SEISMIC SAFETY EVALUATION FOR CONCRETE GRAVITY DAMS

CHENBIN DU[1] YONGWEN HONG[2] JUWEI YUAN[3] ZHIMING LIU[4]

[1]*Professor,Dept. of Engineering Mechanics, Hohai University, Nanjing. China*
[2] *Senior Engineer, Kunming Design and Research Institute for Hydroelectric Projects, Kunming. China*
[3]*Doctoral student, Dept. of Engineering Mechanics, Hohai University, Nanjing. China*
[4]*Doctoral student, Dept. of Engineering Mechanics, Hohai University, Nanjing. China*
Email:cbdu@hhu.edu.cn, hong_yw@khidi.com, yjw@hhu.edu.cn, zhimingliu@hhu.edu.cn

ABSTRACT:

The design of aseismic reinforcement and the seismic safety evaluation for Jinanqiao roller compacted concrete (RCC) gravity dam are discussed in the paper. The results indicate that it is not suitable to determine the amount of reinforcement by the elastic stress at key locations of dam. For the design of the steel reinforcement it is required that the crack does not damage the grout curtain in the dam heel, and cracks at the corners do not have any effect on the stability of the dam. The stress that located near stress concentration is chosen to determine the initial amount of reinforcement, and the final reinforcement is determined by the results of nonlinear finite element analysis of the dam. The separated reinforced concrete nonlinear finite element model is used to analysis the earthquake response of the dam. The strain rate, damage variable and stiffness degradation are included in the concrete elastic-plastic damage model. The ideal elastic-plastic constitutive relationship is adopted in the analysis. Aiming at the possible slip modes of the powerhouse dam section and the dam damage after considering the nonlinear earthquake response, aseismic stability evaluations of the dam are analyzed. The results show that the aseismic stability of the powerhouse dam section of Jinanqiao gravity dam meets the requirements of China Code. The reinforcement scheme provides basis for the design institute to determine the final reinforcement scheme.

KEYWORDS: gravity dam, concrete damage, aseismic reinforcement, dam heel

1.INTRODUCTION

For the aseismic reinforcement design and seismic safety evaluation of the concrete gravity dam, conventional linear-elastic response spectrum method cannot meet the requirement of engineering design. More and more experts and scholars realize that nonlinear FEM procedures are needed for the analysis of the behaviour of dams during strong earthquakes. The linear-elastic method cannot reflect the absorption of the earthquakes loading energy due to the plastic of concrete, the elastic stress may be higher than that of the real situation. It is not only unpractical, but also unnecessary that reinforces totally result from the static load and the analysis results of dynamic Code Spectrum. For the Xinfengjiang gravity dam, design earthquake load was reduced by old China Codes (NHCE 1978). That means the earthquake load effect after multiply 0.25 adds to the static load effect, then reinforced design was carried out according to the non-member system reinforced concrete structure. The dam is still running well after suffered its design earthquake (Huang 1989). It is enough to show that aseismic reinforcement design of the gravity dam according to this method is feasible.

According to the seismic criterions of gravity dams, when suffering the rare earthquake, the dam’s crack is acceptable, as long as the dam meets the demands of bearing capacity, which means no dam-failure occurs (Lin 2001, Chen 2005). Now, the problem is that there is clear explain to the aseismic reinforcement design of gravity dam neither in the Seismic Design Code of Hydraulic Structures (DL5073-2000) (CIRWH 2000) nor in the Design Code for Hydraulic Concrete Structures (SL/T 191-96) (NHCE 1997) in China. In the former, there is just general statement of “should enhance the concrete strength or reinforced near the dam’s top” without specific aseismic reinforcement formula. In the latter, the reinforced method is not suitable for the concrete dam is pointed out.

The design of aseismic reinforcement and the seismic safety evaluation for Jinanqiao roller compacted concrete (RCC) gravity dam are discussed in this paper. The RCC gravity dam is 160 meters high, and the dam site lies in the middle reaches of the Jinshajiang River in Yunnan province in China. The reinforced concrete nonlinear finite element time-history method is used to analyze the earthquake response of the dam. The strain rate, damage variable and stiffness degradation are included in the concrete elastic-plastic damage model. The ideal elastic-plastic constitutive relation is adopted in analysis. For the design of the steel reinforcement it is required that the crack does not damage the grout curtain in the dam heel, and cracks at the corners do not have any effect on the stability of the dam. Final amount of reinforcement is determined by the nonlinear earthquake response results of dam. The reinforcement scheme provides the basis for the design institute to determine the final reinforcement scheme. Aiming at the possible slip modes of the powerhouse dam section and the dam damage after considering the nonlinear earthquake response, aseismic stability evaluations of the dam are analyzed.

2. PROJECT SURVEY

The type of concrete dam of Jinanqiao hydroelectric station under construction is a roller compacted concrete gravity dam, the crest length is 640 m. The lowest dam foundation elevation is 1264 m, and the top elevation of dam is 1424 m. The maximum dam height is 160 m. The project lies in the Lijiang platform margin foldbelt of Yangtze paraplatform western rim, which is strong activity area of the northwest of Yunnan and the southeast of Qinghai-Tibetan plateau. The engineering geology environment condition of this area is very complicated. The river valleys are usually very deep cutting valleys, and the bank slope is very steep. Regional earthquake is very active. According to the result of seismic safety evaluation finished by the Institute of Geology, China Earthquake Administration, the earthquake basic intensity at dam site is Ⅷ, the fortification intensity is Ⅸ, the peak of acceleration at bedrock is 0.399g.

The powerhouse dam section is chosen to be investigated in the paper. The length of transverse joints is 35 m. The bedrock of the dam site is mainly basalt, others are block-fractured, fracture chloritization rock mass and weak seams t_{1a} and t_{1b} made from tuff. The material of dam is mainly roller compacted concrete ($C_{90}20$). There is a little normal concrete ($C_{90}25$) used in the upstream and downstream surfaces and the dam base. In order to calculate stress of the intake, three-dimensional model is chosen. The depth of dam base is 480 m, which is three times of height of dam. The constraints around the base are all normal constraints. The base is considered to be a massless foundation. The dam body and the foundation are divided by 8-node hexahedron. Total number of nodes of the powerhouse dam section is 32614, and the number of elements is 27092. The finite element model of dam body and foundation is shown in Fig.1. Typical dam section is presented in Fig.2. The loads considered in the calculation include self-weight, hydrostatic pressure, sediment pressure, uplift pressure, wave pressure, and the earthquake effect. The hydrodynamic pressure is determined by Westergarrd method that takes the extra mass in the upstream dam face into consideration.

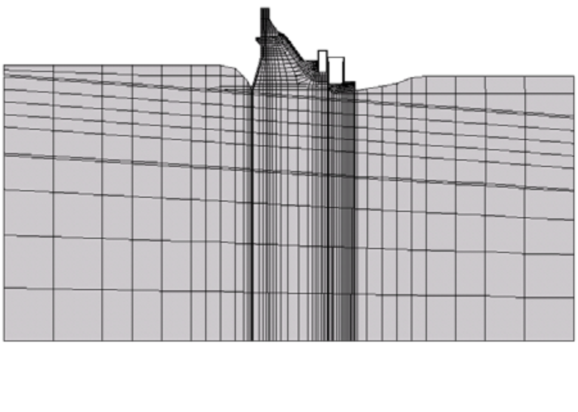


Figure 1 Finite element model of powerhouse dam section

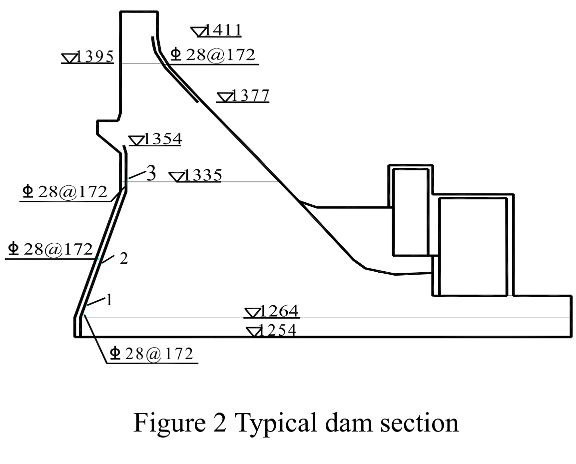


Figure 2 Typical dam section

3. ELASTIC-PLASTIC DYNAMICAL DAMAGE MODEL OF CONCRETE MATERIAL

On the base of the plastic damage model presented by Lubliner(1989) and the concrete plastic damage model under cycle repeat load proposed by Lee and Fenves(1998), the effect of strain rate on the plastic deformation is taken into consideration, concrete elastic-plastic damage model with rate-related is obtained.

The concrete elastic-plastic damage model can be summarized by the following:

$$\sigma = (1-d)\bar{\sigma} \tag{3.1}$$

$$\bar{\sigma} = D_0^{el}(\varepsilon - \varepsilon^{pl}) \tag{3.2}$$

$$\dot{\tilde{\varepsilon}}^{pl} = h(\bar{\sigma}, \tilde{\varepsilon}^{pl}) \bullet \dot{\varepsilon}^{pl} \tag{3.3}$$

$$\dot{\varepsilon}^{pl} = \dot{\lambda}\frac{\partial G(\bar{\sigma})}{\partial \bar{\sigma}} \tag{3.4}$$

the Eqn.3.1 defines the effective stress with damage, Eqn. 3.2 defines the connection of effective stress and elastic strain, the Eqns. 3.3 and 3.4 define the plastic behavior. $\tilde{\varepsilon}^{pl}$ is equivalent plastic strain, $\dot{\tilde{\varepsilon}}^{pl}$ is equivalent plastic strain rate, λ is plastic multiplier, and *G* is plastic potential function.

The concrete stress-strain relationship with the repetitive load is shown in Fig.3. In this model, the concrete elastic-plastic yield surface (shown in Fig.4) adopts the formula presented by Lee and Fenves (1998):

$$F(\bar{\sigma}, \tilde{\varepsilon}^{pl}) = \frac{1}{1-\alpha}[\bar{q} - 3\alpha\bar{p} + \beta(\tilde{\varepsilon}^{pl})\langle\hat{\bar{\sigma}}_{\max}\rangle - \gamma\langle-\hat{\bar{\sigma}}_{\max}\rangle] - \bar{\sigma}_c(\tilde{\varepsilon}_c^{pl}) \tag{3.9}$$

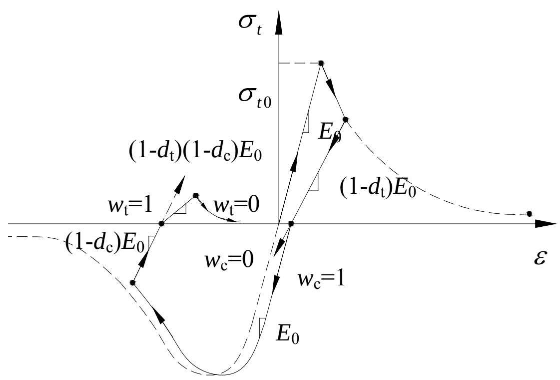


Figure 3 Stress-strain relationship of concrete under cycle loads

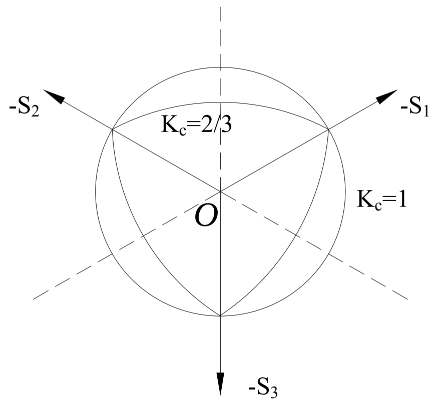


Figure 4 yield surface in the deviatoric plane with different values of K_c

where $\beta(\tilde{\varepsilon}^{pl}) = \frac{\bar{\sigma}_c(\tilde{\varepsilon}_c^{pl})}{\bar{\sigma}_t(\tilde{\varepsilon}_t^{pl})}(1-\alpha)-(1+\alpha)$, $\bar{\sigma}_c$ and $\bar{\sigma}_t$ are the effective cohesive stress in compression and in tension, respectively. $\alpha = \frac{\sigma_{b0}-\sigma_{c0}}{2\sigma_{b0}-\sigma_{c0}}$, σ_{b0} and σ_{c0} are the initial yield stress of biaxial and uniaxial compressive load, respectively. α is in 0.08-0.12. $\lambda = \frac{3(1-K_c)}{2K_c-1}$, for concrete material, parameter K_c can be taken as 2/3, that means λ = 3.

The relation of K_c and the shape of yield surface in the deviatoric plane is shown in Fig.4.

The non-associated flow rule based on the Drucker-prager flow surface is considered in the calculation.

4. THE PRINCIPLE OF ASEISMIC REINFORCEMENT DESIGN

The principle for the dam reinforcement is that it is required that the crack does not damage the grout curtain in the dam heel, and cracks at the corners do not have any effect on the stability of the dam.

If the steel amount is determined by the non-member system reinforced concrete structure reinforcement principle according to the Design Code for Hydraulic Concrete Structures, we have:

$$T \leq \frac{1}{\gamma_d}(0.6T_c + f_y A_s) \tag{4.1}$$

where T is the elastic total tension force calculated by the design load that including static load and earthquake effect multiplied 0.35, T_c is the tension resultant force that concrete bears, $T_c = A_{ct} b$, where, A_{ct} is resultant force of the section whose tensile stress less than the concrete dynamic tensile strength, and b is the design section thickness. f_y is the dynamic design strength of steel bar.

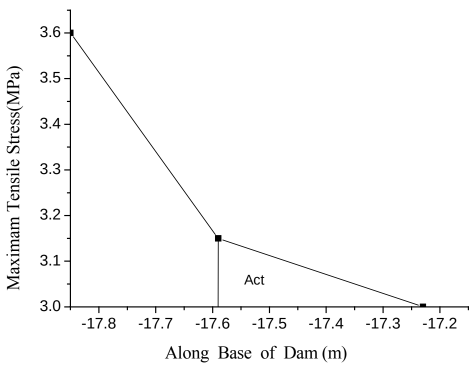
According to the NHCE (1997), the structure coefficient $\gamma_d = 1.2$. Concrete bears 30% of the total tension force, that is $T_c = 0.3T$. Then we have:

$$1.02T \leq A_s f_y \tag{4.2}$$

The maximum tensile stress is 3.5 MPa at the 5 m far from the dam heel, the maximum tensile stress at the 6.46 m is 3.15MPa.

The number of steel bar can be calculated by the Eqn. 4.2 is 287903mm^2, need 282 steel bars of Ⅱ grade with the diameter 36 mm are needed here.

Obviously, the number of steel bar is very large. It is not only unreasonable to need so many steel bars, but also is unnecessary. The large tension stress around dam heel and corners of the dam is caused by stress concentration. The result of field test indicates that the stress around these corners is far less than the result of linear-elastic calculation. The large tension stress related to assumption that the rock and concrete around the dam heel or the changed slopes are ideal elastic solid. In fact, the rock and concrete all have some micro cracks, the large tensile stress will be released due to these cracks. Tensile stress is not so large at these places in real case. It is not suitable to determine the amount of reinforcement by the elastic stress at key locations of the dam.



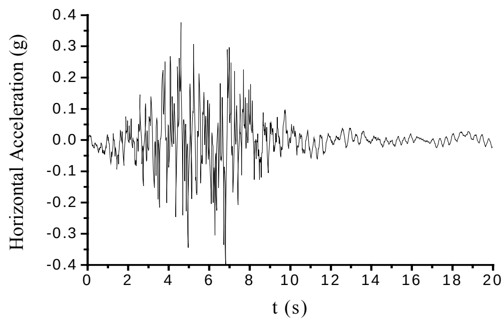


Figure 5 Maximum tension stress along dam base Figure 6 Earthquake waves adopted in calculation

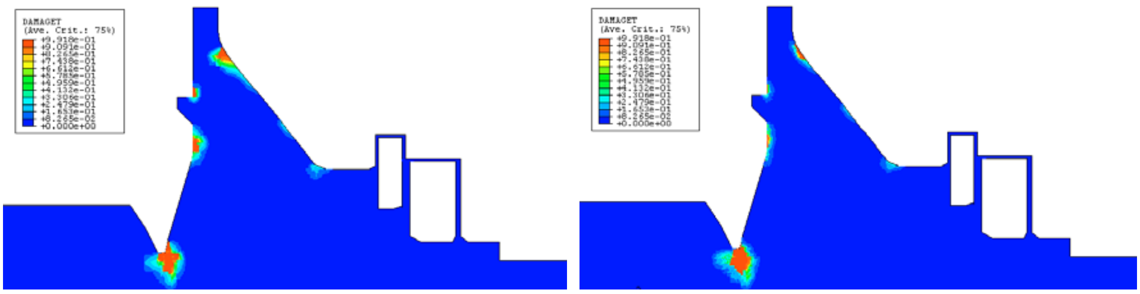
The stress should choose from the place away from the corners. Taking the dam heel for example, the elastic stress at 1269.5 m that is 5 m away from dam heel is chosen. Maximum tensile stress along the base is shown in Fig.5. Here, the concrete is roller compacted concrete C_{20}, dynamic tensile strength $f_c = 3.15$MPa. The structure coefficient $\gamma_d = 1.2$, the steel bar design strength f_y is 335MPa.

The number of steel bar can be calculated by the Eqn. 4.2 is 2612.9mm^2, need 5 steel bars of Ⅱ grade with the diameter 28 are needed here.

At the same time, the nonlinear time-history response analysis with the above model, in which the steel bars elements and concrete elements are completely separated, is carried out. The earthquake acceleration in calculation is shown in Fig.6. The vertical earthquake acceleration peak value is taken 2/3 of that of horizon acceleration. The reinforcement steel is simulated by the ideal elastic-plastic model. Concrete is simulated by previous the elastic-plastic damage model. The results show that when reinforcement according to 5Φ28 or 5Φ25 on the dam heel and the upstream and downstream changed slopes, the crack damaged depth at the upstream changed slope is 1.3m to 1.5m, the crack damaged depth at the downstream changed slope is 2.5m to 3.0m, there is no breakthrough crack. The crack damaged depth at the dam heel is 3.8m to 5.0m, it does not damage the grout curtain in the dam heel. The maximum tensile stress of steel appears at the dam heel, it is about 265 MPa to 288MPa, which is less than the yield strength of steel. Comparison results with different reinforced scheme are shown in table 4.1. Comparing with without reinforcement, the area of tensile stress drops from 20m to 12m. The crack damaged depth of the dam heel and the changed slopes obviously reduced after reinforced. Figs.7 (a), (b) are the damage cracking profiles for cases without reinforcement and with 5Φ28 reinforcement, respectively. Figs.8 (a), (b) are the time-history curves of steel tensile stress with reinforced 5Φ25 at different elevations.

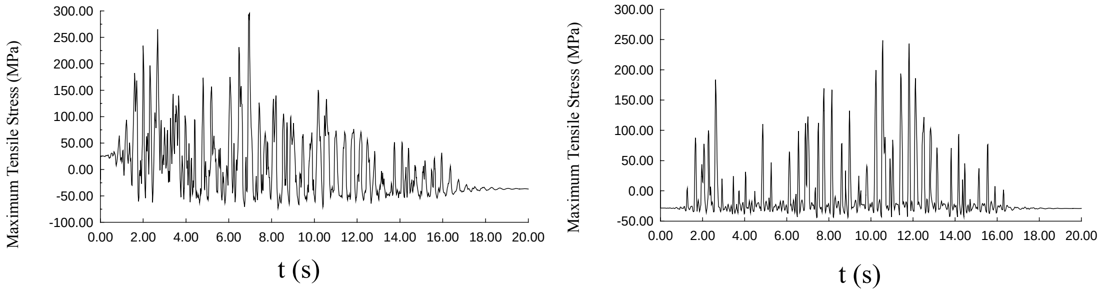
Table 4.1 comparison of different reinforcement scheme

reinforcement scheme	depth/m (Damage degree D=0.9)			The maximum steel tensile stress /MPa
	Dam heel	Upstream changed slope	Downstream changed slope	
Without reinforcement	9.0	3.0	7.0	/
5Φ25	5.0	1.5	3.0	288.0
5Φ28	3.8	1.3	2.5	265.0
7Φ28	3.0	1.0	2.0	202.0



（a） without reinforcement (b) 5Φ28@172

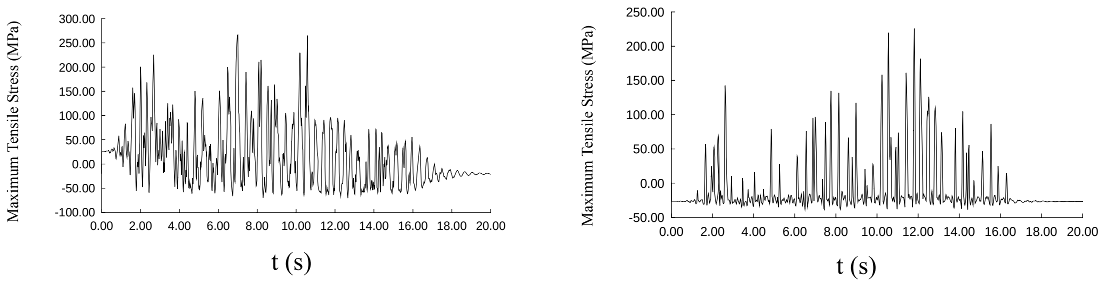
Figure 7 Damage cracking depth of dam



（a）Elevation at 1270m (b) Elevation at 1335m

Figure 8 Steel tensile stress versus time of reinforcement at different elevation with 5Φ25

Figs.9 (a) and (b) are the time-history curves of steel tensile stress at elevations of 1270m and 1335m, respectively. When reinforcement according to 5Φ28 @ 172 at the dam heel and the upstream and downstream changed slopes, the maximum tensile stress of steel is 265MPa at the elevation of 1270 m at 7.0s, while the maximum tensile stress is 136MPa at the elevation of 1280 m at 10.6s, and the maximum tensile stress is 225MPa at the elevation of 1335 m at 11.8s. Comparing with the scheme of 5Φ25@ 172, the steel maximum tensile stress reduced obviously by the scheme of 5Φ28@ 172, which provides more safety for the dam.



（a）Elevation at 1270m (b) Elevation at 1335m

Figure 9 Steel Tensile stress versus time of reinforcement at different elevation with 5Φ28

Besides, the case for reinforcement with 7Φ28 @172 is also calculated. Though at this moment the steel stress reduced obviously, the damage crack at key locations of the dam is not improved, and the number of steel increases largely. Hence, we suggest that reinforcement according to 5Φ28 @172 at the dam heel and the upstream and downstream changed, and reinforcement according to 5Φ22 @200 on the other place of dam. The final scheme of Institute of Design is single row Φ28@200 at the dam heel, double row Φ28@200 at the upstream and downstream changed slopes.

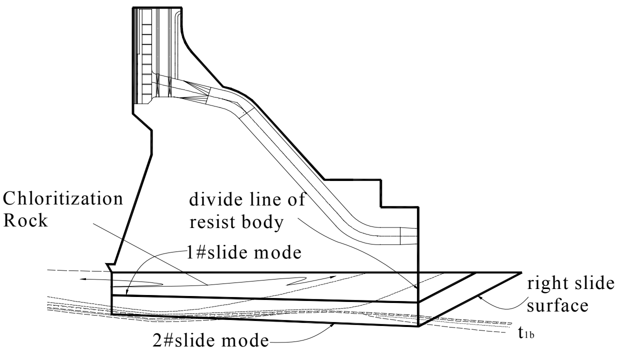
5. SEISMIC SAFETY ANALYSIS OF THE DAM

5.1 Analysis of the whole safety of the dam

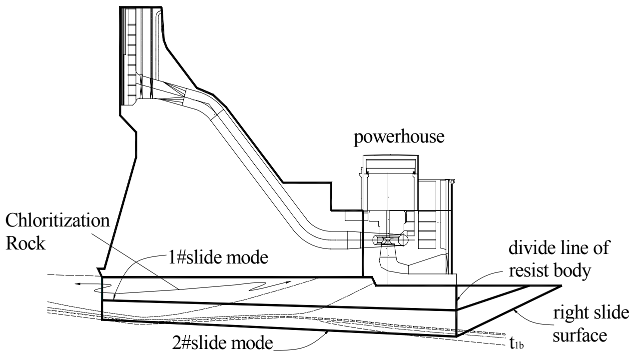
Anti-sliding stability analysis is performed by using rigid body limit equilibrium method based on Chinese Codes. Then, the cases without and with considering powerhouse at the dam toe are analyzed, respectively. Calculation is performed through the combination of dam and plant when powerhouse at the dam toe is considered. The deep sliding modes of the powerhouse dam section are shown in Figs.10 (a), (b).

Sliding mode 1：The near horizontal surface in the crack plane in which chloritization rock mass abundant region under dam foundation is taken as the left sliding plane, and the right sliding plane is sheared from Ⅳa rock mass.

Sliding mode 2：Take t_{1b} weak seam as the left sliding plane, and the right sliding plane is sheared from Ⅳa rock of powerhouse end.



(a) Dam-plant separated structure



(b) Dam-plant jointed structure

Figure 10 Deep sliding modes of the powerhouse dam section

The stability checking formula is given by the following equation (CIWRH 2000):

$$\gamma_0 \varphi S \le \frac{1}{\gamma_d} R \tag{5.1}$$

where γ_0 —structural importance factor, $\gamma_0 = 1.1$, φ —Design situation coefficient， $\varphi = 0.85$, γ_d —structure coefficient of ultimate Limit State, S —structure effect, R —structure resistance.

Taking the rigid body limit equilibrium method, anti-sliding stability along foundation plane and sliding model 1 and 2 is evaluated for cases without and with powerhouse. Structure coefficient is obtained by inverse calculation based on Eqn. 5.1. The results are presented in table 5.1, where γ_d is greater than 0.65, and it meets China Code requirements under earthquake case. Structure coefficient of Ultimate Limit State γ_d under earthquake case is less than that of the normal storage water level and the check flood level. It is the control case of anti-sliding stability of shallow and deep layers in dam foundation. From table 5.1, anti-sliding stability of shallow and deep layers in dam foundation in the case of combination of dam-plant is greater than that in the case of dam-plant separated. That means combination of dam-plant can improve anti-sliding stability of dam.

Table 5.1 Anti-sliding stability results of shallow and deep layers in dam foundation

Sliding mode	dam-plant separated structure			dam-plant jointed structure		
	Effect $\gamma_0\varphi S$ /kN	Resistance $R\gamma_d^{-1}$ /kN	Structure coefficient γ_d	Effect $\gamma_0\varphi S$ /kN	Resistance $R\gamma_d^{-1}$ /kN	Structure coefficient γ_d
Foundation plane	206 961.1	305 665.65	0.96	221 579.2	357 519.05	1.04
Mode 1	242 540.4	250 003.22	0.67	255 747.49	302 962.41	0.77
Mode 2	254 334.0	258 246.85	0.66	268 889.94	310 257.62	0.75

Calculation case: normal case + earthquake

5.2 stability analysis considering local cracking

As a special case without reinforcement, three crack depths are 9m, 3m and 7m at the dam heel, upstream changed slope, and downstream changed slope, respectively. Three layers are shown in Fig.2. Structure coefficient of Ultimate Limit State of three sliding surfaces are calculated as following:

Foundation plane (elevation is 1264 m)：
Effect (normal impound + earthquake) $\gamma_0 \varphi S$ ＝206961.1kN, resistance $R\gamma_d^{-1}$ = 287917.3kN, structure coefficient γ_d by inverse calculation is 0.9.

Horizontal sliding surface at upstream changed slope (elevation is 1345 m)：
Effect (normal impound + earthquake) $\gamma_0 \varphi S$ ＝61427.2kN, resistance $R\gamma_d^{-1}$ = 70184.0kN, structure coefficient γ_d by inverse calculation is 0.74.

Horizontal sliding surface at downstream changed slope (elevation is 1395 m)：
Effect (normal impound + earthquake) $\gamma_0 \varphi S$ ＝139025.0kN, resistance $R\gamma_d^{-1}$ = 14716.0kN, structure coefficient γ_d by inverse calculation is 0.69.

The results show that even without reinforcement, the anti-sliding stability in the above three planes where the damage crack is serious can also satisfy the requirement of the China Code. Structure coefficient γ_d at the downstream changed slope is less than that of upstream slope and dam heel. As long as the depth of crack in the foundation plane does not damage the grout curtain in the dam heel, the dam is safety.

6. CONCLUSION

The dam seismic reinforcement scheme should obey following principles: it is required that the crack does not damage the grout curtain in the dam heel, and cracks at the corners do not have any effect on the stability of the dam. Final amount of reinforcement should be determined by non-linear finite element method. The crack damage depth at the key locations is obviously reduced after reinforcement. Considering the possible sliding modes of the powerhouse dam section and weak surfaces after the nonlinear earthquake response, aseismic stability evaluation of the dam show that the aseismic stability of the powerhouse dam section of Jinanqiao gravity dam can satisfy the requirements of China Code.

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