13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 412

DEVELOPMENT OF 3-D DYNAMIC ANALYSIS METHOD FOR COUPLED DAM-JOINTS-FOUNDATION-RESERVOIR SYSTEM

Yoshiaki ARIGA[1] , Zengyan CAO[2] and Hiroyuki WATANABE[3]

SUMMARY

The various influential factors, such as the dynamic interaction between dam and foundation and between dam and reservoir, the energy dissipation and inflow between foundation and free field, the non-linear effects of dam materials and the discontinuous effects of joints should be considered in order to improve the accuracy and reliability of earthquake safety evaluation of dams. These problems are treated in this study.

In regard to concrete dams, the contraction joints and peripheral joints are generally built in the dam for preventing the cracks due to temperature variation or contraction, etc. So, it is considered that the discontinuous behaviors of joints have significant effects upon the dynamic responses of dam against very strong earthquake motion.

Taking these problems into account, a 3-D nonlinear dynamic analysis method for the coupled dam-joints-foundation-reservoir system has been developed in this study. A 3-D interface element is applied for modeling the joints. The discontinuous behavior such as opening, closing and sliding of joints can be simulated with this method. Concerning the modeling of reservoir, the wave equation is dispersed by the finite difference method. The following considerations were drawn from this study.

The existing joints, such as the contraction joints and peripheral joints, will behave discontinuously against very strong earthquake motions, and exert significant effects on the responses of dam. The discontinuous behaviors of joints decrease the dynamic stresses and dynamic strains remarkably. Consequently, the existing joints have favorable effects on the earthquake performance of concrete dam. In the seismic stability analysis of concrete dam, a coupled dam-joints-foundation-reservoir model should be used. The reservoir water has beneficial effects on the safety of dam against very strong earthquake motions.

INTRODUCTION

Usually, the contraction joints and peripheral joints are built in concrete dams for preventing the cracks due to temperature variation and contraction, etc. Since early 1970's, many researches have been carried out the studies on the behaviors of such joints and their effects on the earthquake stability of dams. In the field of numerical analysis, the analytic method of joint elements has being developed, Clough[1], Watanabe[2] Fenves[3],, with which the behaviors of contraction joints during earthquakes can be

[1] Principal researcher, Electric Power Development Co., Chigasaki, Japan: yoshiaki_ariga@jpower.co.jp
[2] Kaihatsu Computing Service Center Ltd., Tokyo, Japan
[3] Professor, Saitama University, Saitama, Japan

simulated in a general way. However, on the modeling method and the behaviors of such joints in the condition of full reservoirs etc., there are still some problems which should be made clear and solved. Taking these problems into considerations, a 3-D nonlinear dynamic analysis method (program "UNIVERSE") for the coupled dam–joints–reservoir–foundation system has been developed, Ariga[4]. An interface element is used in the method for modeling the vertical joints between cantilevers and the peripheral joint between dam and foundation. Cyclic opening and closing and intermittent sliding of the joints can be simulated with this method. In this paper, the principle of the method and the case study for the existing arch dam, namely the Shintoyone Dam, are described.

3-D DYNAMIC ANALYSIS METHOD DEVELOPED

The effects of contraction and peripheral joints, the interaction between dam and reservoir and that between dam and foundation, the energy dissipation and inflow between foundation and semi-infinite free field, etc., are treated in the following way in this study.

Coupled Dam – Reservoir –Foundation System

The coupled dam – reservoir – foundation system is expressed in the following equations, taking the former researches, Chpora[6], Shiojiri[7].

$$\begin{cases} \begin{bmatrix} M_d & M_{df} \\ M_{fd} & M_f \end{bmatrix} \begin{Bmatrix} \ddot{u}_d \\ \ddot{u}_f \end{Bmatrix} + \begin{bmatrix} C_d & C_{df} \\ C_{fd} & C_f^* \end{bmatrix} \begin{Bmatrix} \dot{u}_d \\ \dot{u}_f \end{Bmatrix} + \\ \qquad \begin{bmatrix} K_d & K_{df} \\ K_{fd} & K_f \end{bmatrix} \begin{Bmatrix} u_d \\ u_f \end{Bmatrix} = \begin{Bmatrix} F_w \\ Te + T_f \end{Bmatrix} & (1.1) \\ \dfrac{\partial^2 \Phi}{C_0^2 \partial t^2} = \dfrac{\partial^2 \Phi}{\partial x^2} + \dfrac{\partial^2 \Phi}{\partial y^2} + \dfrac{\partial^2 \Phi}{\partial z^2} & (1.2) \\ [M_g]\{\ddot{u}_g\} + [C_g]\{\dot{u}_g\} + [K_g]\{u_g\} = \{T_b\} & (1.3) \end{cases}$$

Where Equation (1.1) is concerning the motion of dam and foundation, Equation (1.2) is for the motion of free field. And Equation (1.3) deals with the wave propagation in the reservoir water. The matrices M, C, and K indicate mass, damping, and stiffness matrix respectively. The subscripts d, df (fd), f, g mean, in turn, that these matrices are relevant to the dam, the conjunct part of dam and foundation, the foundation, and the free field respectively. The variables $u, \dot{u}, \ddot{u}$ are the vectors of displacement, velocity, and acceleration. Vector F_w is hydrodynamic pressure load acting on the dam. Vectors T_e and T_f are earthquake load acting on rock surface and traction acting on the lateral boundaries of the foundation result from the difference between the foundation vibration and the free field motion. T_b indicates the earthquake load acting on the rock surface (input surface) of the free field. Φ is the velocity potential function of water particles. Variables x, y, z are the components of Descartes coordinates, and t indicates time. C_0 is the sonic velocity in water. C_f^* is the damping matrix including the component of foundation material C_f and that of the viscous boundaries C_b ($C_f^* = C_f + C_b$). Here, the viscous boundary matrix C_b is derived from the principle of virtual work. Its function for energy dissipating has been verified to be much better than that of the traditional viscous boundary, Lysmer[8]. For details of the matrix C_b, please refer to relevant paper, Miura[9].

Between dam and reservoir, the following continuous condition is applied.

$$\begin{cases} \dfrac{\partial \Phi}{\partial n} = V_d \\ F_w = P_w \end{cases} \qquad (2)$$

On the interface between dam and reservoir, the velocity of water particle $\partial\Phi/\partial n$ equals to the vibrating velocity of the dam. And the hydrodynamic pressure P_w is treated as the surface load of the dam. The coupling between dam and foundation has been naturally considered in Equation (1). As for the traction T_f, it is calculated according to the following equation.

$$\{T_f\} = [K_b]\{u_g\} + [C_b]\{\dot{u}_g\} \quad (3)$$

Where, $[K_b]$ is the stiffness matrix of the lateral boundaries of the foundation, and is used for evaluating the effects of free field motion. $[C_b]$ is the viscous boundary matrix, and has been mentioned above.
As mentioned above, since the free field around the foundation is taken into the analytic model, only the boundaries of the reservoir should be treated here.
On the bottom and the valley of the reservoir, the following viscous boundary condition is applied.

$$\frac{\partial \Phi}{\partial t} - C_0 \beta \frac{\partial \Phi}{\partial n_r} = 0 \quad (4)$$

Where, β is the impedance ratio between reservoir bottom sediment and reservoir water. Variable n_r indicates the normal direction of the boundary surface.
On the free surface of the reservoir, the condition of surface wave is applied. And at the upstream boundary of the reservoir, the following viscous boundary condition is applied, Miura[9].

$$\frac{\partial \Phi}{\partial t} = -C_0 \Big/ \sqrt{1 - \frac{1}{\beta^2}} \frac{\partial \Phi}{\partial x} \quad (5)$$

Modeling of Contraction Joints

For modeling the contraction and peripheral joints, an iso-parametric interface element with 0 thickness , as shown in Figure 1, is utilized. Characteristically, this kind of interface element can model any shape of discontinuities. The initial strength, the sliding behavior of the contraction joints and the interaction with reservoir water can be considered by using such element. The basic theory of the interface element is like that of iso-parametric solid element. The coordinates and displacements of any point of the element can be calculated with the corresponding values of the constitutional nodes of the element with the following shape functions.
For corner nodes there is

$$N_i = \frac{1}{4}(1+\xi\xi_i)(1+\eta\eta_i)(\xi\xi_i+\eta\eta_i-1) \quad (6)$$

For middle nodes of $\xi_i = 0$, there is

$$N_i = \frac{1}{2}(1-\xi^2)(1+\eta\eta_i) \quad (7)$$

And for middle nodes of $\eta_i = 0$, there is

$$N_i = \frac{1}{2}(1-\eta^2)(1+\xi\xi_i) \quad (8)$$

Furthermore, the relationship between inner force $\{\sigma\}$ and displacement $\{u\}$ of the interface element can be expressed in the form,

$$\begin{Bmatrix} \sigma_\xi \\ \sigma_\eta \\ \sigma_n \end{Bmatrix} = \begin{bmatrix} K_\xi & 0 & 0 \\ 0 & K_\eta & 0 \\ 0 & 0 & K_n \end{bmatrix} \begin{Bmatrix} u_\xi \\ u_\eta \\ u_n \end{Bmatrix} \quad (9)$$

Where K_i $(i = \xi, \eta, n)$ is the stiffness coefficient in the direction i shown in Figure 1. Its dynamic properties are shown in Figure 2.

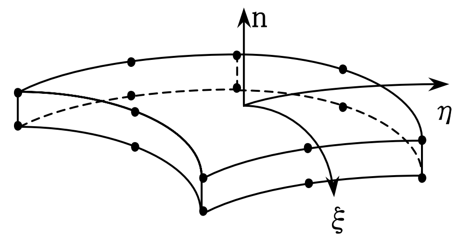
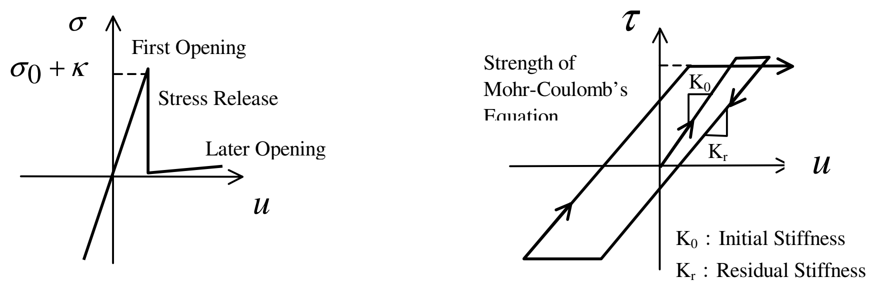


Figure 1 Interface element of 0 thickness



(a) in normal direction (b) in shear direction

Figure 2 Properties of interface element

In the normal direction of the interface element, when the tensile stress exceeds the sum of the initial strength κ and the static stress σ_0 (i.e. $\sigma_t > k + \sigma_0$) an opening of the interface will occur. But once the opening occurred, the resistance of the interface will reduce to σ_0 , because of the loss of the initial strength κ . About the sliding behavior, whether it slides or not will depend on the ratio between the shear stress and the shear resistance defined by Mohr-coulomb's criteria.

For the coupling between contraction joints and reservoir, one node of the reservoir will correspond to two nodes of the joints in the numerical model. Such corresponding relationship is shown in Figure 3 and the coupling condition can be expressed in the following form,

$$\begin{cases} \dfrac{\partial \Phi}{\partial n} = \dfrac{(V_A + V_B)}{2} & \textit{velocity} \quad \textit{condition} \\ P_C = F_A = F_B & \textit{force} \quad \textit{condition} \end{cases} \tag{10}$$

Where Φ, n, V_A, V_B are, in turn, the velocity potential function of reservoir water, the normal direction of the upstream surface of the dam, the velocities of node A and node B of the interface element. The variables P_C, F_A, F_B are the hydrodynamic pressure of point C and the surface load of node A and node B .

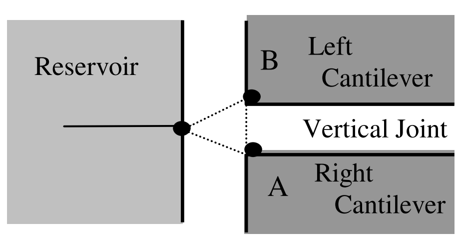


Figure 3 Relation between reservoir and interface element (plane)

APPLICATION TO EARTHQUAKE SAFETY EVALUATION OF EXISTING DAM AGAINST VERY STRONG EARTHQUAKE

Fundamental Flow of Evaluation

First, a simulation of actual behaviors of the existing dam during the 1997 earthquake is carried out. The material properties of the dam and foundation are identified from the reproduction analysis. Then, a strong inland earthquake motion assumed near the dam site is generated. With the analytic model identified above and the earthquake motion assumed, the response of the dam against the possible strong inland earthquake motion is predicated. And based on the analytic results, an assessment of the seismic stability of arch dam is done. The assessment procedure is shown in Figure 4.

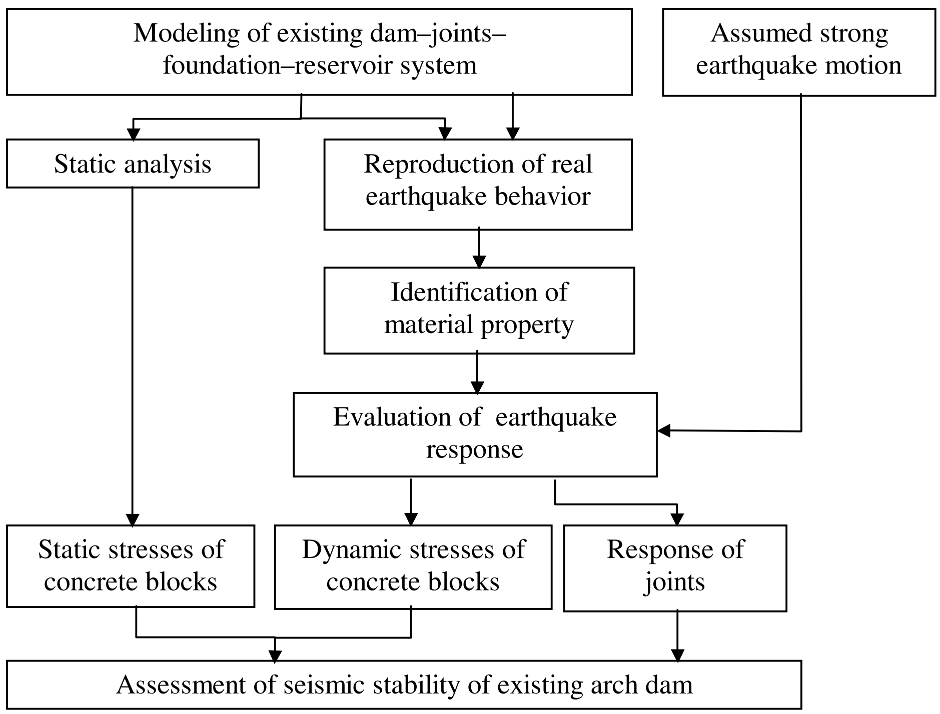


Figure 4 Procedure of seismic stability assessment for existing arch dams

The Shintoyone Dam analyzed in this study

For examining the validity and availability of the analysis method developed in this study, at first, a reproduction analysis for real earthquake behavior of an existing arch dam named Shintoyone has been carried out. The Shintoyone dam is a parabolic thin arch dam constructed in 1972, which is located in Aichi Prefecture about 220km from Tokyo to the west. The height and crest length of the dam is 116.5 meters 311 meters, respectively. It provides us a good opportunity to evaluate the material properties of dam and foundation. Here, a simulation of the earthquake behavior of the dam has been done, and the material property values of the dam and foundation are evaluated based on the analysis.

Earthquake Observation at the Shintoyone Dam

Figure 7 shows the downstream surface, the contraction joint allocation and the arrangement of seismometers .

On 16th March 1997, an earthquake of magnitude 5.8 occurred near the dam site. The epicentral distance was 35km. And the earthquake motion of effective maximum acceleration 709 gal (original value:1000.2gal) was recorded at this earthquake. During this earthquake, the water level of the reservoir was EL.+450 meters (the water depth was 90 meters). Among the records, the time histories of the dam crest center and the dam base inspection gallery are shown in Figure 7. No evidence of joint opening or sliding was found in a visual inspection shortly after the earthquake.

Figure 5 Location of the Shintoyone Dam

Figure 6 The Shintoyone Dam

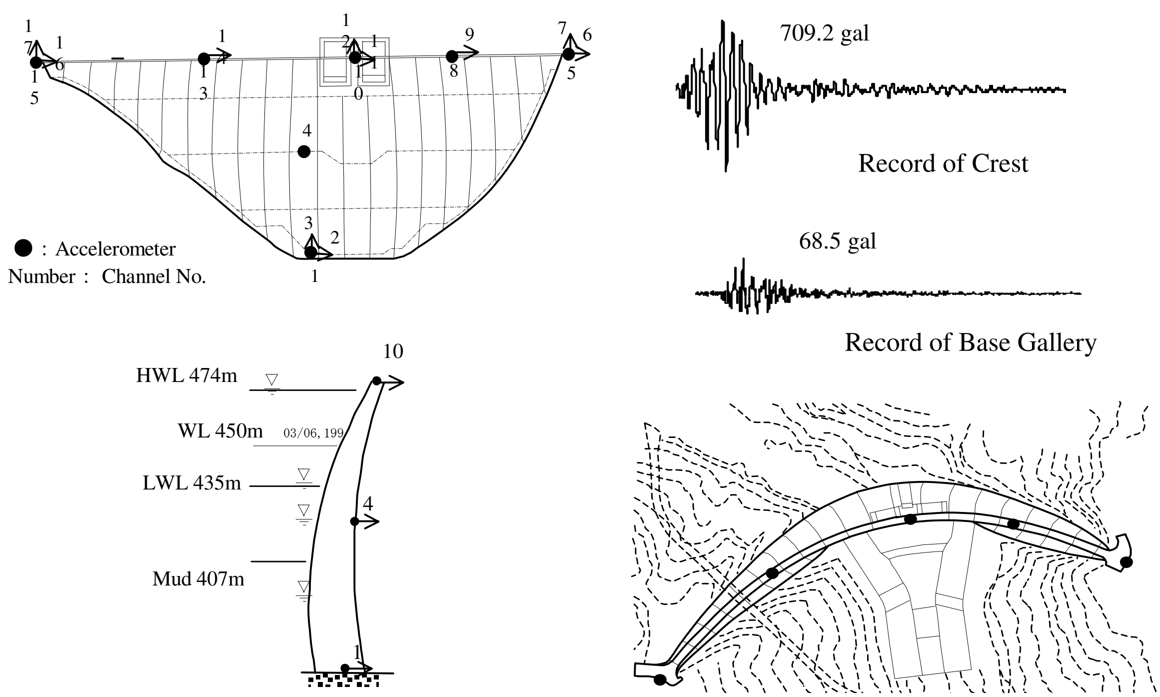
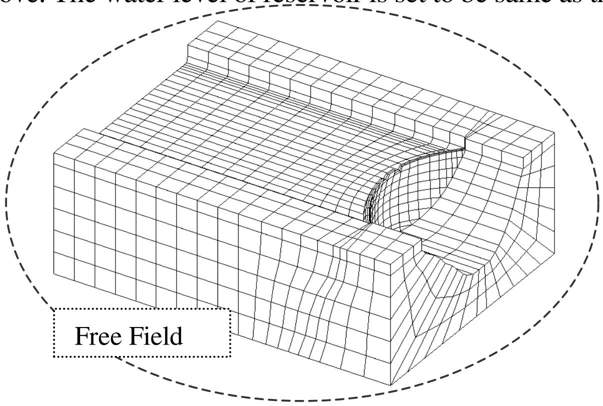


Figure 7 Location of seismometers and acceleration record in the 1997 earthquake

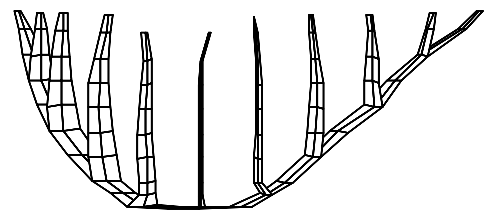
Numerical model

Figure 8(1) shows the 3-D numerical model generated for the coupled dam-joints-reservoir-foundation system. The foundation has a depth (from dam base downward) of 120 meters, and a width of 551 meters

(120 meters extends from abutments to both lateral sides respectively). Additionally, a free field is modeled with layer elements around the foundation, and its imagery is shown in Figure 8(1). The dam and foundation and free field are meshed with finite elements, and the reservoir is meshed with finite difference grids. The vertical joints and the peripheral joints shown in Figure 8(2) are modeled with interface elements mentioned above. The water level of reservoir is set to be same as that during the earthquake with a depth of 90 meters.



(1) 3-D model of the Shintoyone Arch Dam



(2) Position of joint-elements

Figure 8 3-D Model of dam-joints-reservoir-foundation system

Dynamic property values identified

About the material properties, they are identified from the simulation of the dam responses during the 1997 earthquake. Based on the assumption of linear properties, the shear modulus of dam concrete and that of the foundation rock are adjusted until the analysis results approximate the earthquake records. Table 1 shows the identified material properties. About the properties of contraction joints, the values defined based on numerical analysis experience are shown in Table 2, and for all of the joints the initial strength is set to be zero.

The maximum values of acceleration responses got from the simulation are listed in Table 3. The calculated acceleration time history at the dam crest center in the radial direction is shown in Figure 9 and compared with the earthquake observation record. The maximum acceleration calculated is 683gal, about 4% different from that of the earthquake record. Figure 10 shows the transfer function from the base gallery to the dam crest center, and a comparison between analysis and earthquake record is also given in Figure 11. It is clear that the transfer functions are generally consistent even there is a difference around the frequency 7 Hz.

Analysis Condition for Earthquake Safety against Very Strong Earthquake

Against very strong earthquake motions, the joints may show strong nonlinear behaviors such as opening or sliding. And the earthquake responses of arch dam may vary significantly because of the discontinuous effect of joints. For investigating this phenomenon, an earthquake response analysis of the dam subjected to an assumed inland earthquake of Magnitude 8 was carried out (called “Joint Case” later). For examining the effects of joints clearly, a comparative analysis using the model without joint elements (called “No-joint Case” later), has also been carried out and compared with that of the Joint Case. For investigating the effects of reservoir water, two cases i.e., an empty reservoir (called “Empty Case” later) and the case of water level during 1997 earthquake (water depth 90 meters, called “Case of Water” later) are analyzed and compared.

In this analysis, the material properties identified from the simulation mentioned above are used.

Table 1 Dynamic property values identified by the reproduction analysis in regard to the earthquake behavior during the 1997 earthquake

Item	Shear Modulus (MPa)	Density (g/cm^3)	Poisson's ratio	Damping factor (%)
Dam concrete	10500	2.40	0.20	5
Foundation rock	10000	2.60	0.25	5
Free field	10000	2.60	0.25	5

Table 2 Assumed property values of joint elements

Item	Stiffness in normal Direction (N/mm^3)	Stiffness in tangential direction (N/mm^3)	Shear Strength (N/mm^3)	Friction angle
Contraction Joint	2×10^9	1×10^9	0.0	40.0°
Peripheral Joint	2×10^9	1×10^9	0.0	50.0°

Table 3 Maximum acceleration of earthquake observation results and 3-D reproduction analysis

Location	Direction	Earthquake record (gal)		Analysis (gal)
		Original	Treated *	
Crest center	Radial	1000.2	709.2	683.1
	Tangential	790.2	186.0	175.6
	Vertical	548.4	175.1	191.5
1/4 crest from left abutment	Radial	513.8	476.4	423.6
	Vertical	110.6	104.4	110.3
1/4 crest from right abutment	Radial	564.4	550.4	517.9
	Vertical	85.8	88.3	97.6
Middle Height of center	Radial	205.4	125.0	114.3
Base gallery	Radial	70.1	68.5	67.8
	Tangential	46.2	39.5	36.3
	Vertical	46.8	45.1	47.1
Left abutment	Radial	44.1	44.9	45.2
	Tangential	56.8	57.2	46.5
	Vertical	53.4	51.4	48.7
Right abutment	Radial	68.1	66.9	42.8
	Tangential	44.9	45.0	42.2
	Vertical	69.9	69.7	55.9

* Components of the frequency higher than 30 Hz were cut

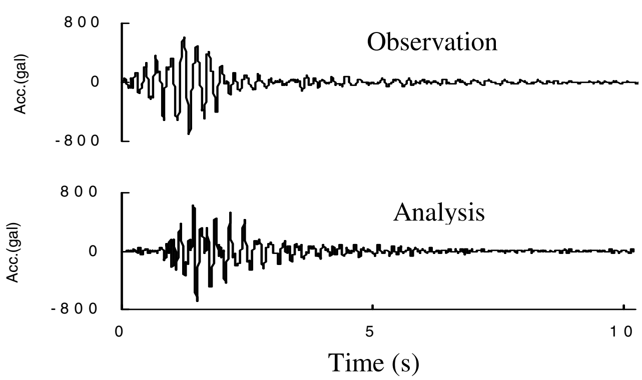


Figure 9 Comparison of earthquake motion at the dam crest center

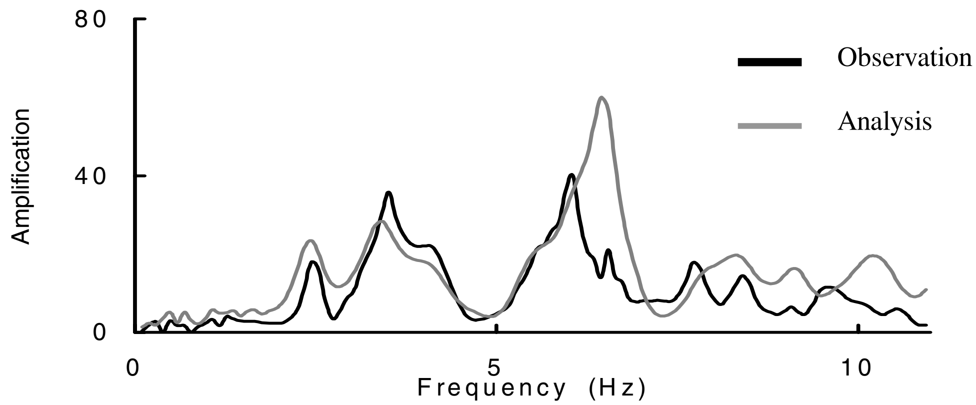
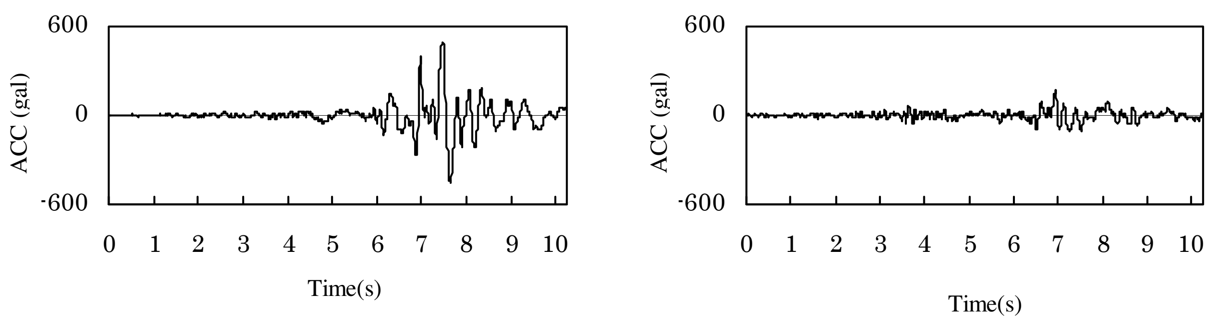


Figure 10 Comparison of transfer function from dam base gallery to dam crest center (1997 Near field earthquake M5.8 in the Aichi Prefecture)

Assumed Strong Earthquake Motion

As for the ground motion at the dam base, the earthquake records observed at the Hitokura dam, which was struck by the 1995 Hyogoken-Nanbu earthquake, is used. The maximum amplitude is normalized to be 500 gal for corresponding to the assumed Magnitude 8 inland earthquake. Figure 11 shows the acceleration time histories in both stream and vertical directions. Unfortunately, no record of vertical component of the earthquake was obtained.



(1) in the stream direction (2) in the vertical direction

Figure 11 Assumed strong earthquake motions at the dam base for Earthquake safety evaluation against very strong motions

Results of Earthquake Safety Analysis against Very Strong Earthquake Motion

Behavior of contraction joints

Figure 12(a) shows the responses of the vertical joint in the middle of the crest on the upstream side, and Figure 12(b) illustrates that of the peripheral joint in the base of the crown cantilever. The maximum openings of the two joints are 1.6 cm and 2.6 cm, respectively. Because of the static water pressure acting on the upstream face, the peripheral joint in the base of the crown cantilever was in a slightly tensile status before the earthquake. Consequently, a relatively large opening response occurred during the earthquake. Generally, initial strength of the contraction joints exists due to the grouting after construction, but in the analysis such advantageous term was neglected. That is to say, the real opening distance may be smaller than the value got from the analysis. About the sliding behavior, a bilateral slippage of 0.19 cm occurred at the joint in the left quarter of the crest, and the maximum slippage of 0.7 cm occurred at the base of the crown cantilever.

Behavior of the dam

The maximum response values are listed in Table 4. The contours of the maximum tensile arch stress and the history of the arch stress of the element in the dam crest center (on the upstream side) are shown in Figure 13 and Figure 14, respectively.

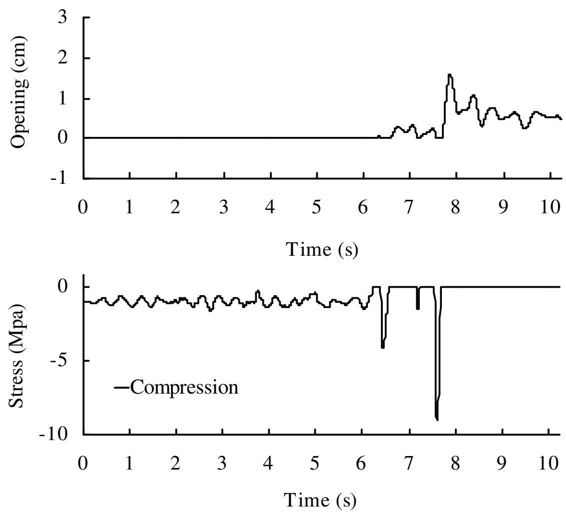


Figure 12(a) Response of joint in the dam crest center on the upstream side

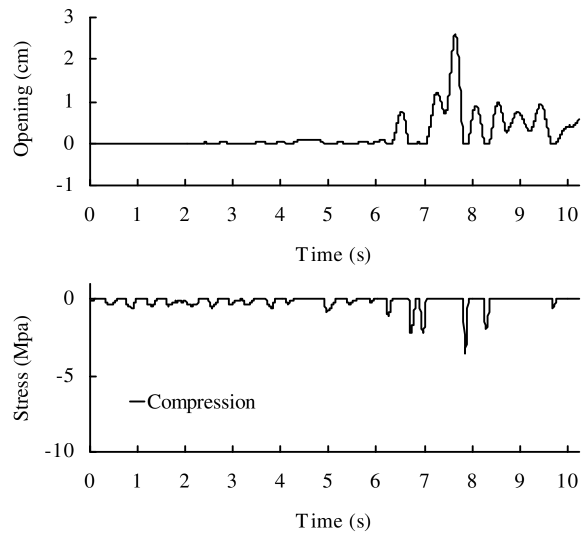
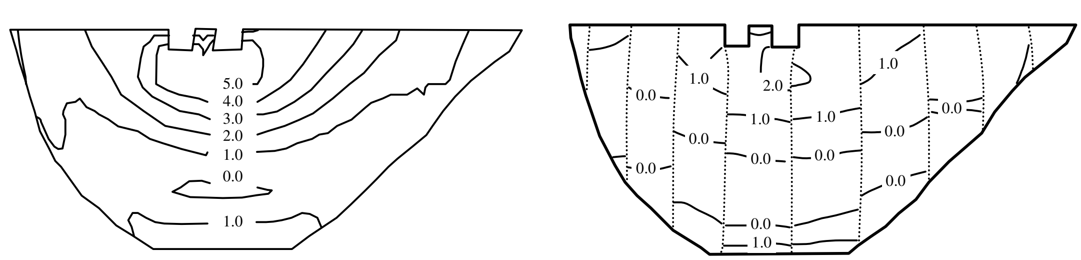


Figure 12(b) Response of joint at the base of the crown cantilever

Table 4 Effects of joints on the responses of dam

Maximum Values	Component	No-joint Case	Joint Case
Tensile stress (MPa)	Arch	9.32	3.17
	Cantilever	6.76	4.05
Compressive stress (MPa)	Arch	9.14	10.59
	Cantilever	7.48	8.64
Tensile strain ($\times 10^{-4}$)	Arch	3.99	1.67
	Cantilever	2.28	1.92
Compressive strain ($\times 10^{-4}$)	Arch	3.25	3.58
	Cantilever	2.77	2.93
Acceleration (gal)	Stream	3027	3080
Displacement(cm)	Stream	11.99	14.23



(a) No-joint Case (b) Joint Case

Figure 13 Comparison of maximum tensile arch stress on the upstream face (MPa)

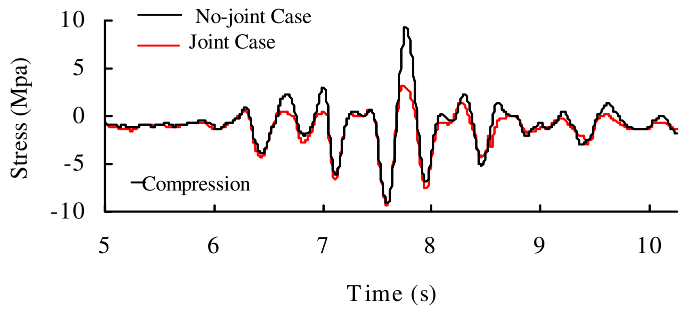


Figure 14 Response of the arch stress of the element at the top of crown cantilever

Comparison with the No-joint Case shows that the tensile stress becomes much smaller in the Joint Case. In the No-joint Case, the maximum tensile arch stress was 9.32 MPa, a value much larger than the tensile strength (3 ～5 MPa) of common dam concrete, Hatano [10]. On the other hand, the maximum tensile arch stress reduced to 3.17 MPa, i.e., about 2/3 of the stress released due to the joint opening in the Joint Case. Correspondingly, the maximum tensile strain (at the dam crest center in the arch tangential direction) was 3.99×10^{-4}, as is the case with stress, the value greatly excesses the common knowledge of concrete. Similarly, the effects of the peripheral joints also resulted in a release of the tensile cantilever stress. About compressive stresses, both the arch stress and cantilever stress increased somewhat in the Joint Case. It is interpreted that such phenomenon is due to the stress redistribution at the moment of joint opening. Furthermore, the behavior of joints affects not only the maximum values of stress and strain, but also their distributions. In the No-joint Case, the tensile arch stress mainly concentrated in the area around crown cantilever, but it gets to be more uniform in the Joint Case. Generally say, these variations result from the effects of joint behaviors are advantageous to the seismic stability of dams. Therefor, these effects should be considered for getting an accurate assessment on the seismic stability of dams. As well, the maximum relative displacement of the dam in the stream direction increased 19% in the Joint Case, but no evident variation of acceleration response was found in this analysis.

Effects of reservoir water

For investigating the effects of reservoir water, the comparison between the Empty Case and the Case of Water has been done. Figure 15 shows the response of the vertical joint in the dam crest center on the upstream side. And Figure 16 shows the tensile arch stress of the upstream face. Comparison between Figure 12 and Figure 15 shows that the tensile arch stress increased in the Empty Case. The maximum stress got to be about 4.0 MPa in the area close to the bottom of spillway pier. On the other hand, from Figure 13 and Figure 16 it can be found that the behavior of the vertical joint in the dam crest center became much more intensive in the Empty Case. The maximum opening of the joint reached 8.0 cm, and a residual opening was generated. However, as mentioned above, the grouting of the joints in a real dam may restrain the joint response. Generally speaking, an arch dam existing in impounding status is safer. This phenomenon is interpreted to be due to two main reasons. One is that, in the Empty Case, some joints are in a naturally opening status before earthquake because of the non-stress (compression) effect. The other is that, in the Case of Water the vibrating energy can dissipate to the half-infinite reservoir water.

Based on the analytic results an assessment of the seismic stability of the dam can be made. In a strong earthquake of the maximum ground acceleration of 500 gal, the maximum tensile stress of the dam may reach the order 3～4 MPa. Since the maximum stress occurs momentarily and in a limited area, even some slight cracks may occur, large-scale collapse will never be possible. If any, some light vertical cracks may occur in the area near the bottom of the spillway pier since big tensile arch stress tends to be here.

And some light horizontal cracks may occur in the area near the bottom of the crown cantilever since big tensile cantilever stress rises here.

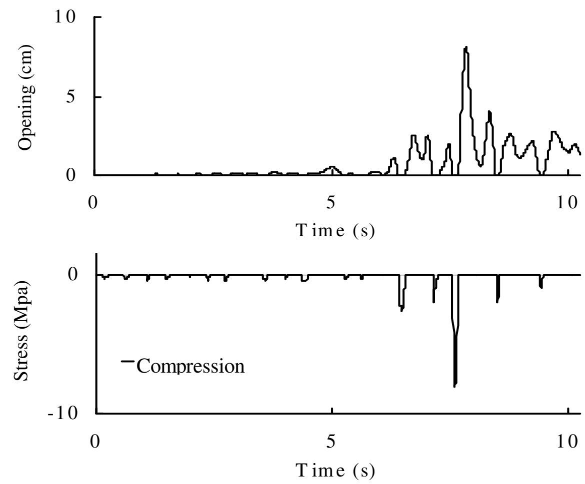


Figure 15 Response of the joint in the dam crest center on the upstream side (Empty Case)

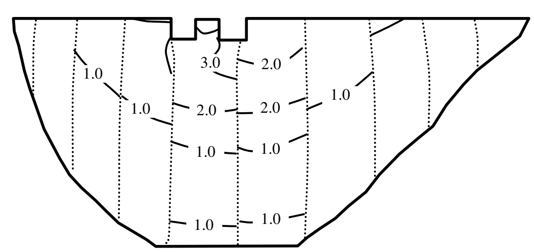


Figure 16 The maximum tensile arch stress on the upstream face (MPa) (Empty Case)

In regard to the possibility of compressive failure, it is no doubt that there is enough safety margin since the maximum compressive stress is only in the order about half of the compressive strength of ordinary concrete. As respects the capacity of the sealing strips instrumented in the contraction joints, the installment conditions are examined with the analytic results. The maximum opening, 8.0 cm in the Empty Case, indicates that the sealing strips may sustain partly breaking failure during such strong earthquakes, but when the reservoir is impounded static water pressure will mitigate joint behavior and let the sealing strips be sufficiently admissible to the possible opening.
The examination mentioned above leads to a recognition that reservoir water has favorable influence on both dam concrete and sealing strips.

CONCLUSIONS

From this study, the following conclusions can be drawn:
1) 3-D dynamic analysis method for coupled dam-joints-foundation-reservoir system has been developed, which can simulate the dynamic interaction between dam and foundation and between dam and reservoir, the energy dissipation and inflow at the lateral boundary, the non-linear effects of dam materials and the discontinuous effects of joints. For examining the validity and availability of the method developed , a reproduction analysis and earthquake safety assessment of an existing arch dam was carried out.

2) When struck by very strong earthquake motion, the contraction joints may show strong non-linear behaviors, and exert significant effects on the responses of arch dam. In seismic stability analysis of concrete dams, a coupled dam–joints–reservoir–foundation model should be used. The method developed in this study is an available tool for such analysis. In numerical analysis, a model without such contraction joints may give an acceptable result for the case of weak earthquake motion, but will result in an unreasonable tensile stress for the case of strong earthquake.

3) The main effect of contraction joints on the responses of dam is on stress. The opening of vertical joints releases the tensile arch stress, and the opening of peripheral joints releases the tensile cantilever stress. But compressive stresses increase slightly in both arch direction and cantilever direction.

4) From the simulation of real earthquake behaviors of the Shintoyone dam, the dynamic shear modulus of the dam concrete is identified to be 10500 N/mm^2, and the damping factor to be 5%. For foundation rock, the dynamic shear modulus and damping factor is 9600 N/mm^2 and 5% respectively.

5) In a strong earthquake of the maximum amplitude 500 gal, the dam may suffer some light damage. Some tiny vertical cracks may occur in the limited area near the bottom of spillway pier, and some tiny horizontal cracks may occur in the areas near the heel and toe of the dam. However sudden collapse or serious damage will never take place.

6) The higher the reservoir water level, the more stable the contraction joints. Due to the effects of static water pressure acting on dam face and radiation damping to the semi infinite reservoir zone, an arch dam in an impounding status is usually of higher seismic stability than that in empty status.

7) As for the effect of reservoir, the added mass method is generally used. The added mass method is convenient and easy way, but it is considered that the added mass method is not an accurate method to reproduce the actual dynamic interaction between dam and reservoir.

8) As future research topic, the earthquake observation of the joint behaviors of existing dams should be improved. Further efforts should be made on the research of material properties of contraction joints as well as the identifying methods.

REFERENCES

1. Clough R.W.: Nonlinear mechanisms in the seismic response of arch dams, International research conference on earthquake engineering, Skopje, Yugoslavia, 1980
2. Watanabe H., Razavi S. D., Takashima K., Taniyama H.: Interaction of Material Nonlinearity and Joint Opening on the Seismic Response of a Concrete dam, Dam Engineering, Vol.10, No.4, 276-288, 2000
3. Fenves G.L.: Response of Pacoima Dam in the 1994 Northridge, California, Earthquake, News EECR, University of California at Berkeley, Vol.17, Number 4 Oct., 1996
4. Ariga Y.,: Study on quantitative evaluation of dynamic property of dams by 3-D reproduction analyses , Thesis for doctorate of Saitama University , 2001.3
5. Watanabe H., Ariga Y., Cao Z.: Earthquake Resistance of a Concrete Gravity Dam Revaluated with 3-D Nonlinear Analysis, Proc. of JSCE, No.696/I-58,99-110,2002.1
6. Chopra A. K., Chakrabarti P.: Earthquake analysis of concrete gravity dams including dam – water – foundation rock interaction, *Earthquake Eng. and Structural Dynamics*, Vol.9, 363-383, 1981
7. Shiojiri H., Ueda M.: Three - Dimensional Response Analysis of the Hitokura Concrete Gravity Dam to the 1995 Hyougoken-Nanbu Earthquake, Proc. of JSCE, No.640/I-50,177-192, 2000.1
8. Lysmer J., Kuhlemeyer R.L.: Finite dynamic model for infinite media, Proc. of ASCE, EM4, pp.859-877, 1969
9. Miura F., Okinaka H.: Dynamic analysis method for 3-D soil-structure interaction systems with the viscous boundary based on the principle of virtual work, Proc.of JSCE, No.404/I-11, pp.395-404, April, 1989
10. Hatano T.: Theory of Failure of Concrete and Similar Brittle Solid on the Basis of Strain, Proc. of JSCE, No.153, pp.31-39, 1968.5