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SYNOPSIS: This paper presents an analysis of the stability against sliding of a gravity dam built on a layered rock formation. During the design of this dam, detailed studies of the dam foundation were carried out from a viewpoint of rock mechanics, including laboratory and in-situ tests of the mechanical properties of rocks, calculations by the theory of limit equilibrium and FEM, as well as model tests. Based on these studies, the dam type was selected.

INTRODUCTION

The dam of Zhuzhuang reservoir is a stone masonry gravity dam. Its original designed height was 110m. The rock at the dam site is a layered silicarenite consisting of nine thick layers in total. The bedding surface dips downstream at an angle of 6-8°. There are many layers of weak intercalations amony which intercalations II-5 and Cn72 (Fig.1) are rather unfavourable to the sliding-resistance and hence the controlling layers for the dam foundation stability. In design, not only the safety factor for the stability against sliding of the foundation was calculated by the traditional theory of limit equilibrium, but also the foundation stability was studied by model tests and the stress in the foundation computed by FEM. Based on the result of model tests and consulting the results of limit equilibrium and FEM analysis, the dam type was selected and a dam height of 95m was determined. In model tests and FEM computation, the effect of fault F_4 which traverses the dam foundation was taken into consideration. The Zhuzhuang dam was completed in 1976 and has worked in a good state for more than 10 years.

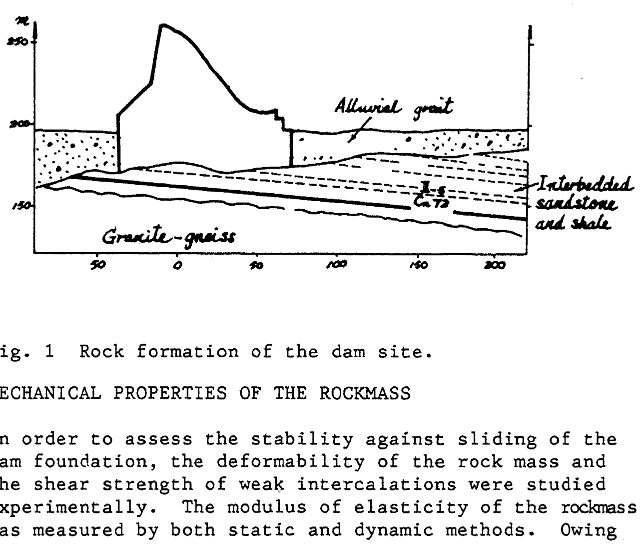

Fig. 1 Rock formation of the dam site.

MECHANICAL PROPERTIES OF THE ROCKMASS

In order to assess the stability against sliding of the dam foundation, the deformability of the rock mass and the shear strength of weak intercalations were studied experimentally. The modulus of elasticity of the rockmass was measured by both static and dynamic methods. Owing to various limilations such as the constructional operation etc., studies were carried out more by static methods than by dynamic ones and more on the surface than in the depth.

The static modulus of elasticity was measured in an adit with a jack and typical stress-deformation curves for loading-unloading cycles are shown in Fig. 2.

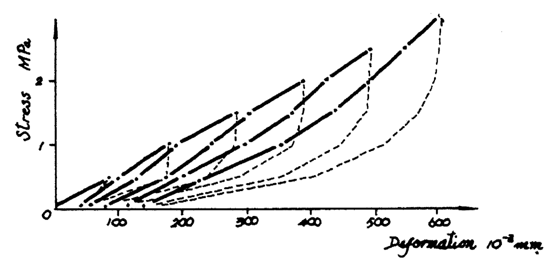

Fig. 2 Stress-deformation curves for rockmass

With the requirement for analysing the stress of the foundation in mind, deformations were measured not only in the vertical but also in the horizontal direction in the test.

Test results showed that for the silicarenite and striped sandstone, every loading-unloading cycle caused an obvious incremental residual deformation. The modulus of elasticity was calculated for each loading-unloading cycle. It was found that the higher the loading stress, the smaller the value of E was, and that the modulus of deformation was much lower than the modulus of elasticity for the same loading level. To the maximum, they may differ by a factor of 3. This was more conspicuous in the horizontal than in the vertical direction. The values of E in the horizontal direction were mostly greater than those in the vertical, and the degree of anisotropy decreased with increasing stress. The anisotropy of the rockmass was mainly brought about by the effects of the occurrence of various discontinuities, the state of fractured zones and the properties of fill-in materials.

Virgin specimens 50×50^{cm} and 15×15^{cm} in size were used in-situ and in laboratory respectively to determine the shear strengths of weak intercalations. Besides, some disturbed soil was collected from the clayey intercalations No. III to VI and remoulded specimens with controlled water content were prepared and tested by consolidated quick shear tests in direct shear apparatus. 14, 16 and 106 groups of tests were conducted respectively for the three kinds of tests mentioned above.

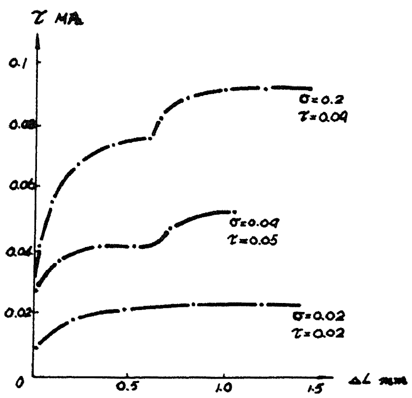

Fig. 3 Typical stress-displacement curves for clayey intercalation NO. II-5.

Typical stress-deformation curves for the clayey intercalation II-5 are shown in Fig. 3. It was found from the test result that the clayey intercalation was characterized by large deformation and low strength. Its shear strength depended on the roughness of the interface and the thickness of intercalation. Four different cases were identified. First, when the interface was smooth and the intercalation thickness was greater than the interface fluctuation, the shear strength was very low since shear fracture took place along the weak interface between the rock and intercalation. Second, when the intercalation thickness was smaller than the interface fluctuation, the shear strength was controlled by the interlocking between the rock surfaces and had a higher value. Third, when the intercalation was locally lacking, the shear strength was increased significantly by the friction between rock surfaces. Finally, when the rock had a zigzag surface, a portion of it was sheared off and thus the shear strength was greatly increased.

The shear strengths of intercalations obtained by laboratory tests on 15×15^{cm} specimens and by in-situ tests were close to each other, while those by geotechnical tests were generally lower than both of them. When the rock surfaces above and below the intercalation were smooth or only had a small fluctuation, the laboratory data were close to or slightly greater than the in-situ data.

According to the yield limit criterion, the peak strength of intercalations II-5 and Cn72 were multiplied by a reduction factor of 0.7 and f=0.29, 0.22 and C=0 were taken for these two layers respectively.

APPLICATION OF THE THEORY OF LIMIT EQUILIBRIUM

The weak intercalation dipping downstream was assumed as a sliding plane (BE) when using the theory of limiting equilibrium to analyze the stability against sliding in design. It was also assumed that there was a vertical shear fracture plane CE (Fig. 4) in the dam site. Since the most developed fracture perpendicular to the river bed in the dam site has a high dip angle of 80°, this assumption seems to be a reasonable approximation of the reality. By assuming that the rockmass behind the dam fractures along plane CD when the sliding plane has reached its limiting state and making use of limit equilibrium analysis, the force R transferred to the downstream rockmass can be obtained as

$$R = \frac{\Sigma H\cos\theta + (\Sigma V + G_1)\sin\theta - f_1[(\Sigma V + G_1)\cos\theta - \Sigma H\sin\theta]}{\cos(\varphi-\theta) - f_1\sin(\varphi-\theta)} \quad (1)$$

Then by assuming that the fracture plane CD in the rockmass behind the dam makes an angle with the horizontal, K can be obtained from

$$K = \frac{f_2[R\sin(\varphi+\alpha) + G_2\cos\ ]}{R\cos(\varphi+\alpha) - G_2\sin} \quad (2)$$

A series of α values were assumed for trial calculation and the one which gives the least value of K was used to determine the plare CD. That K value so obtained was considered as being the safety factor of stability against sliding. In the above expressions, ΣV is the sum of the vertical forces above the calculation plane AE; G_1 the dead weight of rockmass ABCE; G_2 the dead weight of rockmass CDE; f_1, f_2 and f_3 the coefficients of friction for the weak intercalation, the first and second fracture planes respectively, among which f_1 was obtained experimentally while f_2 and f_3 were assumed; θ and α are the angles which the weak intercalation and the first fracture plane make with the horizontal; while φ gives the direction of R (the angle it makes with the horizontal) and is taken to be $\varphi = \arctan f_3$ here.

In some sections, e. g., the overflow dam section, cut-off were excavated on the upstream side. The concrete of the dam body there is in direct contact with the sandstone below the weak intercalation. For those sections, f=0.55 and C=0 were used in calculation. Since the coefficient of friction for the sliding plane varies from section to section, the total load was distributed to the subsections according to the vertical stresses calculated by the mechanics of materials.

Even if the safety factor was high enough to fulfill the requirement of design specifications, the problem was still not ultimately solved for the theory of limit equilibrium was too simplified to reflect the mechanical behaviour and failure mechanism of a loaded formation so complex as in Zhuzhuang reservoir. For this reason, model tests and finite element computation were also used in design to assess further the stability against sliding of the dam foundation.

APPLICATION OF STRUCTURAL MODEL TESTS

This item was mainly carried out in relation to the selection of dam type. Five schemes in total were made for comparison. Only the two intercalations II-5 and Cn72 and the fault F_4 were simulated in our tests.

There are two independent scale factors in the law of similarity for static model tests.
They are:

(1) the length ratio

$$C_L = \frac{L_P}{L_M} \quad (3)$$

and

(2) the stress ratio

$$C_\sigma = \frac{\sigma_P}{\sigma_M} \quad (4)$$

where L and σ represent the geometrical dimension and stress while the subseripts "P" and "M" refer to prototype and model respectively. The volume weight of solid and liquid materials has a dimension of $[\rho] = [\sigma][L^{-1}]$ and hence its scale factor is

$$C_\rho = \frac{\rho_P}{\rho_M} = \frac{\sigma_P L_P^{-1}}{\sigma_M L_M^{-1}} = \frac{C_\sigma}{C_L} \quad (5)$$

Since the problem to be solved in this test was that of the shear failure in weak intercalation and surrounding rock, the condition of strength similarity should be

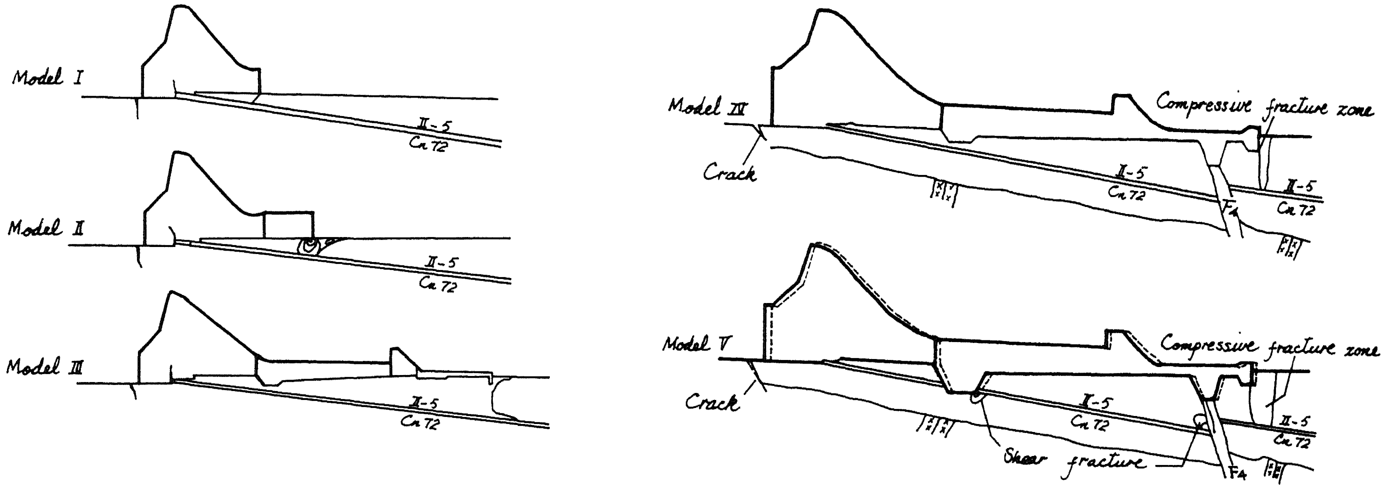

Fig. 5 The mechanism of failure of test models

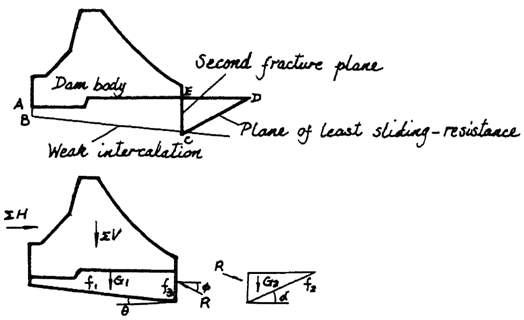

Fig. 4 The force system for limit equilibrium calculation

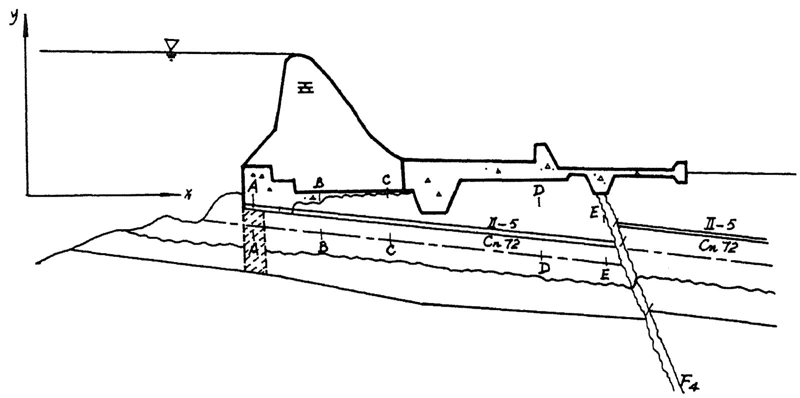

Fig. 7 A schematic diagram showing the sections for computation

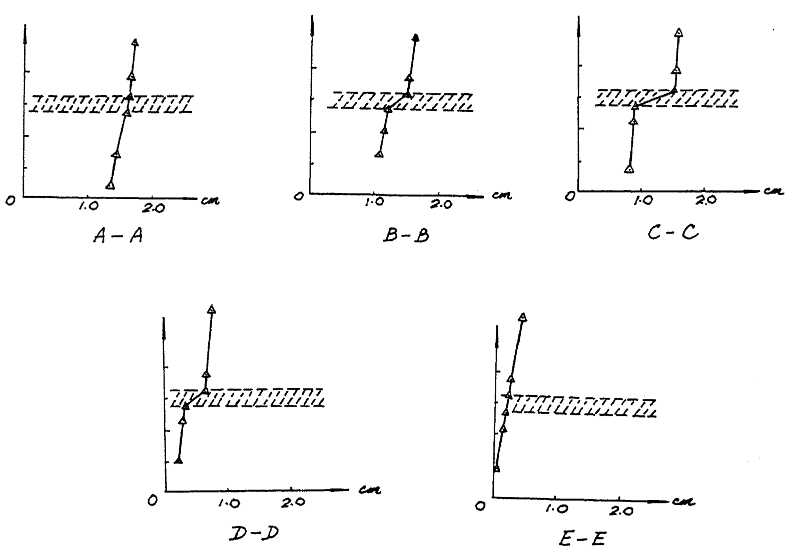

Fig. 8 Displacement distribution in the intercalation along sections A-A, B-B, C-C, D-D, and E-E, where shaded areas show the position of intercalations II-5 and C_n 72

satisfied. The strength ratio is just C_σ, i. e., all the compressive, tensile and shear strengths must have a ratio of C_σ. The model of Zhuzhuang dam only simulated the shear strength of weak intercalations.

The values of C for intercalations II-5 and Cn72 obtained from in-situ tests were very small and hence were taken to be zero in the model. Furthermore, since f is dimension-less, the law of similarity can be satisfied by the shear strength of intercalations provided $f_P = f_M$.

$$[\tau] = C + f \ .$$

The strength of the host rock and the deformability of the intercalation were not simulated. The lower and upper limit safety factors K_1 and K_2 were obtained from the overload capacities for the initial sliding and limiting state. Among them, K_1 had a higher level of approximation than K_2.

Five models were made. They were (see Fig. 5):

Model I: in the foundation of the dam type originally designed before finding II-5 and Cn72, cut-off passing through these two intercalations was excavated in front of the dam;

Model II: the same as model I except that the dam back was thickened;

Model III: underflow-overflow scheme;

Model IV: developed from model III with fault F_4 stabilized by fault plugs and shallow cut-off excavated at the heading part of the apron;

Model V: it differed from model IV in that a deep cut-off cutting out the two intercalations was excavated at the heading part of the apron.

The degrees of safety for the five schemes mentioned above are listed in Table 1 where the safety factor is the ratio of the actual to the design load and the lower and upper limit safety factors refer to the overload factors when the intercalation begins to slide and reaches its limiting state respectively.

Table 1 Safety factors for the five models

Test scheme	Lower limit safety factor K_1	Upper limit safety factor K_2	K_2/K_1
I	0.84	1.72	2.25
II	0.96	2.56	2.69
III	1.49	5.08	3.41
IV	1.79	5.97	3.34
V	1.84	10.01	5.44

Both the lower limit safety factors for model I and II were less than 1. Cracks appeared at the dam heel of both models. The crack was vertical in model I while slightly inclined downstream in model II. Vertical cracks developed in the dam body at the cut-off, i. e., where the host rock changes its shape abruptly. The crack in model II was deeper. The downstream rockmass above the weak intercalation ruptured completely.

Models III to V were all underflow overflow schemes. Model III did not simulate the fault F_4, while models IV and V lowered the dam height by 10m. The lower limit safety factors for all these three models were greater than 1 and model V showed the highest potential safety.

In model III, a vertical upward crack first developed in the dam body where the host rock changes abruptly, then cracking took place at the dam toe and finally the host rock above the intercalation ruptured at the end of the apron.

The modes of failure of models IV and V are shown in Fig. 5. In model V, obvious overall movement of the host rock between the middle cut-off and fault F_4 was observed when ultimate failure took place. This indicated that by excavating a deep cut-off at the middle part to cut out the two intercalations, the integrity of the rockwass above II-5 was increased and hence a greater portion of horizortal thrust was transmitted downstream by the rockmass below Cn72. It was found from the model deformation measured at the heel (Tab. 2) that the deformation of model V was less than that of model IV. In Tab. 2, Po denotes the design load and P the load actually applied to the model during test. In the elastic range, deformations in the two models were very close to each other. However, after the primary cracking, the difference between the deformations in two models was not proportional to their load increments. In a certain extent, this was related to the nonlinearity of materials, but the main reason was that the fracture surfaces in two models propagated in different manners after initiation.

Table 2 Displacements at the dam heel of models IV and V

Model	P/Po	Heel displacement (mm)	Remark
IV	1.00	0.034	
	1.79	0.060	Primary cracking
	2.42	0.123	
	2.93	0.177	
	3.79	0.802	
	5.97	2.000	Ultimate failure
V	1.00	0.032	
	1.84	0.054	Primary cracking
	2.73	0.103	
	3.52	0.175	
	6.03	0.473	
	10.01	5.000	Ultimate failure

APPLICATION OF FEM

Triangular elements were used for the dam foundation. The weak intercalations II-5 and Cn72 were treated as layered media, i. e., in the normal direction of the layer, only compressive stress can be transmitted whilst tensile stress can not exist and the shear stress transmitted by the clayey intercalations can not exceed $|f\sigma_n|$.

The elastic stress field $\{\sigma_0\}^e$ was computed first by considering the layered media as being homogeneous and elastic. Then the result was compared with the strength criterion of the clayey intercalation. There were three possible cases, namely,

(1) $\sigma_{oy'} > 0$, the intercalation was broken by tension and thus lost its ability of transmitting shear stress;

(2) $\sigma_{oy'} < 0$ and $|\tau_{ox'y'}| < -f\sigma_{oy'}$, the layered medium was in its elastic range and the result of first computation gave the solution of the problem.

(3) $\sigma_{oy'} < 0$ and $|\tau_{ox'y'}| > -f\sigma_{oy'}$, the shear stress was higher than the limit value and was actually impossible. The actual state of stress ought to be

$$\{\sigma_{t'}\} = \begin{Bmatrix} \sigma_{ox'} \\ \sigma_{oy'} \\ |f\sigma_{oy'}| \end{Bmatrix} \qquad (6)$$

In this case, adjustment of the difference between the computed and actual stresses

$$\{\sigma\}^e = \{\sigma_o\}^e - \{\sigma_{t'}\}^e \qquad (7)$$

was necessary.

Using the initial stress method for solving nonlinear problems, linear equations

$$\{\delta_i\} = [K]^{-1}\{R_i\} \quad (i=0,\ 1,\ 2,\ldots) \qquad (8)$$

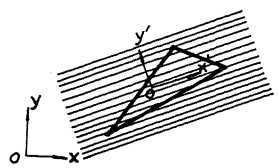

Fig. 6 Coordinate systems for finite element analysis

were solved first. $\{\delta_o\}$ was obtained using actual applied load $\{R_o\}$,

$$\{\delta_o\} = [\ K\ ]^{-1}\ \{R_o\} \tag{9}$$

and the elastic stress $\{\sigma_o\}$ was computed. The calculated $\{\sigma_o\}$ was compared with the above-mentioned judging standard and the corresponding stress difference $\{\sigma_o'\}$ was obtained. Then the unbalanced equivalent nodes forces were computed by

$$\{F_o\} = \int [B]^T\ \{\sigma_o\}\ tdxdy \tag{10}$$

and iteration was repeated using the same procedure by

$$\{\delta_1\} = [\ K_o\ ]^{-1}\ \{R_o + F_o\} \tag{11}$$

until

$$\{\delta_n\} = [\ K_o\ ]^{-1}\ \{R_o + F_n\} \tag{12}$$

resulted in a diplacement difference

$$\{\delta_n\} - \{\delta_{n-1}\}$$

less than the permissible deviation.

In computation, 11 materials of different properties were used and some 900 elements were divided (Fig. 7) which were the main part of the computed region. It was known from the result of computation that horizontal tensile stress occurred in most elements within in the dam heel zone. Owing to the low tensile strength of the rockmass, cracks with high dip angles developed especially in the dam foundation. Therefore, in the first adjustment, cracking along a certain interface between elements was assumed (cracking along a vertical interface at the dam heel was assumed in actual computation). Along that interface, original nodes were replaced by twin-nodes while the interface was considered as a free boundary.

By using the above two methods to treat the intercalations and the dam heel zone in tension, stress distributions in the dam foundation for a number of load combinations were computed. The occurrence of tensile stress at the dam heel was consistent with the cracking in test models. As for the slip in intercalations, no complete data were available for comparison. In computation, the initial slip often occurred between the dam heel and toe. The location of initial slip was not recorded in model tests. According to the modes of ultimate failure, it seemed that the slip in intercalations was greater in the upstream side than in the downstream. The result of stress computation showed that the deformation of the weak intercalation was greater in its upper part than in the lower part. In particular, the slip between the upper and lower parts in section C-C was the greatest (see Figs. 7 and 8). On the curtain (A-A in Fig. 8) and near the fault F_4, the deformation in the upper and lower parts of the weak intercalation varied linearily. This fact implied that no slip took place there. The result of computation indicated that for the calculated sections (corresponding to mokel IV in Fig. 5), the dam foundation was in a stable state under the action of design load.

The stress field computation by FEM is very beneficial to the selection of dam site and directive for the dam foundation treatment. However, when using it to analyze the stability of against sliding of dam foundations, there are still some problems need to be studied further.. At present, the criterion for judging the stability against sliding has not been well established yet. Some ones use the ratio of the total sliding-resistance capability ($\Sigma f\sigma$ +LC) and total sliding force ($\Sigma\tau$) to determine the safety factor, while others use the vertical and horizontal loads regardless whether the weak intercalation slides or not. For the Zhuzhuang reservoir, its stress field was computed by the above-mentioned nonlinear FEM while its stability against sliding was still assessed by the ratio of the total sliding-resistance to the total sliding force.

According to the result of model tests, the safety factor of the stability against sliding for the dam type of model III was high enough to fulfill the design requirement. However, after a comprehensive consideration based upon the analyses of limit equilibrium and FEM stress computation, the dam type of scheme IV was decided with its dam height 5m lowered, i. e., the actual dam hight for the finally selected scheme was 95m.

DISCUSSION ON THE SAFETY FACTOR OF AGAINST SLIDING STABILITY

The overloading factor was used as the safety factor in model tests. K_1 is an overloading factor for the clayey intercalation to slide, or the lower limit safety factor; and K_2 is an overloading factor for the foundation to lose its load-bearing capacity due to failure, or the upper limit safety factor.

In principle, according to the theory of limit equilibrium, values of C and f should be determined from the ultimate strength criterion. The safety factor estimated from them is the upper limit safety factor K_2. Though K=1.1-1.0 (K=1.0 is allowable for some special load combination) has been specified in the design specifications of China, it is insufficient to insure the safe operation of the dam foundation and hence the strength parameters have to be modified. In our design, values of C and f were determined from the yield criterion. In addition to the yield criterion, there are also many other prevailing criteria for evaluating C and f, such as these of the proportional limit, residual strength, test body uplift, maxinum displacement, rheological strength, etc.. So many methods of evaluation have caused serious difficulty and confusion for the choice of strength parameters.

The FEM is a powerful tool for stress analysis. However, it has not yet been brought into full play in assessing the stability against sliding. In fact, it has become possible to determine the lower limit safety factor K_1 from stress distribution. With increasing understanding about the process of sliding failure of layered rock formation and with the establishment of proper mechanical models, it will also be possible to calculate the upper limit safety factor K_2 which can reflect the actual status better than limit equilibrium analysis.

In view of the permissible working conditions of the foundation, it is relatively reasonable to control the shear stress in the clayey intercalation so that slip can not take place in it. Meanwhile, since the failure mechanism of the dam toe is still not very clear, to use the lower limit safety factor K_1 can avoid the disadvantage of assuming a failure mechanism arbitrarily. In the future, even when the upper limit safety factor can be computed rather accurately and reliably by simulating the failure process step by step, the lower limit safety factor will still be one of the basic parameters for assessing the safety of dam foundation.

Since the limit equilibrium method is the recommended method in design specifications of gravity dam in China, its reasonable use should be limited to the determination of the upper limit safety factor K_2. A question arising here is that when a certain stress leading to the initial sliding of the clayey intercalation is used to calculate the lower limit safety factor K_1,

$$K_1 = \frac{f(\sigma_n - u) + C}{\tau} = 1.1 - 1.0, \qquad (13)$$

what a value should be assigned to K_2 so that it can match K_1 ? In eq. (13), σ_n and τ denote the normal and shear stresses in the claysy intercalation and u the water pressure in cracks. It was the model test which gave us some inspiration. Table 1 gives the overloading factors for initial sliding and ultimate failure of the five dam types. It can be seen that $K_1 < 1$ and $K_2/k_1 > 2$ in dam type II while $K_1 > 1$ and $K_2/K_1 > 3$ in dam type III. Inferring from these limited preliminary results, K_2 should lie around 3 when K_1 is taken to be 1.1 – 1.0.

Therefore, K_2 can be considered as being in the range of 4 – 2, i. e.,

$$K_2 = \frac{f\ (W - U) + CA}{H} = 4 - 2\ , \qquad (14)$$

where W and H are the vertical and horizontal components of the total load, U the buoyant force and A the area of clayey intercalation.

Owing to the limited data of model tests and to the lack of systematic data concerning the strengths for initial fracture and ultimate failure in direct shear tests, taking $K_2 = 4 - 2$ was only based on our understanding at that time.

CONCLUSIONS

For analyzing the stability against sliding of dam foundations, the limit equilibrium method is simple and straightforward but too simplified to take many complicated geological factors into consideration. Model tests can provide directly visible phenomena and are convenient for studying the possible modes of failure and load-bearing capacity of dam foundations. However, because the stress-strain relationship of the weak intercalations was not simulated in our models, the test result was still an approximation. The FEM can be used to compute the stress field and to examine the relative shear deformation in weak intercalation, but there are still some difficulties when using it to assess the stability against sliding of dam foundations. For the dam discussed in the present paper, various schemes of the dam type were compared through model tests, the safety factors for stability against sliding were determined by limit equilibrium analyses and stress and deformation of the dam foundation were computed by the FEM. On these bases, the dam type and height were decided and the safety of stability against sliding of the dam foundation was insured. The dam has worked in a good state for 10 years.

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