Seismic Analyses of Concrete Gravity Dam with 3D Full Dam Model

Haibo Wang, Deyu Li & Huichen Yang
China Institute of Water Resources and Hydropower Research, Beijing, China

SUMMARY:
Seismic analyses of concrete gravity dames are usually idealized as two dimensional problems. But for gravity dams built in narrow valleys or the sites of very high seismicity, the interaction between adjacent blocks may influent the seismic responses of dam significantly. In the paper, three dimensional FEM full dam model are adopted to analyze the seismic responses of a concrete gravity dam, in which, dynamic interaction between dam-foundation, dam-water, energy radiation to far field, dynamic contact between monoliths, etc are taken into consideration. Comparison are made between the numerical results of massless foundation full dam model, full dam model with or without contraction joints, the influences of these factors are discussed and proposals are given to improve the aseismic capacity of concrete gravity dams.

Keywords: concrete gravity dam, seismic responses, full dam model, dynamic contact, contraction joints

1. INTRODUCTION

Most often, seismic responses of a concrete gravity dam is evaluated based on a two-dimensional analysis with a single monolith or several independent monoliths, which are supposed to be the most critical to earthquake ground motions, because the gravity dam is long in the axial direction and consists of many monoliths separated by contraction joints. In present design procedures (US Army Corps of Engineers 2007, Ministry of Power Industry 2000, China), the dynamic dam-reservoir interaction is simplified with added masses and the flexibility of the foundation is approximated with a massless foundation. But for gravity dams built in narrow valleys or on sites within seismically active regions, dynamic interaction between dam monoliths may influence the seismic responses of the dam significantly. In that case, a nonlinear dynamic analysis with a three-dimensional full dam numerical model is indispensable, because it is very difficult, if not impossible, to set realistic boundary conditions at the sides if only a portion of monoliths are involved in a dynamic analysis. As a nonlinear dynamic analysis with a full dam model of a concrete gravity dam requires much more computational efforts than a two-dimensional model, research works in this respect are quite limited. El-Nady (1992) did simplified computations of a concrete gravity dam with keyed contraction joints, solely concentrated on the shear forces between the joints. Scheulen (2010) and Knarr (2011) investigated Big Creek Dam No. 7 with 3D dynamic analysis, in which contact surfaces were used and parameters were altered to represent fully closed or fully open joints.

In this study, dynamic analyses with 3D full dam models were performed to investigate the seismic responses of a concrete gravity dam subjected to severe earthquake ground motions. The concrete gravity dam is 440m along the crest and consists of 15 monoliths. The maximum height is 103m with a downstream slope of 0.8. The peak ground acceleration (PGA) is 0.57g for a 2% probability of exceedance in 100 years, according to the probabilistic seismic hazard analysis of the dam site. The design response spectra used is the standard one for rock specified in the reference (Ministry of Power Industry 2000), in which, the spectra is 2.0 in amplification between periods of 0.1 to 0.2 seconds. Artificial waves for three components of ground motion were generated by fitting the design response spectra separately. Ground motions were applied on the dam system in two horizontal plus vertical directions simultaneously, where the level in the vertical direction was 2/3 of the PGA in horizontal direction.

2. NUMERICAL MODEL AND COMPUTATION METHOD

To address the canyon effects and interaction between blocks, a three-dimensional finite element model of dam-foundation system was developed using commercial computer program MSC.Marc and the pre-processor software MSC.Patran. The model consists of 46999 elements and 56581 nodes totally, among them 11363 elements and 17443 nodes are on the dam, as shown in Fig. 2.1. The foundation extends to a distance of two times of maximum dam height below, upstream and downstream, and one time on the left and right of the dam. Damping boundaries were set at the bottom and sides of the foundation to account for the radiation damping present in the semi-infinite foundation (Lysmer 1975, Zhang 2008). This provided a mechanism for energy absorption and wave propagation inside the foundation model. The damping boundary is easy to be applied in FEM dynamic analysis with step by step direct integration, although it does not satisfy strictly the boundary conditions for waves travelling in a non-perpendicular direction to the boundary. As an accompaniment, input seismic motion varied in depth on the sides of the foundation according to free field responses. Hydrodynamic effects of the impounded water were simply represented by Westergaard's added masses (Westergaard 1933), therefore the compressibility of water was neglected.

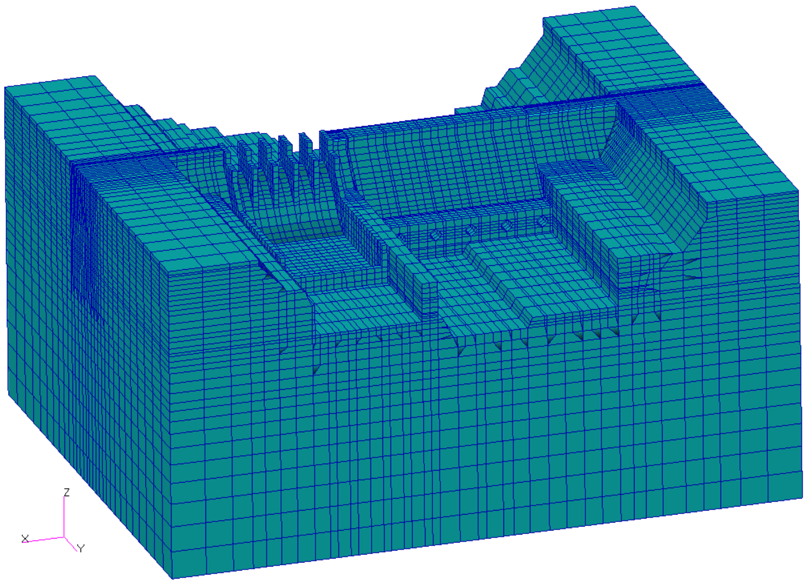

Figure 2.1. Finite element mesh of dam-foundation system

Static modulus of elasticity of the dam concrete and static deformation modulus of elasticity of the foundation rock are 22GPa and 10GPa, respectively, for analyses. The dynamic modulus is assumed to be equal to 1.3 times of the static modulus. The density of the dam concrete and the foundation rock are 2400kg/m^3 and 2600kg/m^3, respectively. And Poisson's ratio of 0.167 and 0.25 were used in the analyses for the concrete and rock, respectively.

The nonlinear responses of the dam-foundation system with contract joints were computed by the step-by-step direct integration procedure. Rayleigh damping was used in the direct integration procedure, a 15Hz higher frequency was used to approximate the 5% critical damping for the dam. The contraction joints can be expected to open and close repeatedly as the dam vibrates in response to severe earthquake ground motions, this contact phenomenon was treated by direct detection of potentially contactable regions at every time step, without special contact or gap element. The static friction coefficient is 1.05 times of the dynamic one which is 0.7, and tensile strength is zero across the joints. Compared with procedures with special elements between contraction joints, the above procedure is applicable to all contact problems, even with very large sliding or distortion between contact bodies, more computational efforts, nevertheless, are required.

In another analysis with the same numerical model, the contraction joints between monoliths were fully closed, named as Complete Dam Model. Furthermore, the density of the foundation rock was set to zero and the sides and bottom of the foundation were fixed to present a massless foundation, commonly used in dam design procedure, named as Massless Foundation Model. The results of both models were employed for the assessment of canyon effects and interaction between contraction joints on the seismic responses of the dam through comparison.

Static loads include the gravity of the dam, the hydrostatic pressure at full reservoir level, the static pressure of the sediment on the upstream face and the hydrostatic tail water pressure on the downstream face.

3. ANALYSIS RESULTS

3.1. Massless Foundation Model

As concrete gravity dams are built generally on competent rock foundations, massless foundations are commonly used to represent the effects of rock in present design procedures. Dynamic analysis with a massless foundation is a direct extended application of the static analysis model, in which the static flexibility of the rock is taken into consideration and an infinite wave speed is implied for the rock. As fixed boundary is used with the massless foundation model, to which input ground motions are applied directly, all dam monoliths experience the same seismic excitation and the energy of structural vibration is trapped inside the region of the computational model and dissipated only by material damping. The results are acceptable in the design if the dynamic stresses due to earthquakes are not too high compared with static ones, because the dynamic responses of the dam with a massless foundation model are usually exaggerated.

In the results of the 3D full dam Massless Foundation Model, as expected, the seismic responses of the dam are closely related to the height of monolith, refer to Table 3.1, in which dam blocks are numbered from left to right in the upstream-to-downstream view. Maximum acceleration responses of the dam are 18.09m/s^2 and 15.5m/s^2 in the stream and cross river directions on the crest of the tallest monolith, respectively. Acceleration responses of the piers on the spillway sections in the cross river direction are extraordinarily high, the maximum one reaches 116m/s^2, because they were modelled as freestanding cantilevers on the spillway sills. Actual responses should be smaller since there are traffic beams on the top of the piers and cracking would happen at the end near spillway sill. Maximum acceleration in vertical is 21.1m/s^2 on the first spillway from the left, and that is 11.7 m/s^2 on the tallest monolith.

Fig.3.1 shows the distribution of maximum principal stresses and Fig.3.2 shows the distribution of minimum principal stresses on the dam. For the monoliths of the same type, the stresses increase with the dam height. The maximum tensile stress at the dam heel of the highest monolith reaches 18.4MPa. The maximum compressive stress at the dam toe of the highest monolith reaches 22.3MPa. The maximum tensile stresses on both upstream and downstream faces exceed 10MPa in a large area. The high stresses on the monoliths at both banks are due to the restriction from the side rocks. Besides, cross river ground motions created very high stresses on the piers. The high stresses on the downstream faces of the monoliths with buried penstock at the left and right ends can be attributed to the cross river ground motions as well, where the neighbouring monoliths have different cross sections.

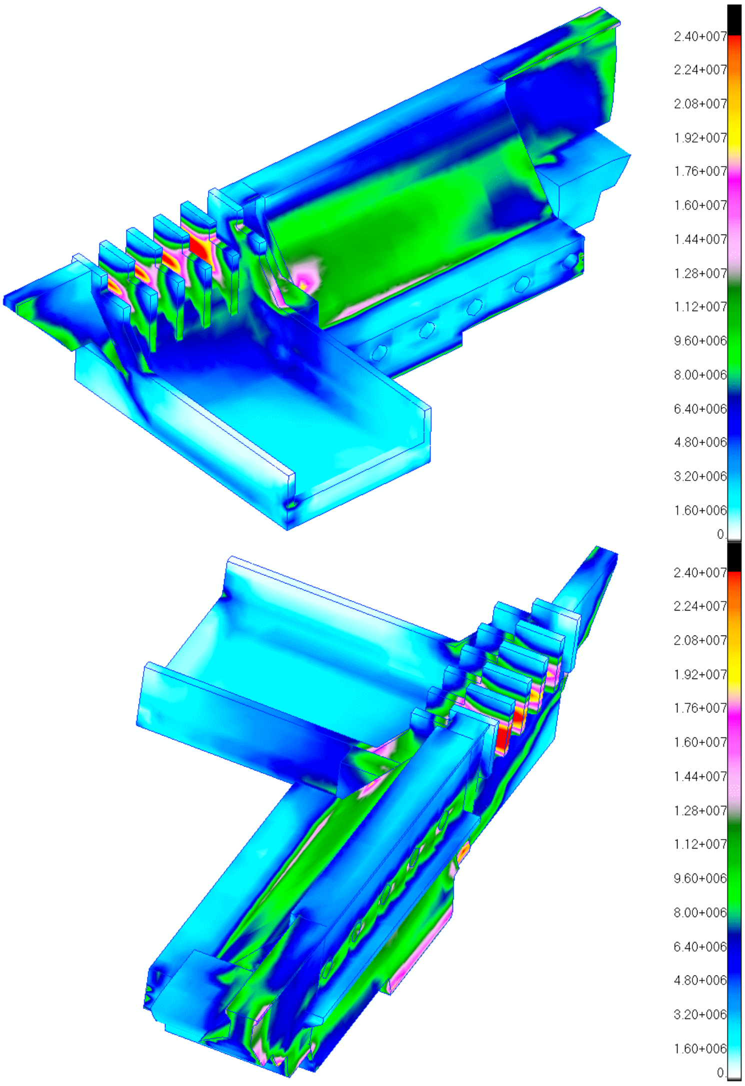

Figure 3.1. Maximum principal stresses by Massless Foundation Model (Pa)

Table 3.1. Comparison for maximum crest acceleration responses between Massless Foundation Model and Complete Dam Model (unit: m/s^2)

No. of Blocks L to R	Stream Direction			Cross River Direction			Vertical Direction		
	Massless Foundation	Complete Dam Model	Ratio	Massless Foundation	Complete Dam Model	Ratio	Massless Foundation	Complete Dam Model	Ratio
1	10.0	7.0	0.70	8.9	5.5	0.62	4.5	4.1	0.91
2	15.9	14.3	0.90	10.5	5.0	0.48	6.7	5.9	0.88
3	15.7	16.0	1.02	12.7	5.0	0.39	8.1	6.0	0.74
4	14.7	13.0	0.88	14.8	4.8	0.32	9.4	5.0	0.53
5	16.5	10.4	0.63	14.6	5.4	0.37	9.8	4.9	0.50
6	18.1	10.4	0.57	14.9	5.7	0.38	10.8	4.6	0.43
7	17.4	11.2	0.64	15.2	5.8	0.38	11.7	4.5	0.38
8	16.4	11.3	0.69	15.5	6.9	0.45	11.5	4.9	0.43
9	14.7	15.1	1.03	18.1	8.2	0.45	13.0	6.35	0.49
10	12.8	18.3	1.43	116.1	29.8	0.26	21.1	8.2	0.39
11	15.3	16.2	1.06	100.6	28.2	0.28	17.3	7.8	0.45
12	14.8	20.8	1.41	85.4	28.0	0.33	13.9	6.8	0.49
13	11.0	17.4	1.58	70.5	27.8	0.39	10.8	6.0	0.56
14	10.0	13.8	1.38	10.4	9.5	0.91	5.3	4.6	0.87
15	4.3	11.1	2.58	6.7	5.3	0.79	3.2	4.2	1.31

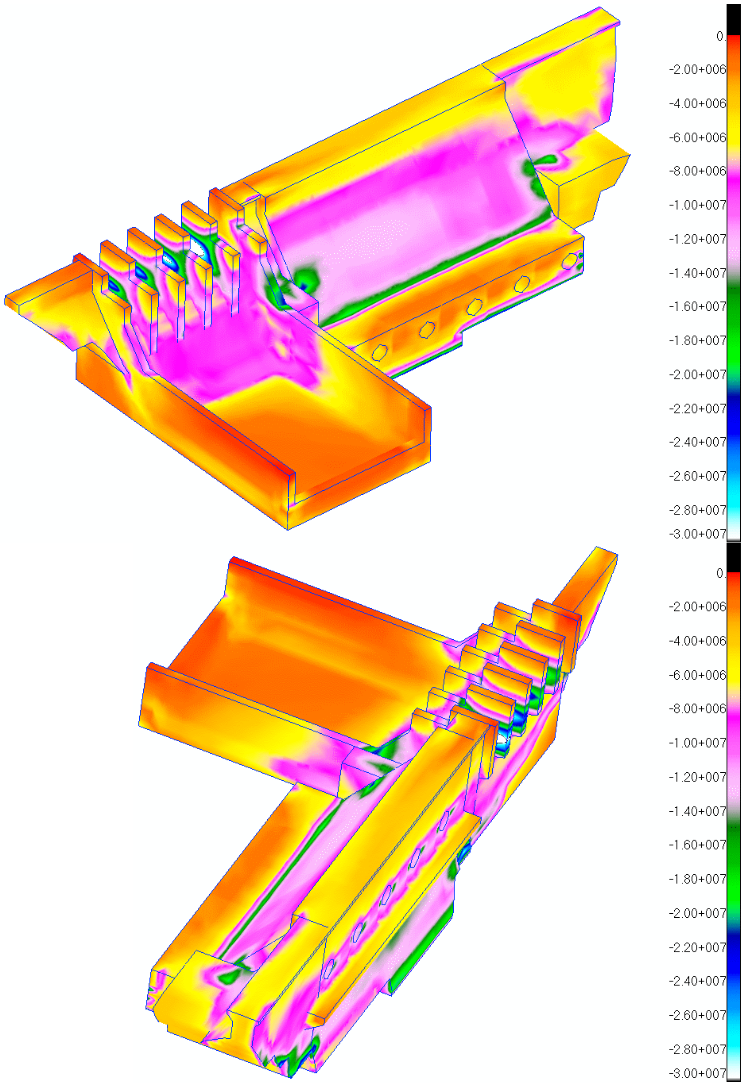

Figure 3.2. Minimum principal stresses by Massless Foundation Model (Pa)

3.2. Complete Dam Model

In Complete Dam Model, the dam-foundation dynamic interaction as well as the wave propagation in the foundation was properly simulated, in consequence, the canyon effects and radiation damping effects were represented appropriately. These dynamic interaction effects can be well understood through the comparison of the responses computed with Complete Dam Model and Massless Foundation Model.

As summarized in Table 3.1, acceleration responses show significant reduction due to dynamic interaction effects in general, except for those in the stream direction on the monoliths near the right bank, where steeper slope topography produces stronger ground motion as a result of the wave reflection and diffraction in the canyon. The maximum acceleration in the stream direction is 20.8m/s^2 at the top of the pier on the centre spillway, 1.41 times of the result with Massless Foundation Model at the same place. The biggest increment in acceleration in the stream direction is 2.58 times at the top of the first monolith on the right bank, and that in the vertical direction is 1.31 times at the same position. For those monoliths on flatter river bed, the acceleration responses decrease by about 35%, 60% and 55%, in the stream, cross river and vertical directions, respectively. The largest reduction is more than 70% at the spillways in the cross river direction, but the maximum value in that direction is still on the pier of the spillway.

Fig. 3.3 is the distribution of maximum principal stresses on the dam. Fig. 3.4 is the distribution of minimum ones. Comparing with the results of Massless Foundation Model, the maximum stresses decrease for more than 60% for all monoliths except for those at both banks where the maximum stresses decrease for about 30%. The maximum tensile stress at the dam heel of the highest monolith was 3.2MPa. The maximum compressive stress at the dam toe of the highest monolith was 12.3MPa. The tensile stresses on both upstream and downstream faces of the dam body were less than 3.0MPa for most monoliths, except for those at both banks and those with a neighbour of different section. Overall, the distributions of the principal stresses on the dam show many differences comparing to the results of Massless Foundation Model. The maximum principal stresses at the end of piers decrease significantly too, by about 80%. The increase of the stresses on the side wall of the spillway near the end of cushion pool could be due to local topography condition, referring to Fig. 2.1, but it is not critical to the safety of the dam.

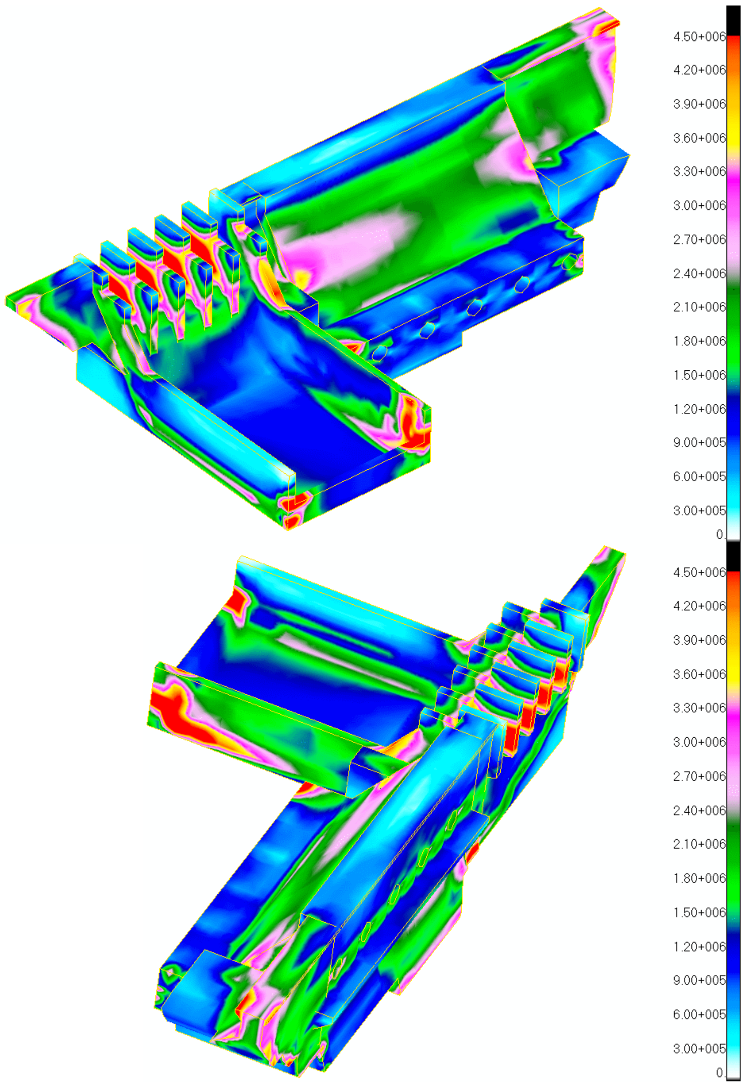

Figure 3.3. Maximum principal stresses by Complete Dam Model (Pa)

3.3. Model with Joints

The contraction joints between monoliths of concrete gravity dams are not grouted usually. Those grouted can only transfer very limited tensile stresses cross the joints, therefore initial cracks in contraction joints may exist before experiencing strong earthquakes due to thermal expansion or contraction of the concrete. The project investigated in present study is to be grouted, therefore, zero tensile strength of the contraction joints without initial gap was assumed in the analyses.

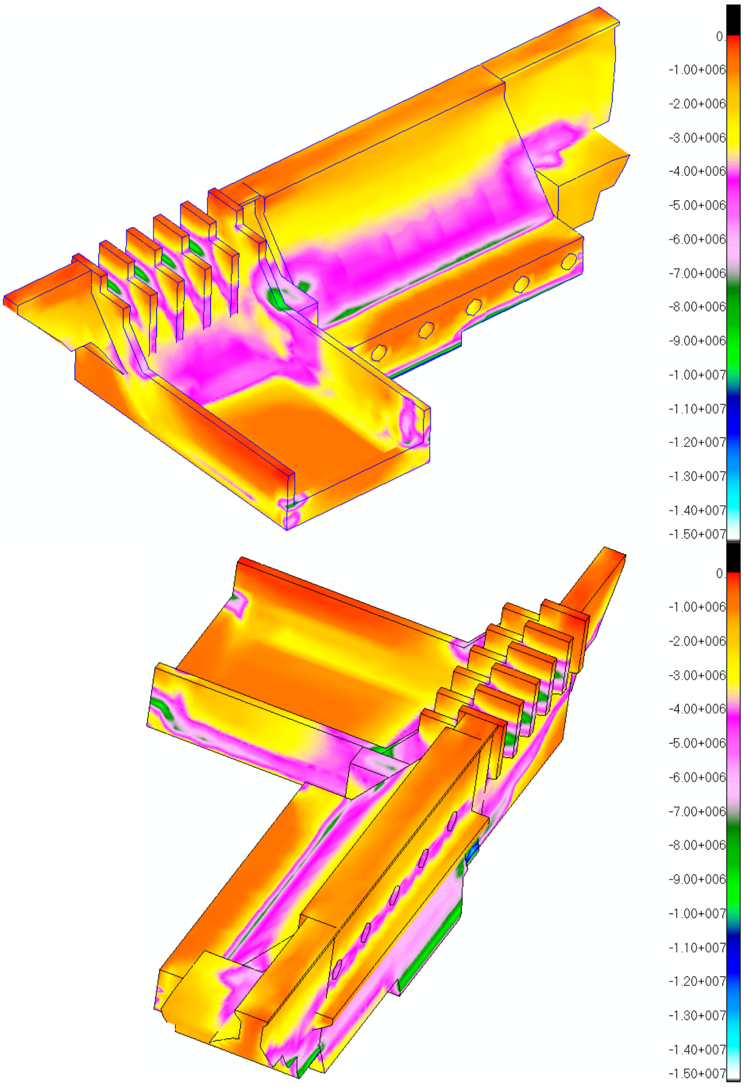

Figure 3.4. Minimum principal stresses by Complete Dam Model (Pa)

Table 3.2. Comparison for maximum crest acceleration responses between Complete Dam Model and Model with Joints (unit: m/s^2)

No. of Blocks L to R	Stream Direction			Cross River Direction			Vertical Direction		
	Complete Dam Model	With Joints	Ratio	Complete Dam Model	With Joints	Ratio	Complete Dam Model	With Joints	Ratio
1	7.0	7.8	1.120	5.5	8.10	1.473	4.1	4.36	1.063
2	14.3	15.0	1.049	5.0	16.0	3.200	5.9	8.19	1.388
3	16.0	14.7	0.919	5.0	13.6	2.720	6.0	9.27	1.545
4	13.0	11.9	0.912	4.8	15.6	3.250	5.0	10.3	2.060
5	10.4	12.7	1.221	5.4	31.1	5.759	4.9	9.58	1.955
6	10.4	12.1	1.160	5.7	25.3	4.439	4.6	8.28	1.800
7	11.2	11.0	0.979	5.8	24.4	4.207	4.5	7.97	1.771
8	11.3	13.1	1.161	6.9	15.1	2.188	4.9	9.45	1.929
9	15.1	14.4	0.951	8.2	14.7	1.793	6.35	10.9	1.717
10	18.3	23.7	1.294	29.8	57.0	1.913	8.2	15.4	1.878
11	16.2	19.1	1.180	28.2	60.0	2.128	7.8	13.2	1.692
12	20.8	19.2	0.925	28.0	58.4	2.086	6.8	12.4	1.824
13	17.4	17.0	0.976	27.8	48.2	1.734	6.0	17.4	2.900
14	13.8	13.4	0.973	9.5	23.9	2.516	4.6	10.3	2.239
15	11.1	11.0	0.987	5.3	9.21	1.738	4.2	4.56	1.086

In Table 3.2, maximum response accelerations on the crest with Complete Dam Model and Model with Joints are summarized. Significant increment of the maximum accelerations in the cross river direction is due to the impact between joints when closing. The taller the monolith, the stronger the impact. However, the largest opening and relative sliding, about 3cm and 4cm, respectively, are at the joint between monoliths with more structural differences. The impacts influence the vertical accelerations as well, but not as strong as in the cross river direction. Acceleration responses in the stream direction are affected little by the impacts, and the biggest increments are 22% and 29% on the monoliths at the river bed and spillways, respectively. On more than half of monoliths, especially those near the right side, the accelerations in the stream direction decrease for several percents, which could be attributed to the energy dissipation by friction.

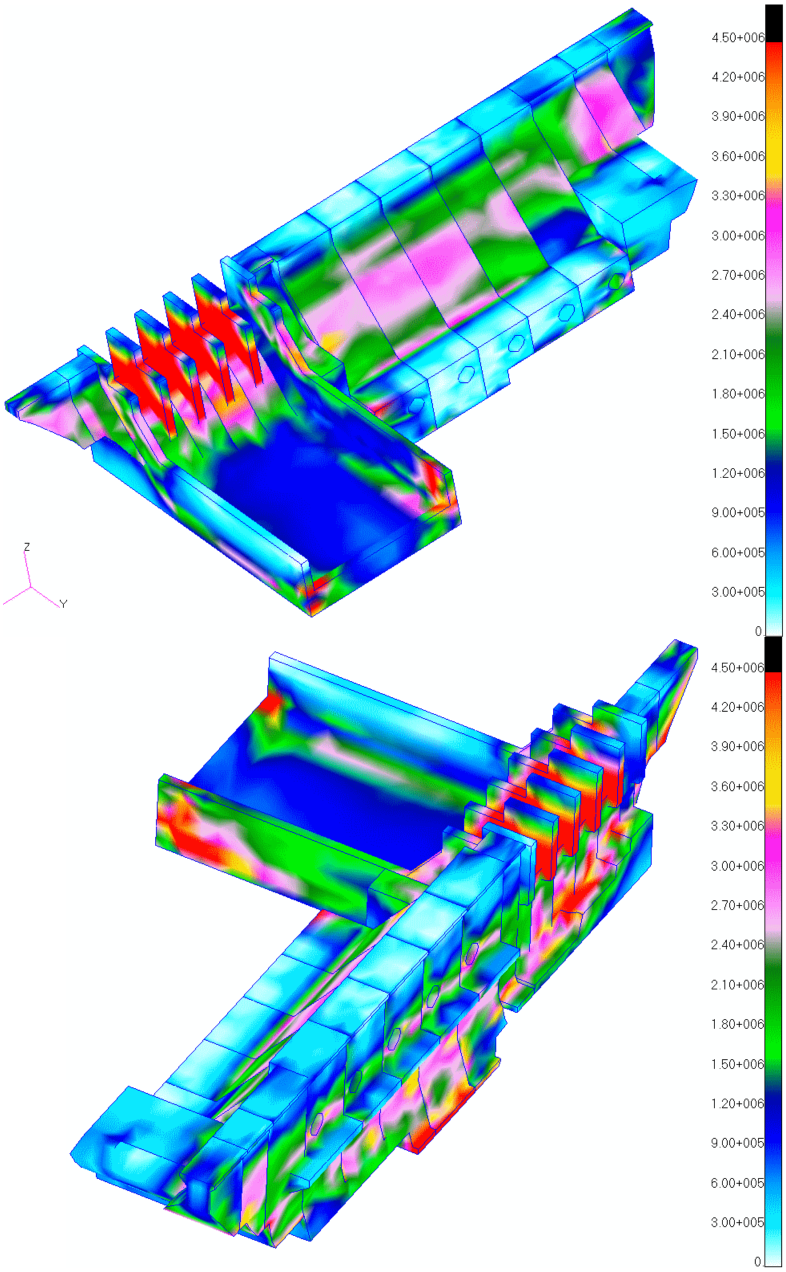

Figure 3.5. Maximum principal stresses by Model with Joints (Pa)

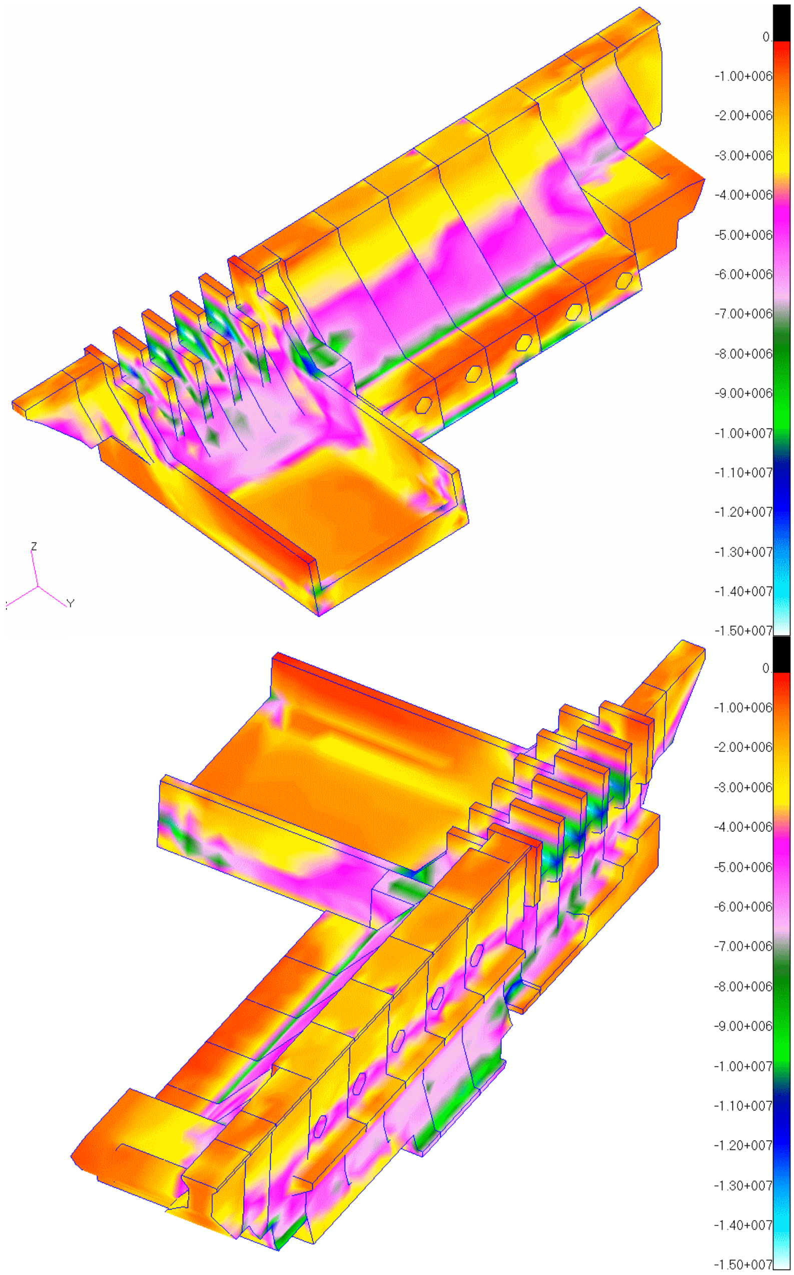

Figure 3.6. Minimum principal stresses by Model With Joints (Pa)

The strong impact of the contraction joints during the earthquake may cause local crushing of the concrete near the contact surfaces, and the energy loss during impact can reduce the responses of the dam, therefore actual acceleration responses must be smaller than above results. Using soft plastic grouting material in contraction joints, however, can decrease the impact forces considerably and prevent local crushing of the concrete.

Fig. 3.5 shows the distribution of the maximum principal stresses. Fig. 3.6 shows the distribution of the minimum principal stresses. Since there is no tensile stress across the contraction joints, the discontinuity of the maximum stresses between the monoliths is evident. Comparing with the results of Complete Dam Model, the stresses on the dam faces increase for most monoliths on the river bed, especially for the spillway sections, but decrease for the monoliths at both banks. Although the increment is over 50% for some region, the maximum tensile stresses on the dam faces are less than 3.5MPa for most monoliths. The maximum tensile stress of 5MPa occurs at the dam heel of the highest monolith. The maximum compressive stress of the dam is about 9MPa. It should be noted that there always exist fissures in the realistic foundation rock although it is commonly assumed as continuous elastic medium in the computation. This will release considerably the tensile stresses at the dam heel. Therefore, 5MPa nominal tensile stress at the dam heel within very limited region will not cause a serious problem from an engineering judgment. The judgment is supported by very scarce cases of damaged gravity concrete dams such as Koyna of India, Xinfengjiang of China and Sefid-rud (Ghaemmaghami, 2010) of Iran during earthquakes.

The assumption of fully closed contraction joints during earthquakes in numerical analyses represents an extreme condition of interconnection between the monoliths. The differences between the results of Complete Dam model and Model with Joints imply that increasing the connection between the monoliths can diminish the seismic responses and enhance the aseismic capacities of the dam.

As the high stresses of the piers on the spillway sections due to the earthquakes, the reinforcement must be carefully designed to provide sufficient capacity of deformation. And this should be checked with the dynamic analyses considering cracking of the concrete and yielding of the steel bars.

4. CONCLUSIONS

Three-dimensional full dam models, with different densities of the foundation rock and different conditions of the contraction joints, were used to assess the seismic responses of a concrete gravity dam. The comparison between the results of different models indicate that the dynamic interaction of dam-foundation reduces significantly the responses of the monoliths on the river bed but increases the responses of the monoliths on the steep slope at both banks, furthermore, the opening and sliding of the contraction joints during the earthquake increase dam responses in general too, very large accelerations are created by the impacts of joint closing in the cross river direction. The cross river ground motion produces very high stresses on the piers of spillway sections. The analyses using three-dimensional full dam model with contraction joints present a more realistic state of the anticipated behaviour of a large concrete gravity dam under strong earthquakes. At the dam sites of seismically active regions, increasing the connection between the monoliths can reduce the seismic responses and enhance the aseismic capacities of the dam. And using soft plastic grouting material in contraction joints will decrease the impact forces considerably and prevent local crushing of the concrete. The reinforcement of the piers must be carefully designed to provide sufficient capacity of deformation and should be checked with the dynamic analyses considering cracking of the concrete and yielding of the steel bars.

REFERENCES

El-Nady, Ahmed Mohamed. (1992). Seismic Analysis of Concrete Gravity Dams With Keyed Contraction Joints, *Open Access Dissertations and Theses. Paper 3502, McMaster University*

Ghaemmaghami A. R.,&Ghaemian M. (2010). Shaking table test on small-scale retrofitted model of Sefid-rud concrete buttress dam, *Earthquake Engng. Struct. Dyn.* **39:1**,109–118. DOI: 10.1002/eqe.928

Lysmer, J., etc. (1975). FLUSH - a program for approximate 3-D analysis of soil-structure interaction problems, *Report No. EERC 75-30, Earthquake Engineering Research Center, University of California, Berkeley*

Mike Knarr, etc. (2011). The Investigation of a Concrete Gravity Dam in a Narrow Canyon Using 3-D Nonlinear Analysis, *21st Century Dam Design—Advances and Adaptations, 31st Annual USSD Conference*, San Diego, California, April 11-15

Ministry of Power Industry. (2000). Specifications for Seismic Design of Hydraulic Structures (DL 5073-2000), Beijing

Scheulen, F., von Gersdorff, N., Duron, Z., Knarr, M. (2010). Numerical model validation for large concrete gravity dams, *30th Annual USSD Conference*

US Army Corps of Engineers. (2007). Earthquake Design and Evaluation of Concrete Hydraulic Structures, EM 1110-2-6053, 1 May

Westergaard, H. M. (1933). Water Pressures on Dams during Earthquakes, Transaction, *American Society of Civil Engineers,* **98**:418-433

ZHANG Shunfu, WANG Haibo. (2008). Equivalent viscous boundary elements and the method of wave input for viscous boundary. *Journal of Hydraulic Engineering*，**39:10**,1248-1255, (in Chinese)