Nonlinear Dynamic Analysis of Concrete Arch Dam Considering Large Displacements

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ABSTRACT :

In this research, geometrically nonlinear dynamic analysis of arch concrete dam is attempted. At first, suitable models for large deformation analysis of massive plain concrete structures are investigated and by considering arch dam special features and properties, proper model for large displacement analysis is developed. A nonlinear analysis of the Morrow point arch dam using the Saint Venant–Kirchhoff model for large displacements is carried out under an intensive ground motion of order of 1.0g for the peak ground acceleration. Fluid-Structure interaction is modeled including water compressibility and reservoir bottom absorption although the foundation is considered as rigid. It was indicated that considering large deformation effects could reduces the displacement response of dam. This reduction of the peak response observed in this analysis was about 6 % in respect to that of the linear dynamic.
On the other hand, large deformation effects reduce compressive stresses and increases tensile ones. Values of these changes are about 9 % for maximum compressive and 6% for maximum tensile stresses for the same ground motion level. Although it could be understood that the structural behavior of an arch dam does not allow large strains in a general manner, but one could not rule out the appearance of large displacements, specially under joint-opening. Thus it is suggested that for large seismic loads a consistent inclusion of this type of nonlinearity is necessary in order to grasp a proper image of concrete arch dam dynamic behavior.

KEYWORDS: Arch Concrete Dam, Dynamic Analysis, Large Displacement , Saint-Venant-Kirchhoff Model , Fluid-Structure Interaction

1. INTRODUCTION

With respect to catastrophic effects of possible failure of concrete dam, continuous safety evaluation seems essential. Investigation on concrete dams has increased extensively during the recent decades. However, considering intrinsic complexity of arch dams, there are considerable ambiguities that require more attention. One of the existing concerns is geometrical nonlinear dynamic behavior analysis of arch dam considering large displacement subject to sever loading such as earthquakes at MCL. This is of great importance not only in view of theory but also in practical applications. Construction of new dams in high seismicity sites and even on the active faults are among the aforementioned issues examples include Shirvan dam (Iran), Klyde dam (New Zealand), and Steno dam (Greek). In all the above cases, actual behavior modeling of concrete arch dam requires geometrical nonlinear analysis [1,2].
In the present study, a suitable formulation to geometric nonlinear analysis of arch concrete dam is presented. This formulation is implemented in a new finite element code named as GFEAP(Generalized Finite Element Analysis Program). Using proposed model nonlinear dynamic analysis of Morrow Point arch dam subjected to intensified El-Centro earthquake record was carried out to show large deformation effects. Foundation was supposed rigid and soil structure interaction was ignored. Other sources nonlinearity such as joint opening, and concrete material nonlinearity were ignored. Only geometric nonlinearity was considered. Fluid Structure interaction was considered by numerical solution of Helmholtz equation with appropriate boundary conditions.
The earlier studies on nonlinear modeling of arch concrete dam categorized in three major group [2]: A)Joint nonlinearity modeling including construction joint , lift joint and Dam-Abutment joints. B)Concrete nonlinearity behavior modeling including cracking model , Plastic etc. models. C) Foundation nonlinearities including rock joint slippage modeling, and also jointed rock material behaviors models.
Apparently geometric nonlinear behavior of concrete dam was ignored in all the earlier analysis. In spite of the fact that geometric nonlinear analysis of shells and plates structures was carried out earlier [2], but to the authors' knowledge no large deformation analysis about concrete dams has been found in the literature [2].
Chung et al. [3] carried out nonlinear vibration analysis of geometrically shell structures. Large rotation and large displacement considered in the model. Total Lagrangian model was used in model with second Piola –Kirchhoff stresses and Green Lagrange strain criteria. Yuakim [4] carried out nonlinear analysis of tunnels in clayey/sandy soil with a concrete lining. Nonlinear behavior of soil and lining material, Large deformation effects and nonlinear behavior of contact between concrete lining and surrounding soil was considered in model. Swaddiwudhipong et al. [5] proposed a nine_node element for dynamic analysis of large strain elasto_plastic plate and shell structures using update Lagrangian description. Chin et al [6] presented a thin plate element for nonlinear analysis of thin -walled structures. Strain supposed small and update Lagrangian approach was used in model. Polak et al. [7] carried out nonlinear analysis of reinforced concrete shells. Both geometric and material nonlinearity considered in model. A rotating smeared crack model was used for concrete modeling. Geometric nonlinear model considered Lagrangian formulation and saint – venant –kirchoff model. Esmond et al. [8] presented geometric and material nonlinear analysis of reinforced concrete shell with edge beams. Concrete model developed based on nonlinear orthotropic elasticity. Saint-Venant_Kirchoff model and updated lagrangian model were used in model. Sathurappan et al.[9] presented nonlinear finite element analysis of reinforced concrete slabs. Both material and geometric nonlinear behavior considered in the model. Material nonlinear model considered plasticity in compression and cracking in tension. Total Lagrangian description and Saint Venant Kirchoff model were used in geometric nonlinear analysis. Roca,et al. [10] presented nonlinear analysis of reinforced concrete structure considering prestressing tendons. Updated Lagrangian model and two dimensional Darwin- Pecknold model were used in analysis. Bathe et al. [11].presented nonlinear analysis of concrete model used in ADINA .They recommended using of second Piola Kirchhoff stress and Green – Lagrange strain in geometric nonlinear analysis of concrete structures .
Accounting the hydrodynamic interaction is the other important issue in the dynamic analysis of dam-reservoir. Reservoir upstream radiation, and bottom partial absorption of acoustic waves, as well as water compressibility must be considered in modeling
In the upcoming sections of this paper, the basic concepts and the nonlinear models employed in the program are explained briefly at first. Then, some simple examples are considered to verify the developed program. Later on, the nonlinear dynamic behaviour of a typical thin arch dam is studied by the application of the models discussed.

2. BASIC CONCEPTS AND METHODOLOGY

2.1 Large deformation model

A large deformation model consists of three main parts: A) Mesh Description and Governing equation, B) Constitutive Relation and C) Equation Linearization and solution algorithm.

2.1.1Governing equation and Mesh Description

In general, four description to kinematics modeling in Geometric nonlinear analysis are presented in l technical itreature : Spatial or Eulerian Description, Material or Lagrangian Description, Arbitrary Eulerian-Lagrangian Description and Co-rotational Description.

In problems with large element distortation since element accuracy degrades with distortion, the magnitude of deformation that can be simulated with a Lagrangian mesh is limited. In Eulerian mesh, on the other hand, since finite element mesh is constant during analysis, there is no any distortion dependency in result. However in Eulerian mesh description, independency between material motions and mesh motion resulting to difficulty in boundary condition implementation. In ALE description node on boundary of initial mesh are constant during deformation and middle node move such as element distortion will be minimized. Indeed this description save all advantages of aforementioned approaches and avoided disadvantages of them. In Co-rotational Description initial configuration is two parts, stresses and strain calculated from rotated configuration and rigid body motion deduced from initial configuration.

In general Total Lagrangian description is used for modeling of quasi-continues problems. Update Lagrangian description is used to fluid like problem and large strain problem and co-rotational description is used to structural elements such as beam, shell and plate.

In concrete dam analyses, since small strain, element large distortion is not occurs and if initial finite element mesh well designed there is no anxiety about distortion and following precise reduction in results. Geometric nonlinear of arch concrete dam is a nonlinear problem wit large displacement and small strain. With respect to stage construction of arch concrete dam and concrete purities itself it expected that under sever earthquake loading , dam monoliths experienced large slip and large deformation caused by joint opening and failure with amount of crashing and plastic deformations. So Lagragian description is useful for our purpose. Finite element discritization with lagrangian mesh are commonly classified as Total Lagrangian and Update Lagrangian formulation. In the update Lagrangian formulation the derivatives are with respect to the spatial coordinates, the weak form involves integrals over the deformed (or current) configuration. In the total Lagrangian formulation, the weak form involves integrals over the initial (references) configuration and derivatives are taken with respect to the material coordinates. Each of these formulations can be advantageous for certain constitutive equation or loading by reducing the number of transformations which are needed. Indeed, if in numerical solution the appropriate constitutive tensors are employed, identical results are obtained. From mathematical view point two descriptions are identical and their equation can exchange with other easily. However update Lagrangian formulation is more effective than TL formulation computationally. It deduced from this fact that UL formulation has not need to calculations of initial stiffness in every iterations. Also in using UL formulation the artificial straining is not occurred. In this study every two approach implement in code. since the Saint-Venant kirchhoff model at references configuration is used , the update lagrangoian for more its transformations from initial configuration to current configuration has more computations

In a nonlinear analysis, the equilibrium of the body considered must be established in the current (deformed) configuration. also it is necessary to employ an incremental formulation to confidentially describe the loading and the motion of the body .Also a suitable constitutive model is needed .

In our Lagrangian incremental analysis approach we express the equilibrium of the body at time t+Δt using the principal of virtual displacement. Using tensor notation this principle requires that [13]:

$$\int_{{}^{t+\Delta t}V} {}^{t+\Delta t}t_{ij}\, d\,{}_{t+\Delta t}e_{ij}\, d\,{}^{t+\Delta t}V = {}^{t+\Delta t}R \tag{2.1}$$

where :

${}^{t+\Delta t}t_{ij}$ =Cartesian component of the Cauchy stress tensor , $d\,{}_{t+\Delta t}e_{ij}$ =Strain tensor corresponding to virtual displacements , ${}^{t+\Delta t}V$ = Volume at time $t+\Delta t$, ${}^{t+\Delta t}\mathrm{R}$ = External forces vector

A fundamental difficulty in the general application of (2.1) is that the configuration of the body at time $t+\Delta t$ in unknown. This is an important difference compared with linear analysis in which it is assumed that the displacement is infinitesimally small so that in (2.1) the original configuration is used. Also continues changes in the configuration of the body entail some important consequence for development of an incremental analysis procedure. For example, Cauchy stress and engineering strain are not valid any longer, because their components are not "objective" and may change due to pure rigid body motions of structure (or element). One must find the stress and strain measures that remain objective in large deformation analysis .In tables 1, 2 the useful strain and stress criteria for use in large deformation analysis is presented. With respect to present study requirements The second Piola-Kirchoff stress measure and Green Lagrange strain measure are used in TL and UL formulations.

Table 1 Different strain Measures for Large deformation analysis

Strain Measure	**Configuration**	**Approach**	**Conjugated Stress**	**Objectivity**
Green-Lagrange	Reference Configuration	UL, TL	Second Piola-Kirchhoff stress	Yes
Almansi	Current Configuration	UL	Cauchy stress	No
Henkey	Current Configuration	UL	Cauchy stress	Yes
Deformation Rate	Current Configuration	UL	Cauchy stress	No

Table 2 Different stress Measures for Large deformation analysis

Stress Measure	**Symmetry**	**Configuration**	**Approach**	**objectivity**	**Conjugated Strain**	**Physical Interpretation**
Cauchy	Symmetric	Current Configuration	UL	NO	Almansi ,Green-Lagrange, Henkey Deformation Rate	Physical
First Piola-Kirchhoff	No Symmetric	Reference Configuration	UL, TL	Yes	Green-Lagrange	No Physical
Second Piola-Kirchhoff	Symmetric	Reference Configuration	UL, TL	Yes	Green-Lagrange	No Physical
Kirchhoff	Symmetric	Current Configuration	UL	No	Almansi, Henkey, Deformation Rate	Weighted Cuachy

2.1.2 Constitutive model

Constitutive models are commonly classified as two major group, Hyperelestic and Hypoelastic models. Also there are some minor models such as Mooney –Rivilin , Neo-Hookean , Chauchy elastic model ,etc. Hypoelastic material laws relate the rate of stress to the rate of deformation. Hyperelastic models have some necessary requirements of a elastic model but not all of them. For example, for large deformation, energy is not necessarily conserved and the work done in closed deformation path is not necessarily zero. Hypoelastic laws are used primarily for representing the elastic response in phenomenological elasto-plastic laws where the elastic deformation are small, and dissipative effects are also small [12]. Hrperelastic materials, on the other hand, are elastic materials for which the work is independent of the load path. Hyperelastic materials are characterized by the existence of a stored (or strain) energy function that that serves as a potential function for the stress. Hyperelastic models classified as some subgroups itself. One of those models is Saint-Venent_Kirchhof model. This model is a suitable model for analysis involved large displacement and small strain motions.

Many engineering applications involve small strains and large rotations. The response of the material may then be modeled by a simple extension of the linear elastic laws by replacing the engineering stress by the second Piola-Kirchhof stress (see table 2) and linear (engineering) strain by the Green –Lagrange strain (see table 1). This model is :

$$S = CE \tag{2.2}$$

where :

C =fourth order tensor of elastic moduli, S = Second Piola-Kirchhoff stress tensor, E = Green-Lagrange Strain tensor

An important characteristic of Saint-Venant-Kirchhoff model is that in large displacement and large rotation but small strain analysis the relation in (2.2) provides a natural material description because the components of second piola-kirchhoff stress and Green-Lagrange strain tensors follow the objectivity criteria. This observation implies that any material description which has been developed for infinitesimal displacement analysis using engineering stress and strain measures can directly be employed in large displacement and large rotation but small strain analysis

provided second Piola-Kirchhoff stresses and Green-Lagrange strain are used. A practical consequence is, for example, that elasto plastic and crack models can be directly employed for large displacement, large rotation, and small strain analysis by use of Saint-Venant Kirchhoff material model. Considering that concrete material can not experienced large strain and maximum uni-axial strain of concrete is a order of 0.003, so this model, is a suitable model for concrete dam modeling in large deformation analysis.
Equation (2.2) in Total Lagrangian Approach is written as[12] :

$$ {}^t_0S_{ij} = {}^t_0C_{ijrs}\, e_{rs} \qquad (2.3)$$

That is used in UL Approach again. However in calculating of internal force Second Piola –Kirchooff stress and constitutive tensor are transformed as :

$$ {}^t_tC = \frac{{}^t r}{{}^0 r}\, {}^t_0X^T\, {}^t_0X^T\, {}^t_0C\, {}^t_0X\, {}^t_0X \quad , \qquad {}^t t = \frac{{}^t r}{{}^0 r}\, {}^t_0X\, {}^t_0S\, {}^t_0X^T \qquad (2.4)$$

Were :
${}^t_0X$ = Deformation gradient , ${}^t r$, ${}^0 r$ = Initial and current density respectively , ${}^t t$ = Cauchy stress tensor

In addition of Saint-Venant-Kirchhoff model, to comparison deduced results with commercial software and code verification, Neo- Hookean model is used in prepared code also.

$$ \Psi(C) = \tfrac{1}{2}\, l_0\, (LnJ)^2 - m_0\, Ln\, J + \tfrac{1}{2}\, m_0\, (trace\ C - 3) \qquad (2.5)$$

Were :
$\Psi(C)$: Potential function, l_0 , m_0 : Lame's constants , J: Deformation gradient determinant , C: the right Green deformation tensor

2.1.3 Equation linearization

Using the principal of virtual work we express the equilibrium of the body at deformed configuration. This principal requires that [13] :

$$ {}^{t+\Delta t}R - {}^{t+\Delta t}F = 0 \qquad (2.6)$$

Were :
${}^{t+\Delta t}R$ = Vector of external nodal forces at time $t+\Delta t$ (includes weight , hydrostatic and dynamic earthquake load)
${}^{t+\Delta t}F$ = Vector of internal nodal forces at time $t+\Delta t$ related to internal stresses.
Integrals of R, F calculated at current configuration. A fundamental difficulty in the general application of (2.1) is that the configuration of the body at time $t+\Delta t$ is unknown and custom stress and strain measures are not objective. So using objective measure of stresses and strain we can convert integrals in (2.1) from unknown current configuration to initial or any previously calculated configuration. This requires that [13] :

$$ \int_{{}^{t+\Delta t}v} {}^{t+\Delta t}t_{kl}\; d_{t+\Delta t}e_{kl}\; d\,{}^{t+\Delta t}v = \int_{{}^t v} {}^t_t s_{ij}\; d\,{}^t_t e_{ij}\; d\,{}^t v \qquad (2.7)$$

Were :
S_{ij}= Second Piola Kirchhoff stresses , e_{ij} = Green Lagrange strain , t_{kl} = Cauchy stresses , t = arbitrary time
In derivation of equation (2.7) we use the second Piola Kirchhoff stresses and Green Lagrange strain measure. Equation of (2.6) however is nonlinear and need to linearization .The Linearized discrete dynamic form of equations of motion about the state at time t in the UL and TL formulations expressed as [13]:

$$ M\,{}^{t+\Delta t}\ddot{U} + {}^{t+\Delta t}_0C\,\dot{U} + ({}^t_0K^{Mat} + {}^t_0K^{Geo})U = {}^{t+\Delta t}R - {}^t_0F \quad \text{in TL} \qquad (2.8)$$

$$ M\,{}^{t+\Delta t}\ddot{U} + {}^{t+\Delta t}_tC\,\dot{U} + ({}^t_tK^{Mat} + {}^t_tK^{Geo})U = {}^{t+\Delta t}R - {}^t_tF \quad \text{in UL} \qquad (2.9)$$

Where M , ${}^tK^{Ma}$, ${}^tK^{Geo}$ are Geometric stiffness matrixes, Material stiffness matrix, Mass matrix, Damping matrix respectively. $\ddot{U}, \dot{U}, U$ also are acceleration, velocity and displacement vectors respectively. F and R are defined earlier.

The linearized form of motion equation can solve by Newton-Raphson family method or other methods incrementally.

3. FLUID STRUCTURE INTERACTION

The governing equation for fluid domain is the Helmholtz equation for hydrodynamic pressure:

$$\nabla^2 P = \frac{1}{C^2}\ddot{P} \tag{3.1}$$

Where p is the hydrodynamic pressure, and C, the acoustic wave velocity in water. The above equation implies small displacements of inviscid compressible fluid with irrotational motion. Water compressibility has a significant influence on the fluid-structure interaction for a wide range of ratio of natural frequencies of structure to fluid domain, including the case of higher and stiffer dams [9]. Thus, for general applicability and completeness of the dam-reservoir formulation, one needs to include the reservoir water compressibility.
Boundary conditions are expressed as :

$$\left[\frac{\partial P}{\partial y} + \frac{1}{g}\ddot{P}\right]_{y=\eta} = 0 \tag{3.2}$$

That is called Cauchy Boundary Condition for the reservoir-free surface,

$$\frac{\partial P}{\partial n} = -r\ddot{a}_{gn} - \frac{1}{bC}\frac{\partial P}{\partial t} \tag{3.3}$$

for the reservoir bottom partial absorption and normal component of earthquake records

$$\frac{\partial P}{\partial x} = -\frac{p}{2h}P - \frac{1}{C}\frac{\partial P}{\partial t} \tag{3.4}$$

for the reservoir upstream face radiation of acoustic waves, and

$$r\,\vec{a}_{ns} = -\frac{\partial P}{\partial n} \tag{3.5}$$

For the interaction boundary between dam and reservoir. In the above equations, z is the vertical coordinate, b, the acoustic impedance ratio of rock to water, n, the vector perpendicular to the boundary, r, the mass density of water, g, the gravitational acceleration, and $\vec{a}_{ns}$, the acceleration of dam upstream face in the normal direction. Here, we have assumed that the hydrodynamic waves satisfy the 1-D wave propagation equation (3.4), through the upstream reservoir near-field truncation surface. If we ignore first term at right hand of equation (3.4) this boundary, sometimes known as the Sommerfeld or viscous boundary, performs well in time domain analysis when applied sufficiently far from the structure. The above equations along with the governing equation for the structure would lead to a simultaneous differential equations set for the coupled dam –reservoir System. These equations are discretized by the finite element method in a standard way similar to that of Ref. [2]. To avoid prohibitively high number of nonsymmetrical equations with large bandwidth, the staggering solution method [2] is employed. Here, the displacement and the pressure fields are solved alternatively in each time step to achieve ``inter-domain compatibility'' or convergence.

3. COMPUTER IMPLEMENTATION AND VERIFICATION EXAMPLES

3.1 Computer Implementation

The proposed models have been implemented at finite element code GFEAP (Generalized Finite Element Code Program). GFEAP have capabilities of time history nonlinear dynamic analysis of arch dam, considering material, geometrical and construction joint nonlinearity and fluid structure interaction. It was prepared in Tarbiat Modares university by writers for complete nonlinear dynamic analysis of gravity and arch concrete dams

3.2 Preliminary example

The validity of the proposed models and numerical algorithms has been checked using the available numerical results. At the first step, large deformation analysis of a shallow arch has been tested. The second model is a geometrical static and dynamic analysis of a plate.

3.1.1. Large displacement analysis of a shallow arch [14]

It is a shallow arch subjected a concentrated load at its centre as shown in fig 1. Its geometry and mechanical properties are presented at fig 1. Twenty 20-node elements are used in modeling. In the fig 2 the analysis results are compared with result of Ref. [14]. As shown in fig 2, the deduced results have excellent compatibility with result of ADINA and Mallet, Bereke [14].

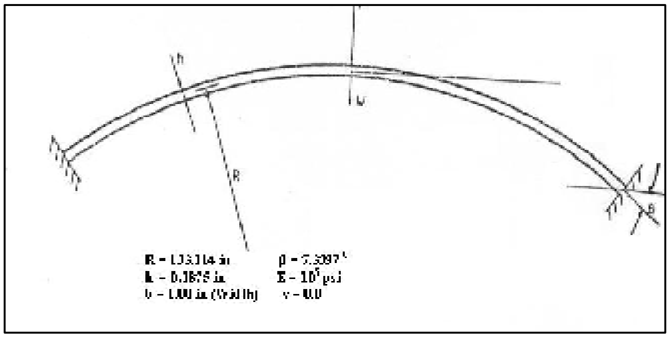


Figure 1 Large deformation analysis of shallow arch

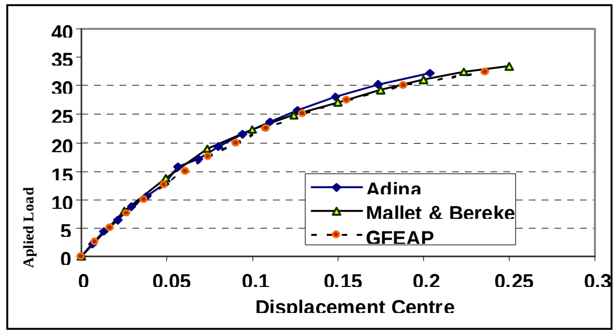


Figure 2 Load deflection curve

3.1.2.Dynamic Large displacement analysis of a simply supported plate [14]

This model is a square simple supported plate subjected to uniform pressure. Its geometry and mechanical properties are presented at fig 3 . One hundred 20-node elements are used in modeling. Neohookean model used as constitutive relation. For comparison similar analysis carried out using ANSYS software. The computed centre deflection as a function of the load is shown in fig 4. As shown in fig 4 the GFEAP's results have good compatibility with result of ANSYS. To control of dynamic part of code, time history analysis of the plate subject three components of Manjil earthquake carried out using ANSYS and GFEAPS. Density of plate was supposed as 2.4 kg/m^3. In fig 5. The deduced results from tow model are presented.

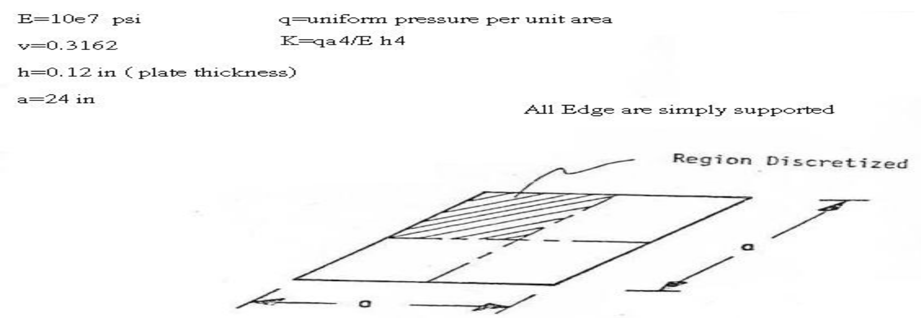


Figure 3 Large deformation analysis of simple supported beam

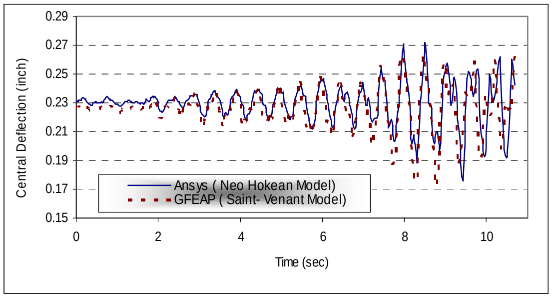


Figure 5 Displacement time history of plate

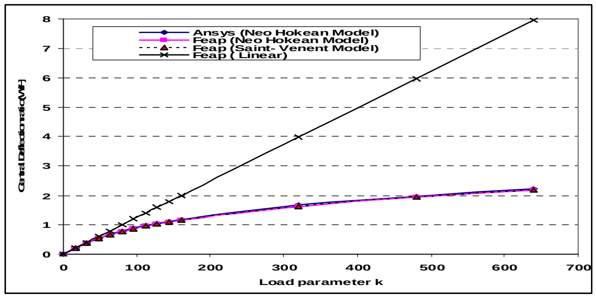


Figure 4 Load deflection curve of plate

3.1.3. Large displacement analysis of a Shallow Shell [14]

This test is a shallow spherical shell subjected to concentrated load in centre of arch. Its properties are presented at fig 6 . It is a famous patch test in large deformation analysis of shells and plates. Thirty six 20 node elements are used in modeling. The results are compared with result of ref [14]. The centre deflection of shell versus of applied load is captured in fig7. As shown in fig 7 computed results has excellent compatibility with results of Ref[14].

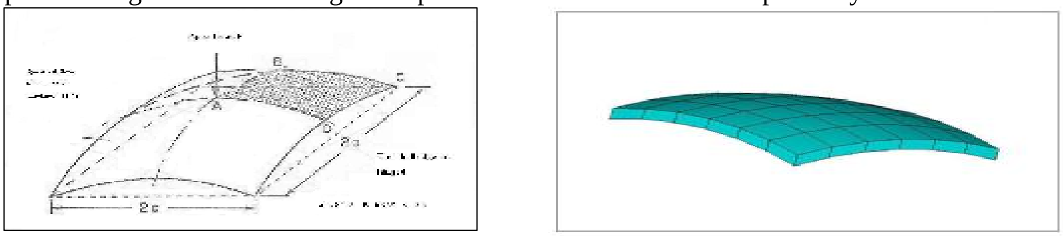


Figure 6 Shallow spherical shell (Geometry and properties and Finite element mesh)

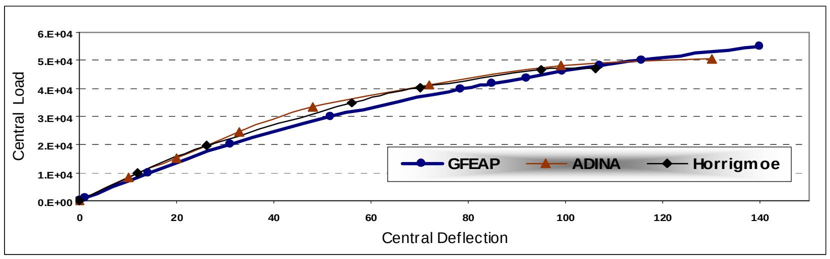


Figure 7 Load deflection curve

4-APPLICATION ON ARCH CONCRETE DAM

In this section, the nonlinear behavior of Morrow Point arch dam is being studied by application of the models discussed above. The dam is 145 m height; with the width of valley at crown elevation is 184 m on the Gunnison River in Colorado, USA. This dam is approximately symmetric, single cantered arch dam. Figure 8 shows the considered system which includes the finite element model of dam body and the reservoir in which the length of reservoir in the upstream direction is about nearly two times the height of dam. Moreover, the dam-foundation interaction is neglected. Sixteen 20-node solid element and 180 20-node fluid elements are used to modeling of dam body and reservoir domain respectively.

The module of elasticity, poison ratio, density are 27 GP , 0.2 , 2483 Kg/m^3 respectively . Considered internal viscous damping ratio is 0.05 for first and fifth vibration modes. Water level elevation for both hydrostatic and hydrodynamic pressure calculations is equal to the dam crest elevation (141.73 m). Acoustic wave velocity in water is C, 1440.0 m/s. The acoustic impedance ratio of rock to water is, $b = 3.444$.The ground motion recorded at Taft Lincoln School during the Kern County, California earthquake of 21 July 1952 is selected as the free-Field ground acceleration (Fig. 9). The records are scaled to 1 g.

The loads applied on the system are self weight, hydrostatic pressure and seismic load. The standard Newmark method is used to integration of dynamic equation in time domain. The Newmark parameters of a , b were assumed as 0.5, 0.25 respectively. The time integration step was 0.01. The 14 point integration rule is used in numerical integration. It was found that this integration scheme compared to costume 3*3*3 integration rule is economical [15].The modified Newton Raphson nonlinear solution algorithm is used. Therefore, the structural stiffness matrix has to be updated only in beginning of every time steps. Convergence tolerance for nonlinear displacement iterations is based on the energy norms defined as $\frac{|E(i)|}{|E(1)|} = 1*10^{-12}$. For pressure iterations in the staggering scheme, convergence is based on the pressure norm as $\frac{|\Delta P|}{|P|} = 0.001$. Maximum number of iterations for pressure is 8, and for displacement is 10.

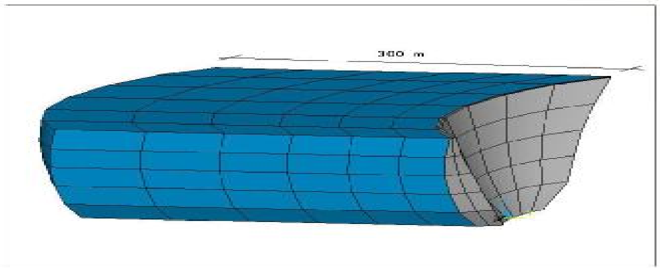
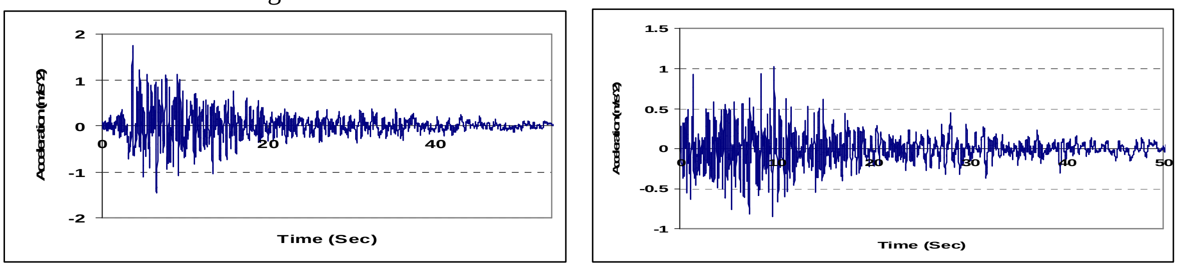


Figure 8 Dam-reservoir finite element model of Morrow Point arch dam.



Horizontal Component *Vertical Component*

Figure 9 Original ground motion of Taft Lincoln California Earthquake of 21 July 1952.

Large displacement analysis carried out using Saint-Venant Kirchhoff model. It implies small strain hypothesis. The history of displacement in the stream direction of nodal point in the middle of dam crest for linear and nonlinear analysis is shown in Figs. 10 The comparison denotes that the results of the two analyses are close to each other. As shown in Fig.10 maximum displacement is 6,5 cm for earthquake record scaled to 1g . The difference between linear and nonlinear analysis is % 5 for maximum crest displacement. However considering large displacement nonlinearity caused to reduction in displacement response of arch dam.

In Figs. 11, and 12, the time histories of the first principal stresses for the Gauss point 1 of elements 15(in the middle of dam crest) and 7 (in the middle of the dam bottom) are presented. As shown in these Figures, there are not noticeable differences between the linear and nonlinear analyses. Only by calculations it is shown that the maximum tension stresses is somewhat increased but the compressive maximum stress decreased moderately.

In general it could be concluded the for the Morrow Point arch dam large displacement analysis has not consider effects on the results. One might have to include foundation interaction effects as well as the dam body material nonlinearities and vertical joint-opening behaviors to have a better assessment of this phenomenon.

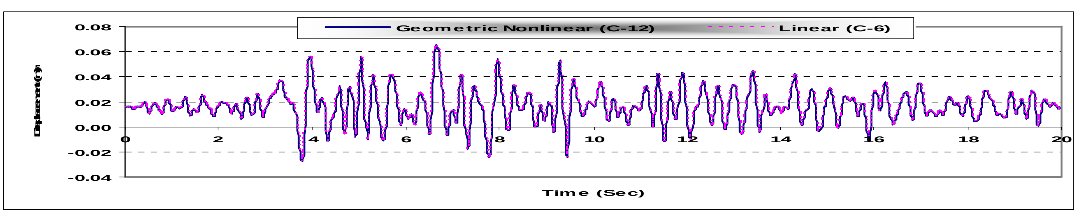


Figure 10 Time history of displacement in stream direction in the middle of dam crest

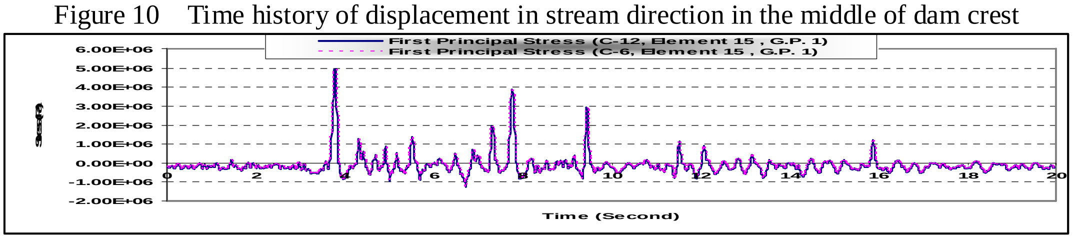


Figure 11 Time history of the first principal stress for the Gauss point 1 of element 15 located in the middle of dam crest

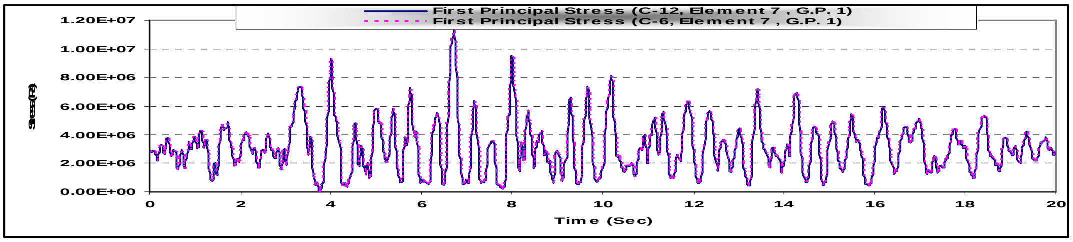


Figure 12 Time history of the first principal stress for the Integration point 1 of element 7 located in the middle of dam bottom ,upstream face

5-CONCLUSION

- In the present study a methodology to nonlinear analysis of arch concrete dam considering large deformation is presented.
- The proposed geometrical nonlinear dynamic analysis methods applied to structural elements such as beams, shells and plates using three-dimensional elements proved excellent results.
- After suitable models for the large displacement analysis of massive plain concrete structures are investigated, the arch dam special features and proper model for large displacement analysis are developed. Thereafter, a nonlinear analysis of the Morrow point arch dam using the Saint-Venant–Kirchhoff model for large displacements is carried out under an intensive ground motion of order of 1.0g. Fluid-Structure interaction, water compressibility and reservoir bottom absorption are included. The foundation is considered as rigid. It is indicated that considering large deformation effects reduces the displacement response of dam. This reduction of the peak response is about 6 % in respect to that of the linear dynamic. On the other hand, large deformation effects reduce maximum compressive stresses and increases maximum tensile ones. Values of these changes are about 9 % for compression and 6% for tension for the same ground motion level
- In addition of earthquake loading, concrete arch dams could experience large deformations due to other loadings such as abutment instability in which the dam can slide and experienced large displacements. The Malpasset dam failure is a good example. The GFEAP program has been successfully employed for such events although not shown here...

However, In the present study, foundation is not modeled. Considering foundation flexibility can motivate geometric nonlinear behavior of arch dam and highlight the large deformation effects. Also the dam was supposed as a continuous body. Considering the vertical construction joints opening and concrete material nonlinearities may also result in higher differences between the small and the large displacements analyses results. The above conclusions are deduced from the analysis of the Morrow Point arch dam. For a generalization one needs further investigation on other dams with various dimension and material properties.

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