Seismic analysis of the Lianghekou rockfill dam

Analyse sismique du barrage en enrochement de Lianghekou

Liu Han-long
Key Laboratory of Ministry of Education for Geomechanics and Embankment Engineering, Hohai University, Nanjing 210098, China
Geotechnical Research Institute, Hohai University,Nanjing 210098, China
Fei Kang
Institute of Geotechnical Engineering, YangZhou University,Yangzhou 225000, China

ABSTRACT

The proposd Lianghekou dam is to be located at the Yalong River. It is a rockfill dam with an earth core. With a maximum height of 293 m, the dam creats a reservoir capacity of 101.54×109m^3. At this site, the design peak ground acceleration (PGA) is 288gal. The safety of dam during earthquakes becomes an extremely important design consideration. Three dimensional dynamic finite element analysis was adopted to study the dynamic behaviour of the dam during earthquake. The dam materials were modeled as equivalent-linear material, whose shear modulus??? varys with the mean effective stress. By using the modified Hardin model, the modulus and damping ratio were modeled as strain dependent. The acceleration response was analyzed. The liquefaction resistance of the dam materials and the permanent deformation induced by earthquake were computed by combining the result of cyclic strength and residual strain obtained by the large-scale dynamic triaxial tests.

RÉSUMÉ

Le barrage de Lianghekou proposé sera situé sur la Rivière de Yalong. Il est un barrage en enrochement avec le noyau en terre. Avec une hauteur maximale de 293m, ce barrage crée une capacité du réservoir de 101.54×109 m^3 Sur ce site, l'accélération pointe à la surface de terre est 288gal. La sécurité de barrage pendant séismes devient une considération extrêmement importante en conception. L'analyse tridimensionnelle dynamique par éléments finis était adopte pour étudier le comportement dynamique du barrage pendant séisme. Les matériaux du barrage étaient modélisés comme matériau linéaire équivalent, dont le module de cisaillement varie avec contrainte effective moyenne. En utilisant le modèle de Hardin modifié, le module et le facteur d'amortissement étaient modélisés comme déformation sensible. La réponse d'accélération a été analysée. La résistance à la liquéfaction du barrage et la déformation permanente induite par séisme ont été calculées par combiner le résultat de force cyclique et déformation résiduelle obtenu par les essais triaxiaux dynamiques à grande échelle.

Keywords : high rockfill dam, seismic response analysis, liquefaction, permanent deformation, safety factor

1 INTRODUCTION

To meet the ever increasing demand for electricity, a series of high dams of up to 250-300 m in height are planned to be constructed in Southwest China in the coming three decades (Pan & He, 2000). Many of these dam sites have suitable geology and topography conditions for high dam, but the level of seismicity is high. According to the statistics of the China Earthquake Administration, 82% of the strong earthquakes that occurred in modern times in China were in this region. Because the earth core rockfill dam (ECRD) have better performance against strong earthquake, this type has being increasingly selected for very high dams in China, such as the Lianghekou rockfill dam with a maximum dam height of 293 m and a reservoir capacity of 101.54×109 m^3. At the dam site, the design peak ground acceleration (PGA) is 288gal. Obviously, the safety of dams during earthquakes is extremely important because failure of such a structure may have disastrous consequences on life and property. In order to evaluate its performance against earthquake, three dimensional dynamic finite element analysis in time domain was applied to study the earthquake response of the dam.

2 THE DAM

The Lianghekou earth core rockfill dam is planned to be located at the Yalong River. The site is about 25km upstream of the Yajiang City, Sichuan Province, China. The purpose of the project is mainly for power generation, flood control and navigation. The catchment area is 65599 km2. The crest elevation is 2875.0m and the normal pool level is 2865.0m with a related storage capacity of 10.154 billion m3 and an installed power generating capacity of 3000 MW.

The basic features of the dam are outlined herein with the help of Fig. 1, which shows a cross-section of the Lianghekou dam. The dam is 293.0m high, the crest width is 16.0m and the crest length is 650.0m. The upstream and downstream slopes are 1V:2.0H and 1V:1.9H, respectively. The main body of the dam consists of earth core, filter layer, transition layer and rockfill. Because the earth core material will be obtained from three different material yards, the earth core is divided into three corresponding zones form the base to the dam crest, i.e. A zone (70.0m high) , B zone (25.0m high) and C zone(198.0m high). For the same reason, the rockfill is also divided into three zones, Ⅰ, Ⅱ and Ⅲ.

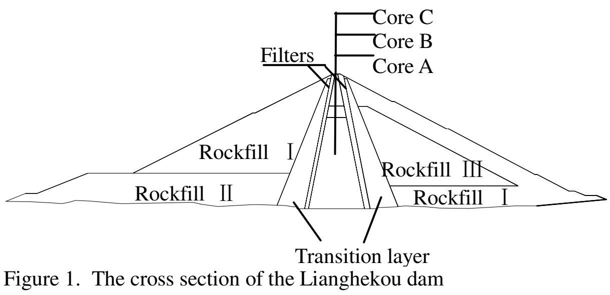

Figure 1. The cross section of the Lianghekou dam

Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering
M. Hamza et al. (Eds.)

doi:10.3233/978-1-60750-031-5-1634

3 THREE DIMENSIONAL DYNAMIC ANALYSIS

3.1 *Finite element discretization and input ground motion*

The dam is discretized with 11470 elements and 11272 nodes. According to the seismic risk analysis, the horizontal peak ground acceleration (PGA) is 288gal, the vertical acceleration component is taken as 2/3 of the horizontal component. The input horizontal base acceleration time history is shown in Fig.2.

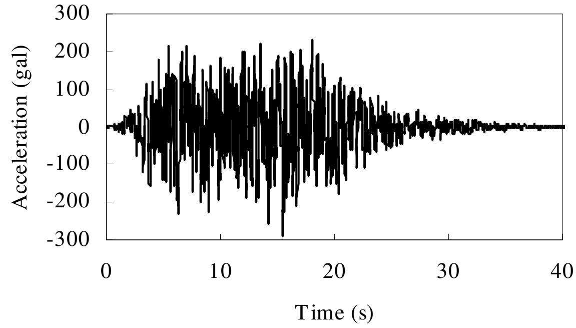

Figure 2. Input horizontal base acceleration time history

3.2 *Material model and parameters*

The dynamic behavior of dam materials are modeled using equivalent linear visco-elastic model, which described through: (1) the initial shear modulus $G_{\max}$ (shear modulus at very low strain level); (2) the decrease of secant modulus G with shear strain γ ; (3) the hysteric damping ratio, λ , which is an increasing function of the amplitude of shear strain γ . For each element, $G_{\max}$ is related to the effective confining stress, $\sigma'_m = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$, by:

$$G_{\max} = k_2 p_a (\sigma'_m / p_a)^n \tag{1}$$

Where k_2 , n are parameters determined from resonant column test, p_a is atmospheric pressure. The spatial distribution of the effective confining stress before earthquake was computed by a static finite element analysis. The decline of secant shear modulus, G , with increasing shear stain are represented by a modified Hardin model (Sheng & Xu, 1996):

$$G/G_{\max} = \frac{1}{1+k_1\bar{\gamma}} \tag{2}$$

Where k_1 is material parameter, $\bar{\gamma}$ is equivalent shear stain defined as:

$$\bar{\gamma} = \frac{0.65\gamma_{\max}}{(\sigma'_m/p_a)^{1-n}} \tag{3}$$

Where $\gamma_{\max}$ is peak shear strain during earthquake. The damping ratio is obtained as:

$$\lambda = \lambda_{\max}\frac{k_1\bar{\gamma}_d}{1+k_1\bar{\gamma}_d} \tag{4}$$

Where $\lambda_{\max}$ is damping ratio at very large shear stain. The parameters used in the analysis are summarized in Table 1. Because there are no test data for core B and core C, the parameters of core A were used in the analysis.

Table 1. Dynamic parameters of dam materials

Material	k_1	k_2	n	$\lambda_{\max}$
Filter layer	11.60	1216	0.321	0.23
Transition layer	15.70	1997	0.328	0.22
Rock fill Ⅰ	18.80	2336	0.268	0.19
Rock fill Ⅱ	17.95	2270	0.273	0.20
Rock fill Ⅲ	17.10	2205	0.279	0.21
Core	21.35	1106	0.556	0.25

After modeling the geometry and dynamic properties of dam, the analysis was performed direct step-by-step by using Wilson's time integration algorithm. In the analysis, the shear modulus and damping ratio were obtained from the 'effective' shear stain level of the previous iteration. The 'effective' shear strain was obtained as 0.65 fraction of the peak shear stain.

3.3 *Results of the dynamic analysis*

3.3.1 *Natural vibration period*

The period of fundamental vibration along the axis of flow, T_1 , was computed as 1.23 sec, which is about 0.42 fraction of the dam height. Fig.3 gives the relationship between the rockfill dam height and the period of fundamental vibration collected from the literature (Okamoto 1984)., the result of the Lianghekou dam is also shown.

It can be seen from the figure that the fundamental period increases with the dam height. Though the data given in the figure are related to the different dam slope gradient and different characteristics of the earth core, it is observed that the fundamental period has a good relationship with the dam height.

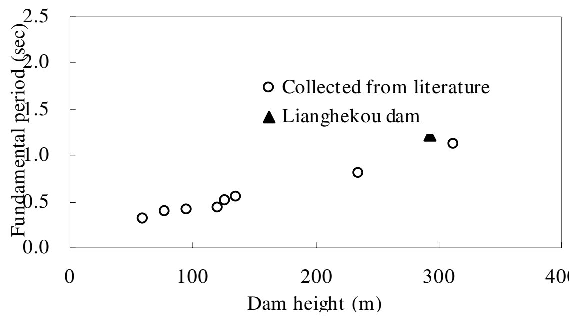

Figure 3. The relationship between the fundamental period and the height of dam

3.3.2 *Peak absolute acceleration*

In order to estimate the stability of the dam by use of the safety factor against sliding, it is essential to know the value and the distribution of acceleration in the dam. Fig.4 shows the computed horizontal peak absolute acceleration's distribution along the height of dam at the centerline. It is observed that the computed accelerations are almost equal to or little smaller than the input PGA value over about 70% of the dam height and exhibit large amplifications in the crest region, the maximum peak absolute acceleration is 596gal.

In the Chinese specifications for seismic design of hydraulic structures, the distribution along dam height is given for different seismic design intensities, while the seismic design intensity is related to the peak ground acceleration (PGA). For examples, for a seismic intensity of 8 degree, the corresponding PGA is 0.2g, while 9 degree intensity has a PGA of 0.4g..

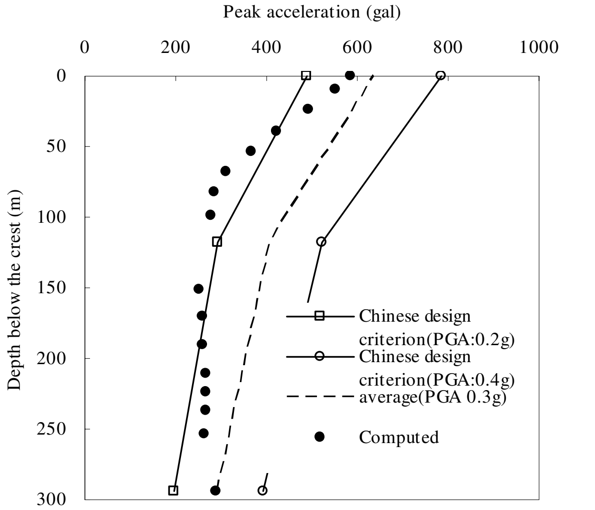

Figure 4. Distribution of the peak absolute acceleration along the dam height

Because the design PGA of Lianghekou dam is 288gal (i.e. 0.293g), predictions using methods given in the Chinese design criterion for seismic intensity of 8 and 9 degree are also included in the same figure for comparisons, their average value may represent the acceleration distribution with a PGA of 0.3g. It is shown that the Chinese design method overpredict the acceleration response when compared to the result of finite element analysis. The difference may be because the distribution given in the Chinese design criterion is based on the measured values of dams lower than 150.0m, it is not suitable for higher dams.

In order to study the spatial distribution of the acceleration, Fig.5 shows the computed horizontal peak absolute acceleration at longitudinal section. As shown in the figure, it is clear that the maximum value of peak absolute acceleration occurs at the top of the biggest cross section. It is also observed that the accelerations at the left abutment and the right abutment are nearly symmetrical, which shows the effect of the topography condition.

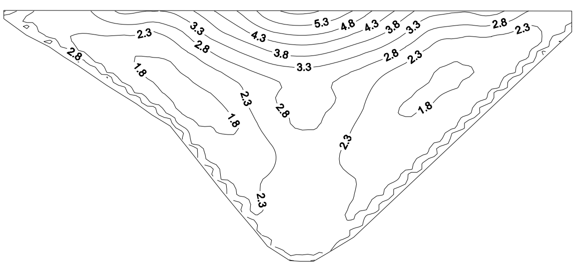

Figure 5. Computed peak absolute acceleration at longitudinal section (m/s^{-2})

3.3.3 *Liquefaction resistance of filter layer*

In the analysis, liquefaction resistance of filter layer was evaluated by pore-pressure ratio r_u, which is defined as the excess pore pressure over initial static effective minor principal stress. If the excess pore pressure ratio of an element equal to one, i.e. the pore pressure reaches the effective confining stress, the soil can be regarded as liquefied.

Some researchers found that the pore pressure ratio is mainly depended on the cyclic ratio, which is the ratio of the number of cycles applied divided by the number of cycles required for liquefaction, i.e. N / N_L where N_L is the number of cycles required for liquefaction at a certain stress level. For high shear stress levels only a few cycles may be required, while for low shear stress levels, a larger number of cycles are required. Fig.6 shows number of cycles required for liquefaction of the filter layer at different effective confining stress and different shear stress ratio, which were obtained from large dynamic triaxial tests. The shear stress ratio is defined as:

$$CSR = \sigma_d / 2\sigma'_{30} \tag{5}$$

Where σ_d is the maximum cyclic axial stress in the dynamic triaxial test; σ'_{30} is the initial effective confining pressure.

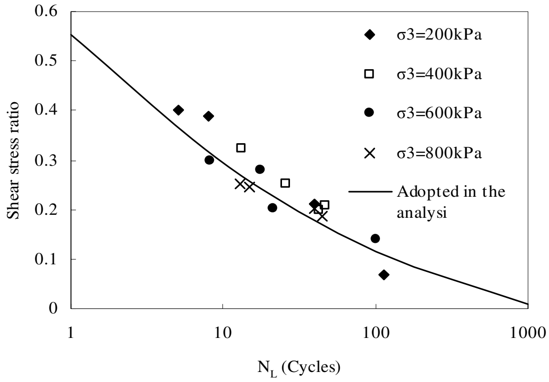

Figure 6. Number of cycles to cause liquefaction of filter layer

As shown in the Fig.6, for the filter material used in the Lianghekou dam, there is no tendency for cyclic strength to decrease with increasing confining stress, so the fit line of all data point was used in the analysis. After get N / N_L from the dynamic analysis, the relationship given in the Fig.7 is used to determine the pore pressure ratio r_u.

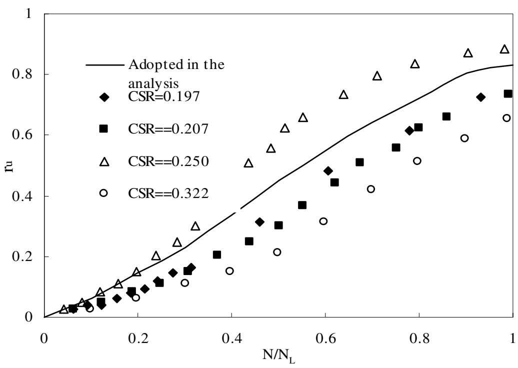

Figure 7. Pore pressure ratio vs. cyclic ratio

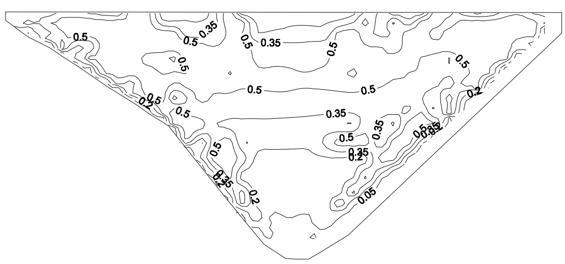

Figure 8. Computed pore pressure ratio of filter layer

The computed r_u of the filter layer is shown in the Fig.8. It is observed that the maximum pressure ratio is 0.63, which occurs at the 2/3rds of the dam height. Because the maximum pressure ratio is less then one, the filter layer is considered to be safe during the earthquake. Since no dissipation of pore pressure was considered in the analysis, the result is conservative.

3.3.4 *Earthquake-Induced Permanent Deformation*

Earthquake induced permanent deformation is a important factor to evaluate the stability of an earth core rockfill dam. In the study, the permanent deformation is computed based on the residual strain potential concept. It is regarded that cyclic loading on an element of soil leads to permanent deformation from two causes: volume change and shear distortion. Many studies have shown that main factors that affect residual strains in cyclic drained triaxial tests are the number of cycles N, the cyclic shear strain amplitude γ_d, and the shear stress level S_l before cyclic loading. The following empirical relationships for residual volumetric ε_{vr} and shear strains γ_r were used in the analysis:

$$\varepsilon_{vr} = c_1(\gamma_d)^{c_2} \exp(-c_3 S_l^2) \log(1+N) \tag{6}$$

$$\gamma_r = c_4(\gamma_d)^{c_5} S_l^2 \log(1+N) \tag{7}$$

Where c_1, c_2, c_3, c_4, c_5 are material constants determined from cyclic drained triaxial test. It is evident that determination of residual strains requires both results of static and dynamic analysis. The residual strain parameters used in the analysis are summarized in Table 2.

Table 2. Residual strain parameters of dam materials

Material	c_1 (%)	c_2	c_3	c_4 (%)	c_5
Filter layer	0.36	0.79	0	8.69	0.76
Transition layer	0.56	0.42	0	8.25	0.40
Rock fill Ⅰ	0.74	0.43	0	9.55	0.38
Rock fill Ⅱ	0.72	0.96	0	9.34	0.37
Rock fill Ⅲ	0.69	0.53	0	9.12	0.35
Core	0.04	0.17	0	18.9	1.02

It is clearly that the residual strains obtained from equation (6) and (7) are realized only when a soil element can deform freely in all directions, or else when no boundary and volume constraints are applied. In order to get the actual permanent strains, the nodal forces necessary to produce the given amount of strain potential must be computed. This is done by a technique similar to the well-know methods of initial stress and initial strain, in which the equivalent node force $\{F\}$ are determined as follows:

$$\{F\} = \iiint_V [B]^T [D] \{\varepsilon_{ij}^p\} dV \tag{8}$$

Where $[B]$ is strain matrix, $[D]$ is elastic matrix which determined based on the current state of static stresses, $\{\varepsilon_{ij}^p\} = \{\varepsilon_x^p, \varepsilon_y^p, \varepsilon_z^p, \gamma_{xy}^p, \gamma_{yz}^p, \gamma_{zx}^p\}^T$ is residual strain vector. Apply $\{F\}$ to the nodes of the finite element grid, and combine the existing boundary conditions, the earthquake-induced permanent deformation can be solved. It also should be noted that the residual strains in equation (6) and (7) are composed of volumetric and shear strain, and these must be rotated to the local finite element coordinates system. This is done by considering the principal strains to be oriented in the same directions as the principal stress before cyclic loading.

Permanent settlement contour at the cross section which calculated based on the above-mentioned deformation analysis method is given in Fig.9. It is observed that the earthquake induced permanent settlement of the Lianghekou dam is about 1.22m under the design earthquake with peak acceleration of 288gal, which is 0.42% of the maximum height of dam. To prevent the overtopping after earthquake, the permanent settlement should be taken into account when design the crest elevation. Fig.10 compares the dam section before and after the earthquake, it is shown that the dam after earthquake is "smaller", which indicates that permanent settlement is larger than the horizontal deformation. This is because that the rockfill material contract considerably under cyclic loading, especially true for high confining pressure. Therefore, both volume change and shear distortion induced by earthquake loading should be considered in the analysis, otherwise the result is unreliable.

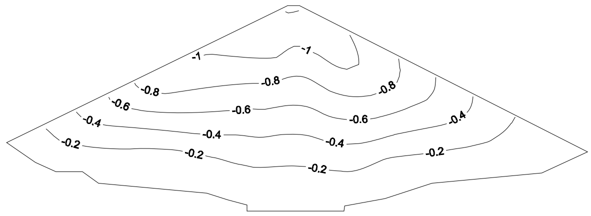

Figure 9. Computed permanent settlement at the cross section (m)

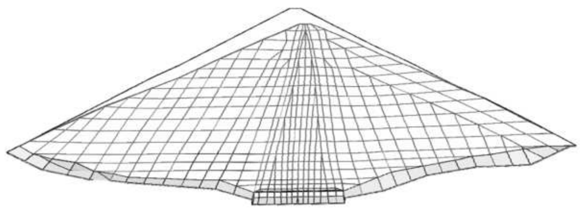

Figure 10. The dam section before and after the earthquake

4 CONCLUSIONS

A three dimensional equivalent linear dynamic finite element analysis was carried out to study the earthquake response of the Lianghekou dam. Based on the results, the following conclusions can be drawn:

(1) The period of fundamental vibration along the axis of flow was computed as 1.23 sec, which is about 0.42 fraction of the dam height. With the help of data collected from literature, the fundamental period is found to have a good relationship with the dam height.

(2) The computed accelerations are almost equal to or little smaller than the input PGA value over about 70% of the dam height and exhibit large amplifications in the crest region, the maximum peak absolute acceleration is 0.608g.

(3) Because the maximum pressure ratio in the filter layer is 0.63, the filter layer will not liquefy during the earthquake.

(4) Both residual volumetric and shear strains induced by the earthquake loading should be considered. The permanent settlement of the Lianghekou dam induced by earthquake is about 122cm, which is 0.42% of the maximum height of dam. To prevent the overtopping after earthquake, the permanent settlement should be taken into account in design.

ACKNOWLEDGMENTS

The support of the National Science Foundation of China (No. 50639050 and 50825901) is acknowledged

REFERENCES

Liu Hanlong. 1996. Permanent deformation of foundation and embankment dam due to stochastic seismic excitation. *Chinese Journal of Geotechnical Engineering*, Vol.18, No.3:19-27. (in Chinese)

Oakamoto, S. 1984. *Introduction to Earthquake Engineering*, second edition, University of Tokyo Press, Japan.

Pan Jiazheng & He Jing. 2000. *Large Dams in China; A Fifty-year Review*, China waterpower Press, Beiing, China.

Sheng Zhujiang & Xu Gang. 1996. Deformation behavior of rock materials under cyclic loading. *Journal of Nanjing Hydraulic. Research Institute*. No.2:143-149. (in Chinese)