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Structural Integrity of PHWR Reinforced Concrete Containment under Seismic Excitation

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ABSTRACT: In this paper, nonlinear transient analysis is carried out to check a structural integrity of an outer reinforced concrete containment structure of a nuclear power plant subjected to an earthquake ground motion. The large transient dynamic analysis is carried out in a supercomputer environment involving thousands of time steps, efficient numerical technique and FEM idealisation within reasonable time. In present work, the containment structure is idealised using 3D twenty noded brick elements. The elasto visco-plastic model is employed to predict concrete cracking, concrete crushing and steel yielding for obtaining the seismic response of the containment shell.

1. INTRODUCTION

The outer reinforced concrete containment of a Nuclear Power Plant (NPP) is required to be designed for extraordinary dynamics loads such as extreme accidental internal loads (e.g. jet impingement forces, thermal and pressure forces due to bursting pipes); and external catastrophic loads (e.g. earthquake, tornado, blast and aircraft impact forces). The damages caused to the structure by these loads may follow a number of hypothetical abnormal events. Though the occurrence of these loads has low probability, adequate measure should to be taken for the containment integrity especially against seismic loading in the limit state design. Under seismic excitation, the behaviour is more complex, as it depends on coupling between excitation frequency and the modified dynamic characteristics of the containment structure. To establish a reliable method of designing reinforced concrete containment, a higher degree of accuracy is required to predict the actual response of structures for the seismic loading as compared to other conventional loads.

Mostly, the research on dynamic analysis of RC structures is concerned with frames, columns and other structural elements. Complicated RC structures such as nuclear containment building are usually simplified as frames or columns with lumped masses, because, it is easier to find the restoring forces of those structural elements and to solve this kind of structural system mathematically. Up to now, few studies have been made on the in-plane nonlinear dynamic response of RC structures like shear walls and containment structures by using the finite element method with three dimensional solid elements, which can represent the properties of actual structures better than simplified structural members with lumped mass. This paper deals with the general finite element method established for static analysis of RC structures, extended to predict the dynamic response of the containment structure of a NPP. In this work, three dimensional transient dynamic finite element method using the 20-noded solid element is adopted to encapsulate the local and global failure modes due to seismic loading in a realistic manner. The probabilistic aspect of the problem has not been covered in the present work.

2. NONLINEAR TIME DEPENDENT FINITE ELEMENT ANALYSIS

A very high degree of accuracy is needed to predict local damage and global nonlinear structural behaviour of nuclear containment subjected to seismic loading. The twenty noded isoparametric (quadratic serendipity) brick element shown in Fig. 1 has been used in the analysis. The concrete and reinforcement are represented with a single element [Zienkiewicz *et. al.* (1988)] for finite element analysis in the present study. Perfect bond is assumed between the surrounding concrete and the reinforcement steel. Each set of reinforcing bars is assumed to be a two dimensional membrane of equivalent 'layer' thickness (Fig. 2) resisting only axial stresses. For numerical integration the fifteen Gauss point integration rule is used [Irons (1971)].

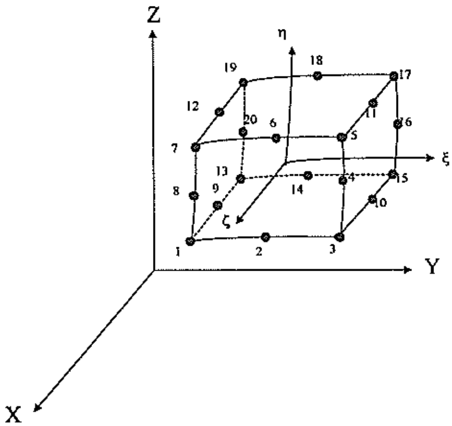


Fig. 1 Twenty noded isoparametric element

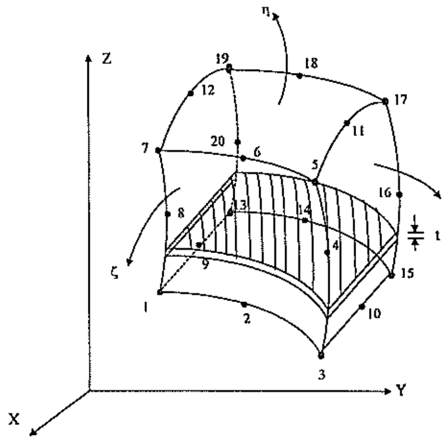


Fig. 2 3D element with smeared reinforcement

2.1 Dynamic equations of motion

Dynamic analysis of structures exhibiting nonlinear behaviour is performed by using direct integration, to trace the response in the time domain. The nonlinear dynamic equilibrium equation can be written as

$$M\ddot{d}_n + C\dot{d}_n + p_n = f_n \tag{1}$$

where M and C are the global mass and damping matrices respectively, p_n is the global vector of internal resisting nodal forces and f_n is consistent nodal forces for the applied body and surfaces traction forces grouped together, $d_n, \dot{d}_n$ and $\ddot{d}_n$ are the global nodal vectors for displacements, velocities and accelerations. The mass matrix is lumped by scaling the diagonal term of the consistent mass matrix so that total mass is preserved. The Rayleigh damping is used since it leads to the computationally attractive banded structure the same as the stiffness matrix.

When the structure is subjected to seismic excitation, the external applied body forces is defined by

$$f_n = -MI\ddot{u}_g(t) \tag{2}$$

where $\ddot{u}_g(t)$ is a ground acceleration and I is a vector indicating the direction of the earthquake excitation.

The discrete equations governing the nonlinear behaviour of the structure can be derived from the principle of virtual displacement using the finite element method. The resulting nonlinear equations of equilibrium at an instant of time t can be written as

$$\psi(d) = f_n - p_n = 0 \tag{3}$$

where, $\psi(d)$ is the residual force vector; an incremental solution procedure is adopted, in order to trace the nonlinear nature of the structure using Eq. 3. No check is performed on the global equilibrium due to unreliable pure incremental method. At the end of the equilibrium its performance is improved when iteration is performed for balancing the residual forces. During the typical load increment, the linearised equations to be solve for each iteration (say i), have the form

$$K_i \delta d_i = \psi_i \tag{4}$$

where, δd_i is the incremental nodal displacement during the i^{th} iteration and K_i is the tangential stiffness matrix in the i^{th} iteration

2.2 Constitutive model for concrete and reinforcement

The constitutive material model employed here is a strain rate sensitive elastic visco-plastic model suitable for transient analysis. In this two surface model, the failure and yield surface is represented by a five parameter model. The five parameter model is a non-circular as well as non-afine on a deviatoric plane. The curve meridians on the deviatoric plane are described by the cubic and non circular trace is represented by an ellipse.

The tensile cracking is considered by adopting a smeared crack approach with fixed angle crack. The initiation of cracks relies on the maximum principal strain. The closing and reopening is allowed following the secant path. A reduced value of shear modulus is adopted for the shear transfer across a crack due to aggregate interlock and dowel action. The accumulated damage caused by degradation of material strength is also considered.

3. CONCRETE CONTAINMENT STRUCTURE UNDER SEISMIC EXCITATION

The structural integrity of a reinforced concrete containment of NPP has been analysed for the actual acceleration time histories of Koyna (1967) earthquake (longitudinal component) and El-Centro (1940) earthquake (S00E component) and as well as for Johnson and Epstein (1976) sine sweep analytic earthquake. These accelerograms (Fig. 3) are used as a prescribed horizontal acceleration history. The reactions from the inertial forces i.e. $f_n = -MI\ddot{u}_g(t)$ for the analysis are calculated for each time step using Eq. 1. The displacement configuration of the structure at the previous time step has been considered in the evaluation of the last term of the equation.

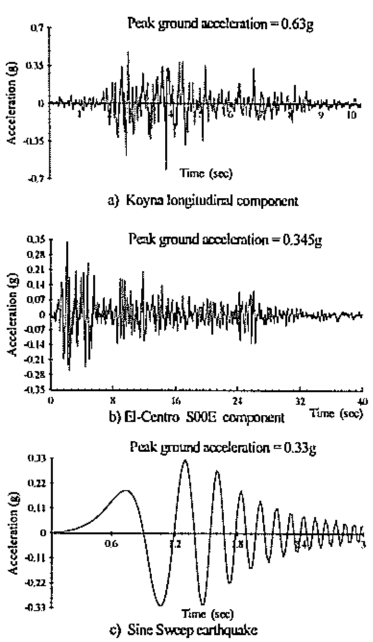
The soil structure interaction has been neglected and the base of the containment shell is assumed fixed at the foundation level. The displacement configuration of the structure at the previous time step has been considered in the evaluation of the last term of the equation. The structure is considered failed when 1) a sufficient number of sampling (Gauss) points in a subelement have cracked and 2) a large iterative displacement is caused by the increment in residual forces and dissipated energy. The model is not able to predict the penetration, scabbing or perforation.

Table 1 Material properties of reinforced concrete

S.No.	Property		
1.	Young modulus of concrete (MPa)	(E_c)	200,00
2.	Young modulus of steel (MPa)	(E_s)	200,000
3.	Poisson's ratio	(ν)	0.17
4.	Yield stress of steel (MPa)	(f_y)	460
5.	Crushing strength (MPa)	(f_c')	35
6.	Crushing strain	(ε_{cu})	0.0035
7.	Mass density (Kg/m^3)	(ρ)	2400
8.	Fracture energy (Kg/m)	(G_f)	10
9.	Fluidity parameters of concrete	(a_0')	0.3055
		(a_1')	0.76
10.	Fluidity parameters of steel	(a_0')	1.53
		(a_1')	0.97
11.	Factor for end elasticity	(α)	0.4
12.	Biaxial strength (MPa)	(f_{bc}')	40.6
13.	Ultimate tensile strain (μ)	(ε_{tu})	180
14.	High compression along	(σ_{m1})	128
	$\theta = 0^\circ$(MPa)	(τ_{m1})	20.6
15.	High compression along	(σ_{m2})	126
	$\theta = 60^\circ$(MPa)	(τ_{m2})	26.6
16.	Strain softening parameter	(α_c)	10
17.	Failure surface function	(β_0)	1.84
		(β_1)	1.09

3.1 Geometry and material properties

An outer containment shell used for this paper has been earlier analysed by Abbas (1992) for missile impact. The geometry of the concrete containment shell and corresponding mesh discretisation is shown in Fig. 4. The 4% reinforcement is considered in the containment. The material properties of the containment structure used for the analysis are given in Table 1. For the analysis, one half of the containment shell structure is discretised into fifty two isoparametric brick elements with a total of 427 nodes leading to 1095 degrees of freedom. The containment is analysed for three different configurations. (i) The containment cylindrical wall (1.2m to 0.61m) and dome (0.61m to 0.20m) with varying thickness (ii) 1.2cm cylinder wall thickness and 0.61m dome thickness (iii) 1.2m cylinder wall and 1.2m dome thickness. Each configuration is analysed for all three components of earthquake acceleration, considered independently to observe the seismic behaviour.



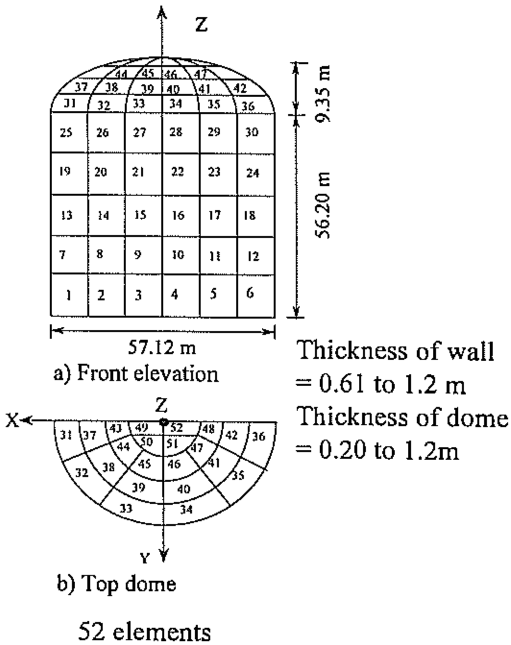


Fig. 3 Acceleration time histories

Fig. 4 Geometry and mesh discritisation

3.2 Behaviour of containment shell

The fundamental time period for the containment configuration (i) is 0.15 sec, for (ii) is 0.2 sec and for (iii) is 0.24 sec. A global structural damping of 1% is considered to be realistic, as the constitutive material model includes some energy dissipative effects. In the following section behaviour due to all three acceleration time histories is discussed. Linear analysis is carried out for each case with gravity loads.

The containment configuration for case (i) is found to be safe as no nonlinear behaviour is observed when subjected to all three acceleration time histories.

3.2.1 Containment configuration (ii) subjected to earthquake ground motions

No nonlinear behaviour is observed when subjected to El-Centro S00E component. When containment is subjected to sine sweep earthquake a few Gauss points cracked at the bottom of the cylinder (46) and top of the dome (13). Cracking starts around 2.0 seconds of the sine sweep earthquake and stabilises after 2.30 seconds. The containment is subjected to Koyna longitudinal component and cracking starts at 2.8th seconds of the earthquake. The cracking stabilises after the 5.6th seconds of the analysis. In containment 184 Gauss points are cracked in cylindrical portion and 141 Gauss points are cracked in the dome. No nonlinear behaviour is observed for 5% damping.

3.2.2 Containment configuration (iii) subjected to earthquake ground motions

No nonlinear behaviour is observed when the containment is subjected to the El-Centro S00E component. When the containment is subjected to the sine sweep earthquake cracking starts around 1.65th seconds of the earthquake and stabilises after 2.30 seconds. The Gauss points cracked are at the bottom of the cylinder (113) and top of the dome (32). In the containment subjected to Koyna longitudinal component cracking starts at 2.57th seconds of the earthquake. No further cracking is observed after 5.6th seconds. In the containment 206 gauss points are cracked in cylindrical portion and 20 gauss points are cracked in the dome. No nonlinear behaviour is observed for 5% damping.

The deflected shape and propagation of crack in the time domain for the sine sweep earthquake and Koyna longitudinal component are shown in Fig. 5 and Fig. 6 respectively. The cracking is observed in the crown of the dome due to the larger diameter of the cylindrical shell. The period of vibration is elongated for containment configuration (iii) as compared to configuration (ii) and some dissipation due to nonlinear effects are evident. High frequencies are clearly observed for configuration (iii), especially in the later stage of the analysis, when the force is reduced and the structure is vibrating almost freely. The nonlinear horizontal displacement history of monitored nodal point B is shown in Fig. 7. The peak stresses for containment configuration (iii) (wall and dome of 1.2 m constant thickness) when subjected to Koyna longitudinal component is shown in Fig. 8. It has been observed that the steel has not yielded for any of the prescribed horizontal acceleration histories.

4. CONCLUSIONS

The study presented in this paper shows that the material and analytical models employed, as well as analysis procedure, are capable of tracing the non-linear response of a three dimensional reinforced concrete structure under simulated or earthquake induced ground motions. The model is able to predict cracking, yielding and the load displacement curves. The containment with segmental dome or dome with lesser thickness than that of cylindrical wall is able to resist higher seismic forces. Nonlinear effects substantially modify the seismic response of a reinforced concrete structure. From tabulated results it appears that for lower damping values nonlinear response is more sensitive to the uncertainty in natural periods.

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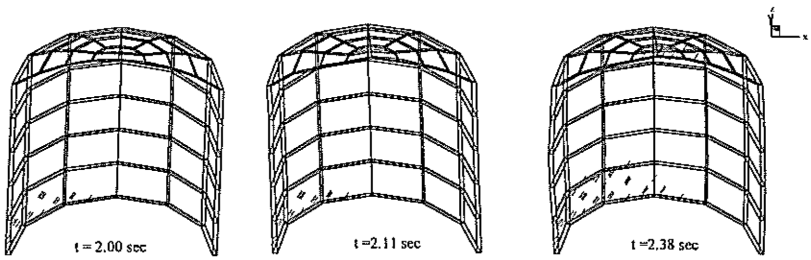
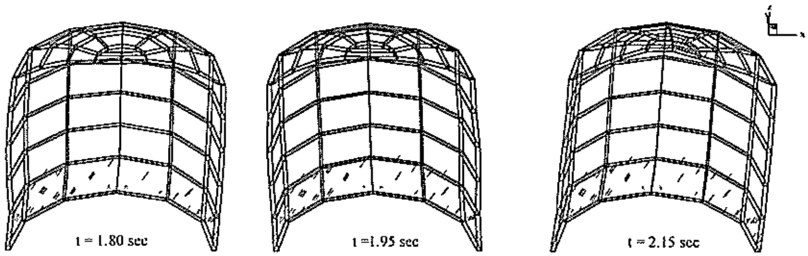


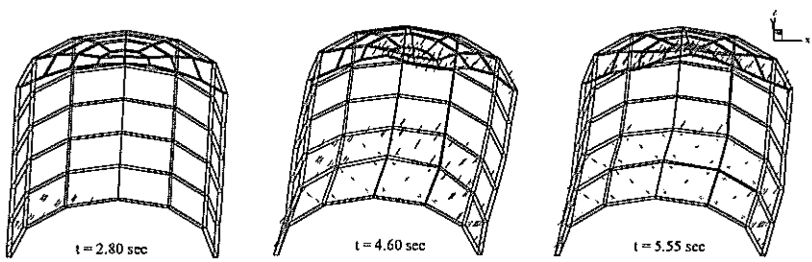
a) Cylinder thickness =1.2m & dome thickness = 0.61m [Case(ii)]



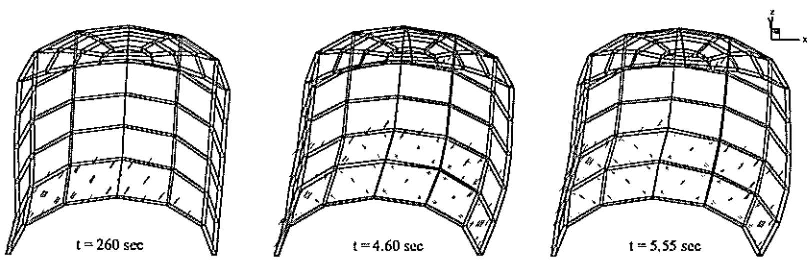
b) Cylinder & dome thickness = 1.20m [Case(iii)]

/ Single Crack
Double Crack

Fig. 5 Deflected shape and cracking in containment due to sine sweep earthquake in time domain



a) Cylinder thickness =1.2m & dome thickness = 0.61m [Case(ii)]



b) Cylinder & dome thickness = 1.20m [Case(iii)]

/ Single Crack
\ Double Crack

Fig. 6 Deflected shape and cracking in containment due to Koyna (longitudinal) earthquake in time domain

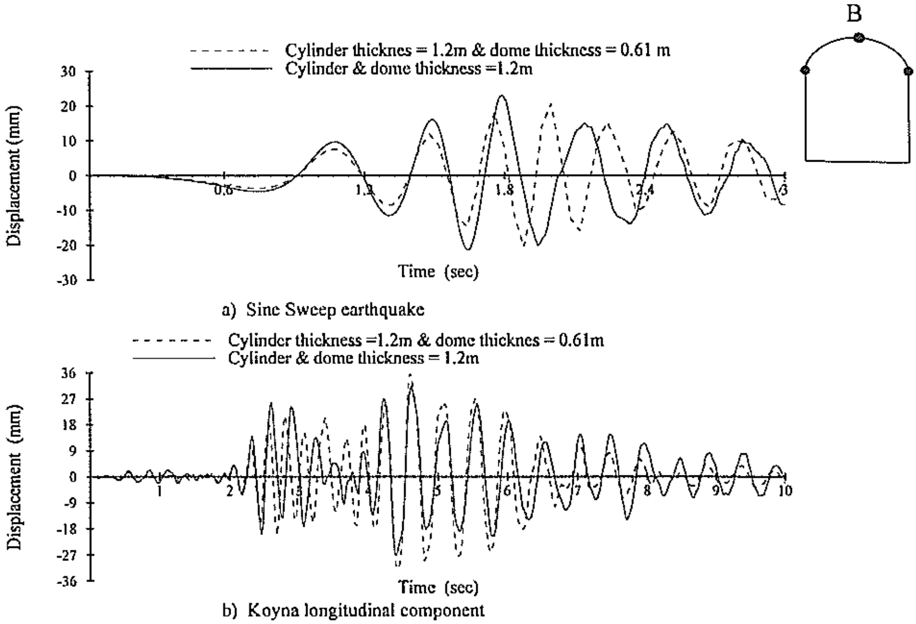


Fig. 7 Horizontal displacement (nonlinear) of Point B

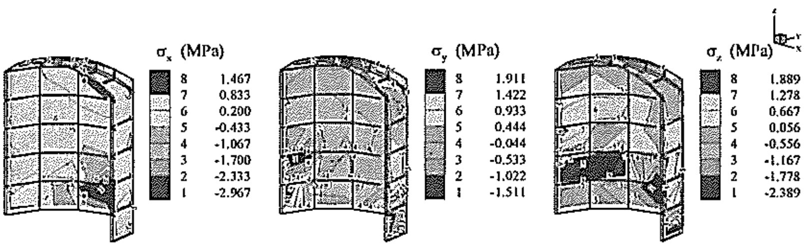


Fig. 8 Peak stresses in concrete subjected to Koyna (longitudinal) earthquake
[Cylinder & dome thickness = 1.2m]