. 2, different block model combination specimens which were assumed from 5 geological zones in scale models were shown and triaxial test results for each sample were given. These block combination samples behaves much flexible, i.e. their modulus of deformability have a smaller value than that of one intact block. These constitutive relations and strength parameters of rocks are kept in similarity with that of prototype.

The faults F20 contained some breccia of modest thickness (2-3cm). To reproduce this breccia plastic sheets 0.1mm thick were inserted between the contact surfaces of the model which give a fairly good imitation of the rhelogical characterios of the infiling material C=0, ϕ=25°

To determine the safety factor of arch dam models, we use the increasing load method, i.e. we apply water load to the model increasingly and meanwhile observe its behavior until rupture occurs. Fig.3 shown the cross section of horizontal elevation 950m, it is about 1/3 hight of the dam. Of course, in this elevation exsists the most critical safety factor. Studying its safety is significant to the whole dam. The water load was applied to the model by means of oil jacks and its behavior was observed using induced eletro-deformometers, autorecorded in electronic computer.

The behavior of model during tests were given, in Fig.3, a tensil crack AD could be observed in the zone near upstream dam heel this referred to K_1 when the load increased a little more than the mormal load. Generally, the model behaved still elastically under this loading step, after

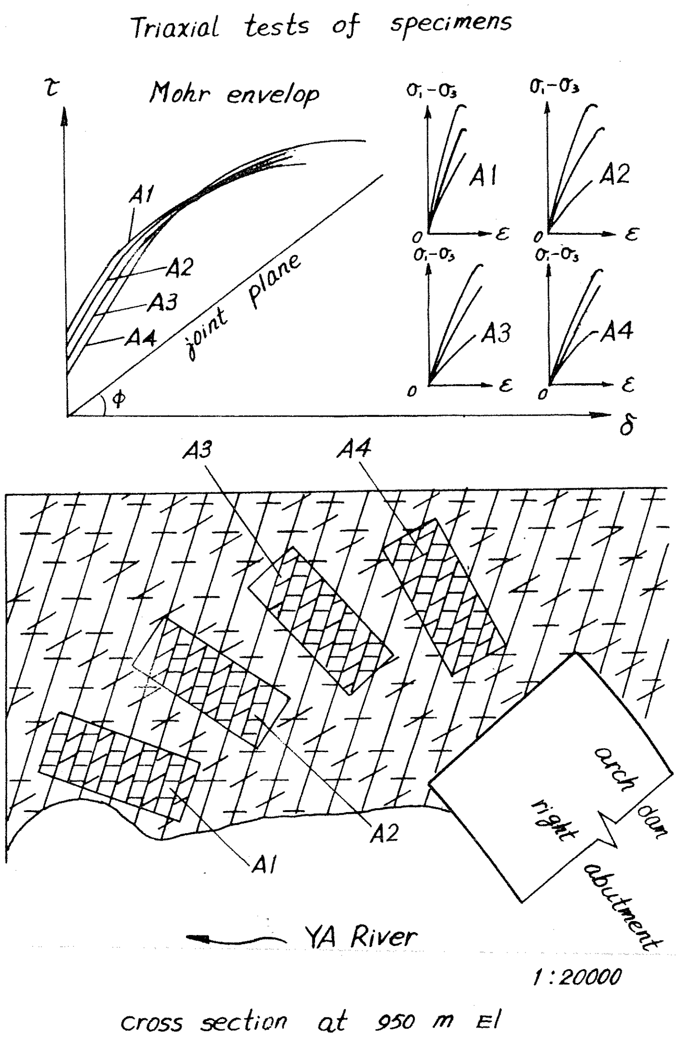

Figure 2
Ultimate triaxial test specimens and their strength charateristics

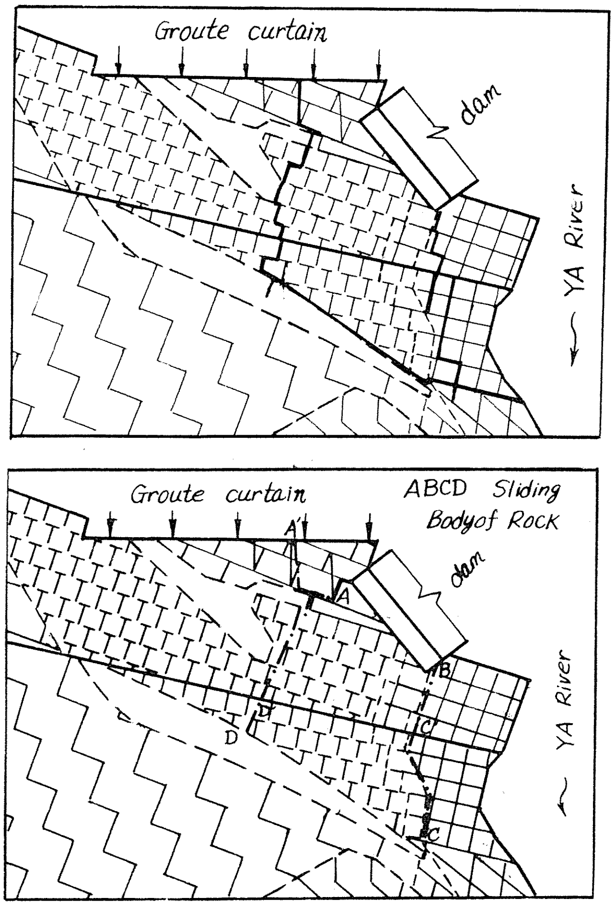

Figure 3
Geomechanical models after failure tests

increasing load, some cracks "BC" can be found in the area near the dowmstream face of dam, by this time, the model behaved plastically, and a large deformation could be observed this referred to K_d. Finally a sliding crack along CD, the weak zone surface occured, the whole structure failed this referred to K_2. During this progressive failure safety factors may be defined as this:

$K_1=P_1/P_0$

$K_2=P_2/P_0$

P_0 normal load

P_1 elastic limit load while the structure just come to plastic stage

P_2 ultimate load of structure

K_1 safety factor referred to elastic state

K_2 safety factor referred to ultimate state

Seven two-dimensional models of cross section 950m elevation have been performed, to which different geomechanical conditions were assigned. In Tab.II, the conditions were listed, all of them are 1:200 scale. The results of tests can be seen in Fig 3.

Model No. 2 corresponds to the pretreatment. As in Tab II, 6.5×10^4kg-cm^{-2} was assigned to deformation modulus of P_2, whereas 1×10^4kg-cm^{-2} to deformation modulus of P_3. This means that the deformation modulus of weak rock P_3 is much lower when compare it with that of P_2. From Tab III, it can be observed that K_1 and K_2 are 0.7 and 2.2 respectively, which cann't be recognized safe. From Tab IV, its displacement ratio is 212%, which is much bigger.

TABLE II

SEVEN GEOMECHANICAL MODELS AND THEIR PARAMETERS ASSIGNED

No. of models	Initial elasticity modulus			Treatment
	P_2 zone	P_3 zone	L_r zone	
1	6.5×10^4	3×10^4	3×10^4	P_3: cement grouting
3	8×10^4	3×10^4	3×10^4	P_2, P_3: cement grouting
2	6.5×10^4	1×10^4	3×10^4	No
4	8×10^4	3×10^4	5×10^4	P_2, P_3, L_r: cement grouting
5	8×10^4	3×10^4	3×10^4	P_2: cement grouting P_3: cement grouting and concrete replacement
6	6.5×10^4	6.5×10^4	6.5×10^4	Homogeneous foundation assumed
7	6.5×10^4	1×10^4	5×10^4	P_3: Concrete replacement and prestressed cables

Model No. 1, corresponds to treatment of P_3, to which, some higher deformation modulus and strength parameters were assigned. The deformation modulus 3×10^4kg-cm^{-2} was assigned to P_3 this corresponds to the value after treatment. From Fig 3 and Tab III K_1, K_2 are 0.75 and 2.4 respectively, which cann't be permitted in design. The displacement ratio is too big to be permitted as well.

In model No. 3 a treatment of P_2, P_3, was adopted, some higher D. M. and strength parameters were assumed, where 8×10^4kg-cm^{-2} was assigned to P_2. From Tab III, we have observed that K_1, K_2 equal 0.7 and 3.1 respectively which are just permitted in design. A much better improvement can be observed in model No. 4 and model No. 5. A treatment of P_2, P_3 was adopted: cement grouting in P2 and L_r, concret replacement of P_3. K_1, K_2 equal 1.3—1.5 3.4—3.6 respectively, which can be accepted in design and their displacement ratio falls in permitted extent.

Model No. 7 corresponds to another treatment: in weak rock zone P_3, the rock is to be adopted prestress cabling. The deformation modulus of P_2, P_3 will be remained unchanged as the intact rock, From Fig 3 and Tab III, K_1 and K_2 of this alternative are 1.0, and 2.5 respectively, and it's displacement ratio is too big to be permitted. This treatment cann't be recognized assured.

Model No. 6 corresponds to an ideal state of abutment of which deformation modulus and other mechanical parameters were assumed to be like those of homogeneous rock foundations. By means of this model, some permitted indices can be obtained, so as to be compared with other alternatives.

TABLE III

THE OVERLOAD FACTORS REFERRED TO FRACTURE DEVELOPMENT

Fracture stage / No. of models	Crack of AD zone	Initial crack of BC zone	Crack of whole BC zone	Initial crack of DC	Crack of whole DC zone	Failure of whole structure
	K_1	K_a		K_b	K_d	K_2
1	0.75	1.9	2.1	2.0	2.4	2.4
2	0.7	1.6	1.8	1.3	2.2	2.2
3	1.1	2.3	2.5	2.2	3.1	3.1
4	1.3	2.5	3.0	2.1	3.4	3.4
5	1.5	2.6	3.5	2.4	3.6	3.6
6	2.0	2.6	3.5	2.4	3.7	3.7
7	1.0	1.9	2.4	1.8	2.5	2.5

* The overload factor $K = P_i/P_0$

TABLE IV

DISPLACEMENTS OBSERVED IN MODEL TESTS unit: cm

No. of Models / Points	1	2	3	4	5	6	7
Upstream pt of dam	1.65	1.68	1.49	0.99	1	0.99	1.46
Downstream pt of dam	2.23	2.63	1.43	1.14	1.06	1.01	1.88
Mid-pt of dam base	1.94	2.13	1.13	1.01	1.04	1	1.5
Ratio with 6# model	193%	212%	112%	101.2%	104%	100%	150%

In Tab IV, a comprision of deformations of 7 models were shown, the deformation should be held in the ratio range of 10% than that of isotropic homogeneous model, other-wise, the reinforcement is needed. From post-treatment model No. 3, No. 4, the mechanism of failure is in some extent different with pretreatment. Owing to the grouting of P_1 zone, K_2 was obviously rised, owing to reinforcement of weak belt, P_3 the sliding wedge take F20 as its sliding plane, and the downstream cracks appeared with much higher loading. Model No. 3, and No. 4, satisfy the above mentioned criteria, and accepted in design.

3 F.E.M.

Two kinds of mechanical models were assumed in F.E.M: the 3-dimensional elastic model and two-dimensional nonlinear model.. The latter refferred to a horizontal cross seetion as in scale model test.

3.1 Elastic Model

By three dimensional F.E.M, a series of calculations were carried out to determine the stress distribution of arch dam and its safety factors of abutments, the dam and its foundation are divided into 198 elements which are 20 point isoparametric hexahedron elements.

From the stress field in the foundation, the punctual stability factor can be obtained as follows:

$$S.F. = \tau_R / \tau_0$$

$$S.F. = \left(c + tg\phi\left(\frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\phi\right)\right) / \frac{\sigma_1-\sigma_3}{2}\sin 2\alpha$$

$$\alpha = \frac{1}{2}\cos^{-1}\left(\frac{-tg\phi\left(\frac{\sigma_1-\sigma_2}{2}\right)}{c+\left(\frac{\sigma_1+\sigma_2}{2}\right)tg\phi}\right)$$

Where τ_R —shear strength, τ_0 —punctual stress

Fig I shows the isolines of S.F. Here a good agreement of isolines with failure mechanism of scale models presents. The isolines of S.F. referred to 2.0, are presented along the trace of the cracks of models, that means the inadeguate S.F. isolines correspond to the failure cracks in models.

3.2 Elasto-plastic Model in F.E.M.

Elasto-plastic model calculation was used as another approach to determine its safety factors. The approach usually is called plastic increament method. The drucker-prager criteria has been adopted as the rock yield criteria.

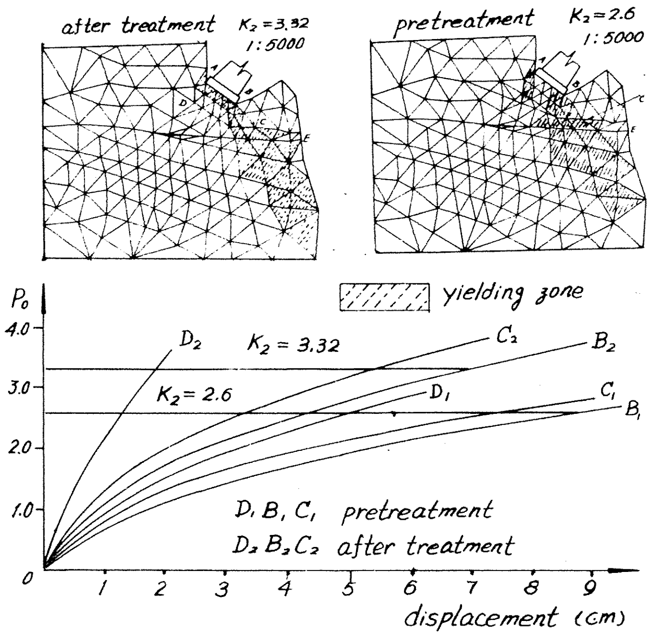

Figure 4
Calculations of stability by nolinear F.E.M.

In Fig 4, were shown the results of E-P F.E.M. they are right abutment of cross section at elevation 950m. One corresponds to pre-treatment and the other to post treatment.

We can obtain K_1, K_2 in the calculation process of load increament. The meaning of K_1, K_2 are the same as before mentioned. In Fig 4, the calculated K_1, K_2 were given. In calculation, the fracture, originated from tension stresses presented at upstream heel of dam, this correponds to K_1, afterwards, when loads increased, a yield zone presented in the downstream toe of dam, which originated from shear failure. Finally, the whole structure failed due to the occurrance of plastic zone covering the weak zone of rock, here the load correspond to K_2. In Fig 4, $K_2 = 2.6$ for pretreatment, $K_2 = 3.92$ after treatment. This shows ER-Dam abutments when pretreatment, requires reinforcement to keep adequate S.F. The yielding zone of rock presented in downstream toe of dam, usually threatens the stability of dam abutments where prestressed cables are needed to strengthen the rock masses.

4 SAFETY FACTORS

There are two kinds of concepts most freqnently used to estimate the safety of dams.

1). By safety factor we understand the inverse ratio existing between the resultant of the forces which are really acting on the foundation and another one, with the same direction and line of action of the forces, which provokes the sliding in rock foundation.

2). Ratio between the parameters defining the strength of the foundation rock and other theoretical parameters which under the loads that really act in the structure would lead to plastic equilibrium.

The first concept is based on a variation of the stressesapplied by the dam on the rock, and the second one is based on the variation of the rock properties. By the first S.F, failure of structure is due to an increase of the external forces applied to it, which usually used in scale model tests, including geomechanical model tests and elasto-plastic model in F.E.M. Whereas the failure of structuce by second conception is due to an overestimation of the rock strength or to an unwaited rock strength decrease in certain zones of the foundation, which is usually used in "rigid block method" and elastic model in F.E.M. as punctual safety factor. The first concept is referred to overload S.F. whereas the seeond is referred to strength capacity, it is difficult to say which kind of S.F. is better. For large dams as ER-Dam, over-load and overestimation of strength parameters are equally probable, so that these two kinds of S.F. have been adopted and studied in our work.

Two criteria K_1 and K_2 bave been adopted in this paper. These criteria in essence are established on the basis of deformation of structure. For large dams, by means of evaluation on deformation of structures under load, different alternatives can be compared and the preferable can be selected. In addition to these criteria, the permissible displacement criteria is also proposed in this paper, the displacement of abutments under complex geologic condition should be kept in a certain extent to the ideal state of dam that can be recognized permissible. For example in the ER-Dam, the maximum displacement of abutment under normal load should be less than 1.5cm which correspond to the elastic deformation of foundation of homogeneous sound rock.

5 CONCLUSIONS:

Geomechanical model test and F.E.M. are two approaches which available for determining S.F. of arch dam abutments.

Through comparison of S.F. of pretreatment and treatment, an reinforcement design can be verified. Whether in scale models or in F.E.M. models, the geological conditions and geological disturbances can be reproduced in them. Different mathematical and mechanical models can be utilized to simulate the rock masses: Elastic, Elasto-plastic, or Elasto-Brittle etc.

As shown in this paper, the S.F obtianed, according to different approches, are in good agreement. Their S.F. are inadequate for pretreatment of right abutment of ER-Dam. A reinforcement of it is needed: such as sufficient excavation into the abutment rock, prestressed cablesadopted in the critical zones, and widening the arches at the abutment, especially the weak rock zone should be treated.

The bounds of S.F. in elastic model and elasto-plastic model are different. Generally, the punctual S,F. is assumed to be 2.0, where the K =3.0 in elesto-plastic model.

For large dams, under complex geological conditions, it is necessary to study the mechanism of abutments under acting loads, including the behavior of dam, faults and some tectonic structures, which can't be offered by rigid block method. It is obvious that in such cases, the model tests and F.E.M. are more preferable than rigid block method.

6 ACKNOWLEDGMENTS

The authors are particulary indebted to the Research Group of ER-Dam, Department of Hydraulic Engineering, Tsinghua University, who hard work for the tests of models.

7 REFERENCES

CHOW WEI YUAN, and YANG RUO QIONG (1979). On rupture studies of FENG TAN Dam. Tsinghua University scientific report.

CHOW WEI YUAN, and YANG RUO QIONG (1981). The calculation of stresses around the hell of Hollow dam by means of F.E.M. Tsinghua University scientific report.

CHOW WEI YUAN, WANG YU TAL (1982). The Analysis of stability of arch dam abutments. Kwal Chu people's publication press.

CHOW WEI YUAN, and YANG RUO QIONG Finite element evaluation and experimental study with model blocks for Dam foundations stability. Selected Papers Symposium on High Arch Dam 1980. China.