INTERNATIONAL SYMPOSIUM ON
DAMS FOR A CHANGING WORLD
-Need for Knowledge Transfer across the Generations & the World
Kyoto, Japan June 5, 2012

Preliminary Study on Earthquake-induced Progressive Failure of Jointed Rock Foundation Supporting Arch Dam

H. Kimata, Y. Fujita & P. Rungbanaphan
Shimizu Corporation, Tokyo, Japan
kimata@shimz.co.jp

ABSTRACT:
A 2-dimensional nonlinear analysis has been carried out to simulate an experiment which has been performed by other researchers in order to evaluate the seismic performance of arch dams. The effect of jointed rock foundation is taken into consideration in both experiment and simulation and good agreement is found between their results. Therefore, it can be said that the proposed model can roughly simulate the progressive failure of jointed rock foundations and can be used in further similar researches.

Keywords: dam foundation, progressive failure, jointed rock, experiment, 2-D nonlinear FEM

1. INTRODUCTION

On December 2, 1959, the Malpasset dam failed explosively due to the shift of its left abutment caused by a thin clay filled seam in the rock behind the abutment. This displacement and the loss of support led to the cracking at the center and finally collapse of the dam as shown in Fig. 1 (MA 1960). Since then, the study and prediction of the behaviour of rock foundations of the dams have gained much attention. Authors already studied the behaviour of RC gravity dams on jointed rock foundation using nonlinear analysis and could effectively evaluate the seismic performance of this type of dams (Kimata et al. 2009a, 2009b and 2010).

In this paper, a 2-dimensional nonlinear analysis has been carried out to simulate an experiment which has been performed by other researchers in order to evaluate the seismic performance of arch dams (Takano 1962). The effect of jointed rock foundation is taken into consideration in both experiment and simulation and, therefore, by comparing the experiment data and analysis results, the validity of the constitutive model which is used in modelling of the joints at the rock foundation can be evaluated.

2. OVERVIEW OF THE EXPERIMENT

2.1. Experiment set-up and specimen

The experiment set-up and loading frame can be seen in Fig. 2. In order to assess the seismic safety of arch dams in the experiment, the abutment of arch dams is modelled as shown in Fig. 2. The thickness of the specimen which includes both dam body and rock foundation is 50 mm. The specimen is placed inside a loading frame with a dimension of 1000mm×860mm and loading is applied using 13 jacks on the dam body in order to represent the water pressure (including increased dynamic water pressure due to seismic motions).

As for the rock foundation, it is constructed using square elements with a dimension of 20mm×20mm made by mortar. In order to reproduce the joints and discontinuity inside the rock, a thin strip with a thickness of 2 mm made by the mixture of polyester with a certain shear property and diatomite is located between each two rock elements. Finally, the load is applied incrementally and the deflection of the model is measured.

Figure 1. Malpasset dam (before and after the collapse, MA 1960)

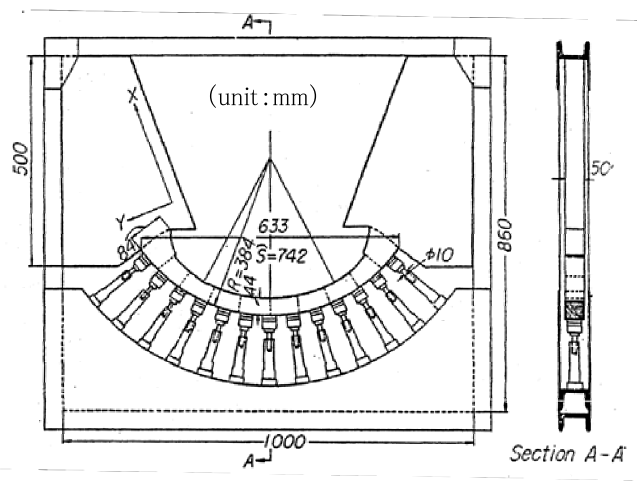


Figure 2. Experiment setup to assess the seismic safety of arch dams (MA 1960)

2.2. Failure pattern of the experiment

The failure pattern of the experiment is shown in Fig. 3. As it can be observed in this figure, the compressive load is applied to the rock foundation through the arch of the dam based on arch action. This pushing force would cause shear damage to the jointed rock foundation parallel to the load direction and make some openings between the rock elements which penetrates deeply inside the rock foundation. This damage pattern would make the rock foundation swell towards the inner part of the specimen which is not restraint. This swelling of rock foundation occurs not only at the vicinity of the connection with the dam, but also penetrates to almost deep parts of the foundation. Therefore, it can be said that if the thickness of the rock foundation which supports the dam at its inner side would not be sufficient, remarkable damage can happen which may lead to the collapse of the dam. Finally, it can be concluded that the failure of the model takes place when the openings between the rock elements spread through the foundation and result in a great loss of contact force which supports the dam model.

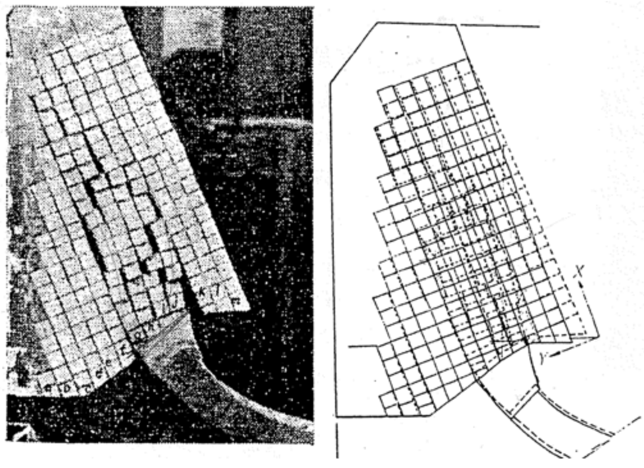


Figure 3. Failure pattern of the specimen

3. OVERVIEW OF THE ANALYTICAL MODEL

A two dimensional plane strain Finite Element model which is shown in Fig. 4 is used in this paper to simulate the above-mentioned experiment. 4-node quadrilateral elements with linear elastic properties are utilized to model the rock foundation since they do not suffer any damage during the experiment. Due to the symmetry of the experiment model, only half of that is modelled in the analysis as shown in Fig. 4.

Boundary condition of the analytical model is considered to the same as that of the experiment, i.e. the nodes which are located next to the loading frame are fixed and leaving the other nodes free as shown in Fig. 4. The incremental loading is also applied at the location of jacks in the experiment by some concentrated loads.

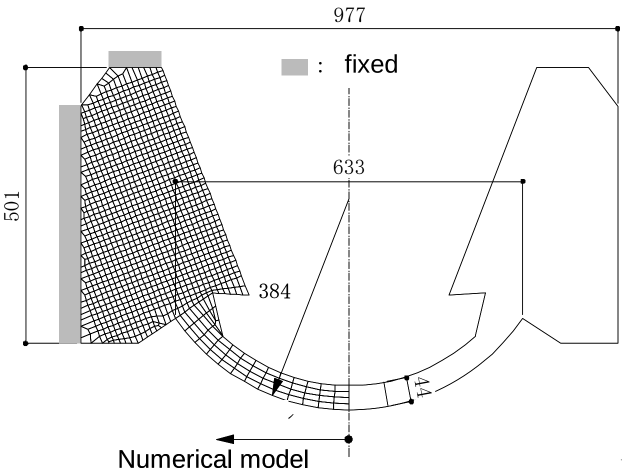


Figure 4. Analytical model

The deformation properties of interface element which is used as joints are as follows:

(i) The relationship between shear stress τ and shear strain γ is assumed to be elasto-perfectly plastic hysteresis as shown in Fig. 5a. The shear strength τ_f is defined by Mohr-Coulomb's failure criteria, Eq. 1. Tensile strength and shear stress at tensile normal stress are equal to zero as shown in Fig. 5c.

$$\tau_f = \pm c \mp \sigma \tan\phi \tag{1}$$

where c: cohesion, ϕ: internal friction angle.

(ii) The relationship between normal stress σ and normal strain ε is elastic linear in compression side and the rigidity is zero in tension side as seen in Fig. 5b. The interface elements are defined inside the rock elements as shown in Fig. 5d.

(iii) In the stress-strain relationship shown by Eq. 2, matrix $[D]$ is expressed by Eq. 3 for the elastic strain component and Eqs. 4a to 4c for the plastic strain component. As seen in the non diagonal term in Eq. 4a, shear stress increment due to the confining pressure is taken into consideration when the value of σ is positive in the plastic region. On the other hand, the influence due to the roughness of the joints, namely dilatancy, is disregarded. An extreme small value is given to S_v in order to avoid numerical ill-conditioning when the value of diagonal component is zero. The material properties of rock foundation and interface elements which are assumed in the analysis are shown in Table 1.

$$\begin{Bmatrix} \tau \\ \sigma \end{Bmatrix} = [D] \begin{Bmatrix} \gamma \\ \varepsilon \end{Bmatrix} \tag{2}$$

where

$$[D] = \begin{bmatrix} G & 0 \\ 0 & E \end{bmatrix} \tag{3}$$

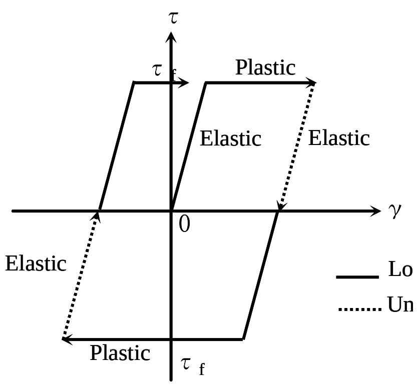
for elastic strain component, and

$$[D] = \begin{bmatrix} S_h & -S_v \cdot \tan\phi \cdot Sign(\tau) \\ 0 & S_v \end{bmatrix} \tag{4a}$$

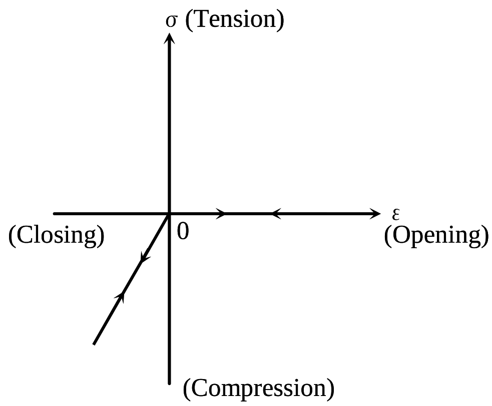
$$S_h = 0(= G/1\times10^4) \tag{4b}$$

$$S_v = \begin{cases} E(\sigma < 0) \\ 0(= E/1\times10^4)(\sigma \geq 0) \end{cases} \tag{4c}$$

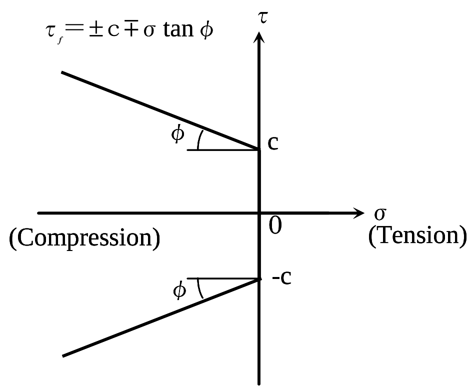
for plastic strain component. where E: Young's modulus, G: shear modulus, Sign(τ): +1 ($\tau \geq 0$), –1 ($\tau \leq 0$).



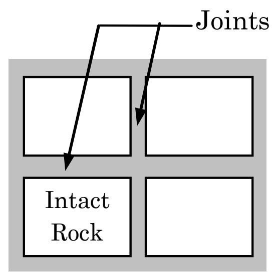
(a) Shear stress vs. shear strain



(b) Normal stress vs. normal strain



(c) Shear strength (Mohr-Coulomb failure criterion)



(d) Arrangement of joint elements

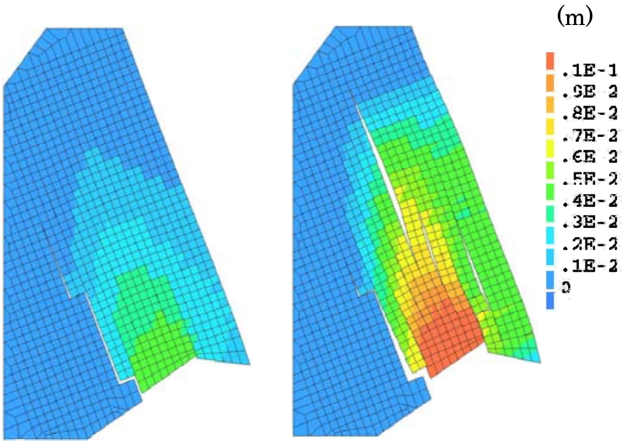
Figure 5. Constitutive model for joint interface elements

Table 1. Material properties assumed in the analysis

Material	Rock Foundation	Joint Interface
Young Modulus E (GPa)	2.0	2.0
Poison ratio ν	0.2	0.2
Density γ (kg/m^3)	2000	-
Cohesion (MPa)	-	0.12
Internal friction angle (°)	-	18

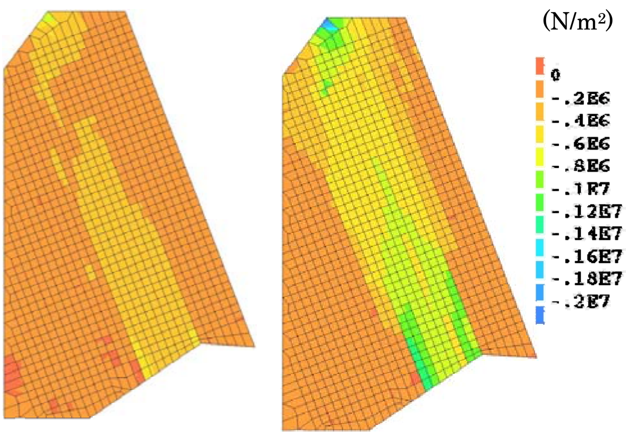
4. ANALYSIS RESULTS

The deflection pattern and minor principal stress of the rock foundation which are obtained by conducting two-dimensional nonlinear analysis are shown in Fig. 6 to 7, respectively. It can be said that by increasing the applied load, the rock elements tend to deflect towards the inner part of the foundation as observed in the experiment. As shown in Fig. 6, this trend can be easily seen when the load becomes large enough. Furthermore, Fig. 7 shows that applied load would be transferred to the deep parts inside the rock foundations and finally both ends of the abutment would damage due to the incremental loading. Therefore, it can be said that the analytical results have the same trend as the experimental in general data and the shear damage and openings at the joints between the rock foundations are well simulated and observed in the analysis as well.



(a) Apply load = 88kN/m^2 (b) Apply load = 196kN/m^2

Figure 6. Deflection pattern of rock foundation in the analysis



(a) Apply load = 88kN/m^2 (b) Apply load = 196kN/m^2

Figure 7. Minor principal stress pattern of rock foundation in the analysis

In addition, the swelling of the rock foundation in the analysis is picked up as a damage index and is compared to that of the experimental data in terms of relative displacement in 5 locations inside the rock foundation as shown in Fig. 8. At some certain level of loading, the relative displacement sharply starts to increase in the analysis results while experimental data show a smoother incremental trend. When the opening failure occurs, in fact, abrupt increase of the relative displacement along the opening surface is expected. It can thus be said that the analysis can simulate more accurate failure bahavior in this case. However, the relative displacement of the model roughly demonstrates the similar trend in both experiment and analysis. Therefore, the constitutive model used for simulating the joint elements is considered to be accurate and acceptable.

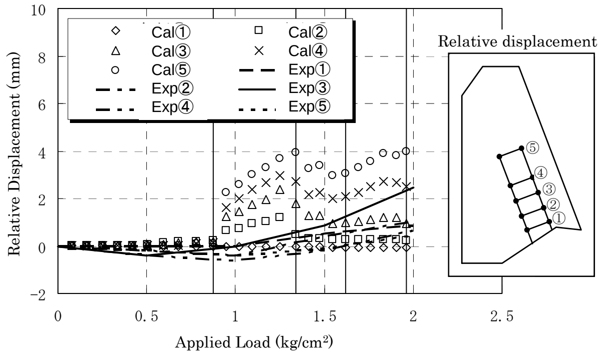


Figure 8. Comparison between the experimental and numerical results

5. CONCLUSION

In this paper, a two-dimensional nonlinear Finite Element analysis is performed using joint interface elements with special deformation properties in order to simulate the progressive failure of jointed rock foundations of arch dams. Obtained results are compared with the data of an experiment under similar conditions and good agreement is found between them. Therefore, it can be concluded that the proposed model can roughly simulate the progressive failure of jointed rock foundations and can be used in further similar researches.

REFERENCES

Kimata, H., Fujita, Y., Horii, H. and Yazdani, M. (2009a): Dynamic crack propagation analysis of concrete gravity dams with jointed rock foundation, 64th Annual Conference of JSCE (in Japanese).

Kimata, H., Fujita, Y., Horii, H. and Yazdani, M. (2009b): Dynamic crack propagation analysis of concrete gravity dams with jointed rock foundation, 3rd Japan-Greece Workshop on Seismic Design, Observation, and Retrofit of Foundations, proceedings of the workshop: pp.486-494.

Kimata, H., Fujita, Y., Tamai, S. and Okhoval, M.R. (2010): Nonlinear dynamic behavior of concrete gravity dams with jointed rock foundation, 65th Annual Conference of JSCE (in Japanese).

Ministere de l'Agriculture (MA) (1960): Final Report of the Investigating Committee of the Malpasset Dam, France: pp. 166–172.

Takano, M. (1962) Experimental study on the safety of arch dam foundations, Journal of JSCE, No.78.