Proceedings of the World Congress on Engineering and Computer Science 2010 Vol II
WCECS 2010, October 20-22, 2010, San Francisco, USA

Seismic Analysis of Double Curved Arch Dams Based Performance

E. Mirzaei, S. Vahdani and R. Mirghaderi

***Abstract*—The response of dams subjected to seismic excitation can be evaluated by a number of analysis methods. The traditional approach is to employ linear static or dynamic analyses coupled with appropriate modification to account for inelastic response while current design practice is moving towards an increased emphasis on nonlinear static analysis methods. In this study, a seismic response analysis of a double-curvature arch dam has been performed using nonlinear static analysis similar to capacity spectrum method. The method involves choosing the proper coordinate to evaluate the structure's behaviour using a suitable push load profile, conversion of seismic demand to the same coordinate, and determination of the performance state. The results of the developed algorithm have been compared with time history dynamic analysis of Karoun IV double curved arch dam which shows reasonable agreement with considerable less expenses and complications.**

***Key Words*—Concrete Arch Dams, Nonlinear Analysis, Performance Based Design; Pushover Analysis, Seismic Assessment**

I. Introduction

Concerns about the seismic safety of concrete dams have been growing during recent years, partly because the population at risk in locations downstream of major dams continues to expand and also because of the inadequacy of seismic design codes once the old dams were being built. [1] The hazard posed by large dams has been demonstrated since 1928 by the failure of many dams of all types and in many parts of the world. However, no failure of a concrete dam has resulted from earthquake excitation; in fact the only complete collapses of concrete dams have been due to failure in the foundation rock supporting the dams. On the other hand, two significant instances of earthquake damage to concrete dams occurred in the 1960s: Hsinfengkiang in China and Koyna in India. The damage was severe enough in both cases to require major repairs and strengthening, but the reservoirs were not released, so there was no flooding damage. This excellent safety record, however, is not sufficient cause for complacency about the seismic safety of concrete dam, because no such dam has yet been subjected to maximum reservoir. For this reason it is essential that all existing concrete dams in seismic regions, as well as new dams planned for such regions, be checked to determine that they will perform satisfactory during the greatest earthquake shaking to which they might be subjected. [1]

The prediction of the actual dynamic response of concrete arch dams to earthquake loading is a very complicated problem and depends on several phenomena such as interaction of the dam with its foundation and the reservoir water [2], [3], [4], [5], hydrodynamic pressure produced in reservoir [6], [7], effects of foundation inhomogenetity [8], the presence of contraction joints in the dam body [9], [10], [11], concrete cracking and the nonlinear inelastic behavior of concrete material. Therefore, the nonlinear dynamic time history analysis of the dam, its foundation and water seem to be the only solution in the case of seismic assessment of concrete arch dams.

Push over analysis has been introduced to overcome the difficulties of time history analysis in the case of tall buildings. Over the past twenty years the static push-over procedure has been presented and developed by several authors, including Saiidi and Sozen, Fajfar and Gaspersic, Bracci et al. , amongst others. This method is also described and recommended as a tool for design and assessment purposes for the seismic rehabilitation of existing buildings. [12] The purpose of push-over analysis is to evaluate the expected capacity of a structural system by estimating its strength versus its deformation under the effect of a lateral profile in a static inelastic analysis, and by comparing this capacity with seismic demands to achieve the performance of the structure.

In the present work, an attempt has been made to simplify the nonlinear time history analysis of double-curvature arch dams using the basic concepts employed in pushover analysis. The key aspects such as selecting the proper coordinate system, pushing load profile, setting the nonlinear analysis including the main affecting nonlinear phenomenon, and converting the seismic demand presented by codes to the selected coordinate, has been considered and implemented to evaluate the effectiveness of the method and the results are compared with the time history analysis.

II. Coordinate System and Required Conversion

In nonlinear static procedure of tall buildings, the lateral displacement of the highest point and the lateral base shear of the building have been chosen as the described coordinate system, which can characterize a unique behavior of the structure. In the case of arch dams, the maximum nodal displacement on the crest and the total lateral reactions of the dam body seem to be the desired coordinate system, and has been chosen in this work. Fig. 1 shows the schematic capacity curve in the case of arch dams.

Manuscript received August 2, 2010.
E. Mirzaei is Graduate Student of Civil Engineering, University of Tehran, Tehran, Iran (corresponding author to provide phone: 00989123805825; e-mail: elahehmirzaei82@yahoo.com).
S. Vahdani is Faculty of Civil Engineering, University of Tehran, Tehran, Iran (e-mail: svahdani@ut.ac.ir. colostate.edu).
R. Mirghaderi is Faculty of Civil Engineering, University of Tehran, Tehran, Iran (e-mail: rmirghaderi@ut.ac.ir . colostate.edu).

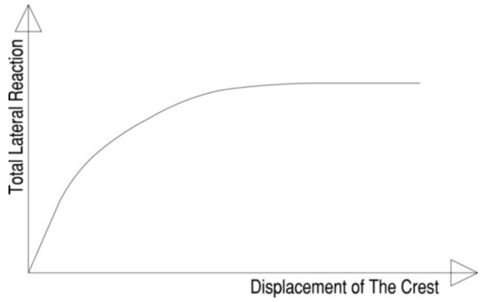

Fig. 1 schematic capacity curve in the case of arch dams

III. CONVERSION TO ADRS FORMAT

Application of the capacity-spectrum technique requires that both the demand response spectra and structural capacity curves be plotted in the spectral acceleration versus spectral displacement domain, which is known as acceleration–displacement response spectra (ADRS). To convert a response spectrum from the standard pseudo-acceleration S_a versus natural period T format to ADRS format, it is necessary to determine the value of S_{di} for each point on the curve, S_{ai}, T_i. This can be done with the equation:

$$S_{d_i} = \left(\frac{T_i^{\,2}}{4\pi^2}\right) S_{a_i} g. \tag{1}$$

In order to develop the capacity spectrum from the capacity curve, it is necessary to carry out a point by point conversion to the first mode spectral coordinates. Any point of base-shear V and roof displacement Δ_{roof} on the capacity curve is converted to the corresponding point S_{ai} and S_{di} on the capacity spectrum using the equations:

$$S_{a_i} = {}^{V_i/W}\!/_{\alpha_1} \tag{2}$$

$$S_{d_i} = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}} \tag{3}$$

where α_1 and PF_1 are the modal mass coefficient and participation factors for the first natural mode of the structure, respectively. $\phi_{roof,1}$ is the roof level amplitude of the first mode and W is the total weight of the structure. The mathematical expressions for the aforementioned parameters are as follows:

$$PF_1 = \left[\frac{\sum_{i=1}^{N}(w_i\phi_i)/g}{\sum_{i=1}^{N}(w_i{\phi^2}_{i1})/g}\right] \tag{4}$$

$$\alpha_1 = \frac{[\sum_{i=1}^{N} w_i\phi_{i1}/g]}{[\sum_{i=1}^{N} w_i/g][\sum_{i=1}^{N}(w_i{\phi^2}_{i1})/g]} \tag{5}$$

IV. MODELING AND BASIC PARAMETERS

The proposed method is used to calculate the seismic dynamic responses of the arch dam Karun IV. Karun IV dam is constructed on the Karun River, immediately upstream of the Monj and Karun confluence, in the south-west of IRAN. Completed in 2010, the 230m high double-curvature arch dam currently ranks 28th among the tallest in the world. It has 19 blocks, with a crest length of approximately 440m, and nine inspection galleries. The crest width is 7m and varies from 37 to 52m at the base.

A. *Model*

Modelling and analysis of the dam-foundation structure has been performed using nonlinear FE program. The arrangement of finite element meshes and nodes (1490 elements and 2680 nodes) is shown in Fig. 2. The Solid 65 element has been employed because of its capability of modelling cracking in tension and crushing in compression, therefore is suitable to account for material nonlinearity in the dam's structure. The element consists of eight nodes and utilizes isotropic material properties.

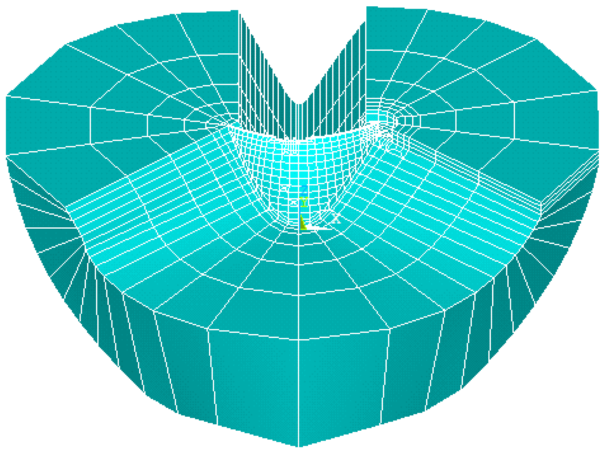

Fig. 2 FE model of dam-foundation structure

B. *Material Properties*

In this study; the failure criterion for concrete due to a multiaxial state of stress is that defined by William and Waranke's five parameter model [14]. Two input parameters viz., the uniaxial tensile strength (f_t) and the ultimate uniaxial compressive strength (f_c). Due to the fact that several contraction joints exist along the structure of the dam which have zero tensile strength, and for the purpose of avoiding further complexity in the modeling process, the tensile strength of concrete which is approximately 3.9 MPa is assumed to be 0.1 MPa. The uniaxial compressive strength of the one-year concrete used in the dam's structure has been reported by Mahab Ghodss Consulting Engineers to be 39.0 MPa under seismic conditions. The crack interface shear transfer coefficient (β_t) for open cracks is assumed to be 0.3 whereas for closed cracks the shear transfer coefficient (β_c) is assumed to be 0.7.

ISBN: 978-988-18210-0-3
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)

C. Basic Parameters

The concrete used in the dam's structure is assumed to be homogeneous with nonlinear behavior and has the following basic characteristics:
Elastic modulus = 23.6 GPa.
Poisson's ratio = 0.2
Unit weight = 24 KN/m^3

The foundation is assumed to be mass less, homogeneous with isotropic linear behavior and has the following basic characteristics:
Elastic modulus = 8 GPa.
Poisson's ratio = 0.3Equations

V. LOAD PROFILE AND REQUIRED MODIFICATION

As it was necessary to choose a suitable lateral loading pattern to apply on the structure, for this purpose a uniform lateral loading pattern is applied along the height of the structure. The magnitude of this uniform pressure is increased incrementally until the structure reaches collapse. An alternate approach would be to use a different lateral loading pattern which could be considered a suitable field of study for further research. By applying the aforementioned lateral load and increasing it incrementally, the total applied load in each step is plotted against the maximum lateral displacement of the nodes located on the upstream side of the dam's structure. Using this procedure the capacity curve of the dam is obtained.
In order to obtain the capacity curve of the structure in ADRS coordinate through equations 2 and 3, the following assumptions must be considered: The weight W used in this equation should take into account the effect of Hydrodynamic interaction; i.e. the interaction between the reservoir and the dam during an earthquake. This interaction has a significant effect on the earthquake response of the dam and must be considered in any dynamic analysis. Because the inertia force of a structure is a function of acceleration and mass, hydrodynamic interaction has a larger influence on thinner and less massive dams. In such structures the ratio of the water mass to the structure's mass is higher; hence the structural response will be more influenced by the water mass. In this study Westergaard's added mass formulation has been used to take this interaction into account.
The added-mass representation of dam-water interaction during earthquake ground shaking was first introduced by Westergaard (1933). In his analysis of a rigid 2-D gravity dam with a vertical upstream face, Westergaard showed that the hydrodynamic pressure exerted on the face of the dam due to the earthquake ground motion is equivalent to the inertia forces of a body of water attached to the dam and moving back and forth with the dam while the rest of reservoir water remains inactive. He suggested a parabolic shape for this body of water with a base width equal to 7/8 of the height, as shown in Fig. 3. [15]
A general form of the Westergaard added-mass concept which accounts for the 3D geometry (Clough 1977; Kuo 1982) can be applied to the earthquake analysis of arch dams. The general formulation is based on the same parabolic pressure distribution with depth used by Westergaard (Fig. 3), with the exception that it makes use of the fact that the normal hydrodynamic pressure P_n at any point on the curved surface of the dam is proportional to the total normal acceleration, $\alpha\ddot{u}_n^t$:

$$P_n = \alpha\ddot{u}_n^t \tag{6}$$
$$\alpha = \frac{7}{8}\rho_w\sqrt{H(H-Z)} \tag{7}$$

where ρ_w is the density of water, α is the Westergaard pressure coefficient, and H and Z are as defined in Fig. 3. The normal pressure P_n at each point is then converted to an equivalent normal hydrodynamic force by multiplying by the tributary area associated with that point. Finally, the normal hydrodynamic force is resolved to its Cartesian components, from which a full 3x3 added-mass matrix at each nodal point on the upstream face of the dam is obtained (Kuo 1982):

$$m_a = \alpha A\lambda^T\lambda \tag{8}$$

where A is the tributary surface area and λ^T is a vector of normal direction cosines for each point. Note that while the added-mass terms are coupled with respect to the nodal degrees-of-freedom, they are uncoupled with respect to individual nodes. Such a 3x3 full nodal added-mass matrix can easily be incorporated in a computer program using consistent mass matrix (non-diagonal), but it should be generalized for those programs that employ diagonal mass matrix. [15]

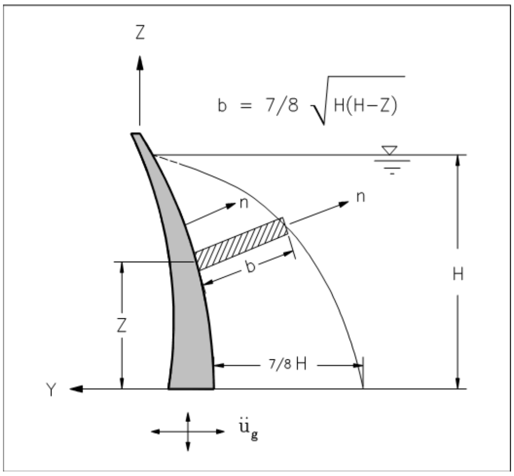

Fig. 3 Generalized Westergaard added hydrodynamic mass model for arch dams.

Based on the modal analysis of structure, the second mode is the dominant mode, therefore, when in Eq. [2] and [3], α_2 and PF_2 are used instead of α_1 and PF_1. The formers are modal mass coefficient and corresponding participation factors for the second natural mode of the structure.
Moreover, when using equations 4 and 5 assume that the weight of the structure is condensed and concentrated in the points along the height of the structure such that the weight of each point w_i, equals the half of the weight of the elements in the upper and lower rows contiguous to that row of nodes plus the added masses allocated in the nodes of that row.

ISBN: 978-988-18210-0-3
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)

VI. FINDING PERFORMANCE POINT

The Elcentro accelerogram that is shown in Fig. 4 is used to obtain the demand curve; afterwards the standard spectrum of this accelerogram was obtained using the Seismosignal program which along with eq.1 is converted to the ADRS format.

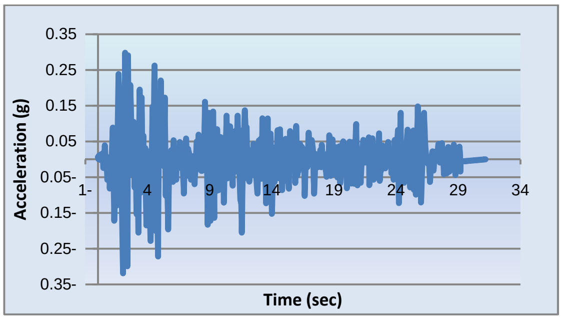

Fig. 4 Elcentro Accelerogram

Both capacity and demand curve were drawn in a coordinate system and the performance point of structure was obtained and shown in Fig. 5. Therefore, the base shear of the structure is obtainable and is equal to the vertical coordinates of this point. This value should be compared with the result of the time history analysis of the structure.

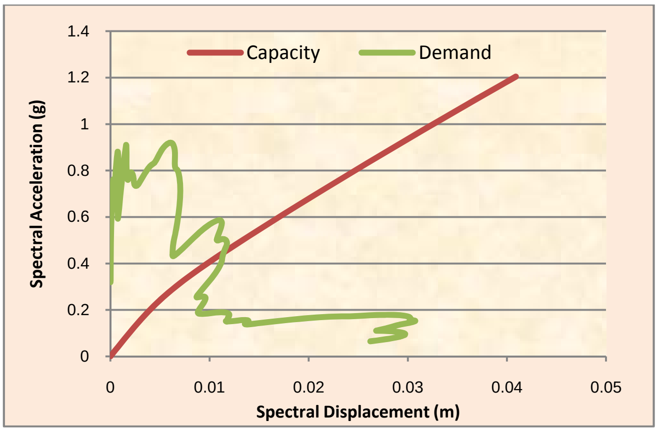

Fig. 5 Performance Point

According to the above diagram, it can be observed that the vertical coordinate related to the performance point is 0.4519. Multiplying this value by the structure's mass yields the earthquake force related to this spectrum, which is equal to 3.94034×10^{10} (N).

The same time history of acceleration has been used in the dynamic analysis and it is applied to boundary of the foundation mass. Moreover, to consider the effect of water interaction, the reservoir is also presented in the model.

The maximum base shear that is obtained from the dynamic analysis is equal to 4.4609×10^{10} (N) which is 52 percent of the structure's weight, whereas the base shear derived from using the method proposed in this paper is 45 percent of structure's weight. Therefore a %7 difference can be seen between the results obtained by using the time history and the proposed method.

VII. CONCLUSION

This paper has presented a simple computer-based method for push-over analysis of double-curved arch dams subject to equivalent-static earthquake loading. Results of this method have been compared with those of time history analysis of the Karun IV concrete arch dam. Overall, the main conclusions obtained by the present study may be explained as follows:

Considering the calculated error, it seems that this method can be improved by applying some changes so as to decrease the error and obtain the results that are much closer to the time history analysis results. For example if the contraction joints that exist in the dam's structure are modeled it may be possible to improve the result. Another option could be examining the effect of changing the loading profile that was used for the push-over analysis and find the most suitable profile applicable to this special form of structure. All in all, considering the obtained results it seems that the proposed method is a reliable tool to analyze double-curved arch dams and if improved can be considered a suitable alternative to the common methods for the analysis of special structures.

REFERENCES

[1] Earthquake Engineering for concrete dams: Design, Performance, and Research Needs, Panel on Earthquake Engineering for Concrete Dams, Committee on Earthquake Engineering, Division of Natural Hazard Mitigation, Commission on Earthquake and Technical Systems, National Research Council, NATIONAL ACADEMY PRESS, Washington. D. C.

[2] X. Du, B. Zhang, J. Tu, and Y. Zhang, "Earthquake Analysis of Arch dam-Foundation Rock Nonlinear Dynamic Interaction. Advances in Building Technology," Vol.1, 2002.

[3] S. Kucukarslan, "Dam-Reservoir Interaction Including the Reservoir Bottom Effects Time Domain," [Dig. 16th ASCE Engineering Mechanics Conference, University of Washington, Seattle, July 16-18, 2003]

[4] B. F. Chen. "Nonlinear hydrodynamic effects on concrete dam Engineering Structures," 18(3), pp. 201-212

[5] A. A. Sani, Vahid Lotfi, "Dynamic analysis of concrete arch dams by ideal-coupled modal approach, "Engineering Structures, 32(5), pp. 1377-1383, May 2010

[6] L. GAO, D. Jianguo, and H. Zh. Qiang. "Earthquake analysis of arch and gravity dams including the effects of foundation inhomogeneity," Higher Education Press and Springer-Verlag, 1(1), pp. 41-50, 2007

[7] Y. Calayir, A. A. Dumanoglu and A. Bayraktar, "Earthquake Analysis of Gravity Dam-Reservoir Systems Using The Eulerian and Lagrangian Approaches," 59(5), pp. 877-890, 1996

[8] L. GAO, D. Jianguo and H. Zh. Qiang, "Dynamic Dam-Reservoir interaction analysis including effect of Reservoir Boundary Absorption, Science in China Press, 2007

[9] P. J. Wen, Zh. C. Han, W. J. Ting and X. Yanjie, "Seismic Damage-Cracking Analysis of Arch Dams Using Different Earthquake Input Mechanics," Science in China Series E: Technologicl Sciences, 52(2), pp. 518-529, 2009

[10] V. Lotfi, R. Espandar, "Seismic Analysis of concrete arch dams by combined discrete crack and non-orthogonal smeared crack techniqe," Engineering Structure, Vol. 26, pp. 27-37, 2003

[11] X. Du, J. Tu, "Earthquake Analysis of Arch Dam with Joints Including Dam-Foundation Rock Nonlinear Dynamic Interaction," Advances in Building Technology, pp. 563-570, 2002

[12] N. Jianguo, Q. Kai, and X. Yan. "Push-over analysis of the seismic behavior of a concrete-filled rectangular tabular frame structure," Tsinghua science and technology, Volume 11, No. 1, pp. 124-130, February 2006

[13] Seismic Evaluation and Retrofit of Concrete Buildings, Volume 1, ATC-40 Report, Applied Technology Council, Redwood City, California, 1996

[14] *K. J. William* and E. P. *Warnke*, "Constitutive Model for the Triaxial Behavior of Concrete," Proceedings, International Association for Bridge and Structural Engineering, Vol. 19, ISMES, Bergamo, Italy, p. 174, 1975

[15] Engineering guidelines for the evaluation of hydropower projects, 1999

ISBN: 978-988-18210-0-3
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)