



**NUMERICAL METHODS FOR EVALUATION OF SEISMIC FRAGILITY
ANALYSIS OF NUCLEAR POWER PLANT BUILDING**

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ABSTRACT

Evaluation of seismic vulnerability for nuclear power plant structure in seismicity origin is very important. Because we can't deny the ground motion which is more than the maximum ground motion that we assumed in designing. One of the main component for evaluating of seismic vulnerability of nuclear power plant structure is the evaluation of seismic fragility functions. Fragility curves are defined as a probability of structure responses at the special damage state or exceeding from that state for different earthquake intensity. Over the last years, for determining the fragility curves of nuclear power plant building, incremental dynamic analysis preferred. Lognormal distribution is used for obtaining the fragility curves in each level of hazard. In this research the structure is modeled by two numerical methods, finite element model and lumped mass stick model and then these two methods compared with each other. For this aim, modal analysis done in each methods. The results represented that the dynamic properties which obtained from these two methods are approximately the same. Incremental dynamic analysis done for 30 records with seismo geological origin. Finally, the fragility curves obtained from IDA curves for different limit state and it was concluded that the lumped mass stick model estimated more damage than finite element model.

Keywords: Nuclear containment, lumped-mass stick model (LMS), Finite element model (FEM), incremental dynamic analysis, Fragility analysis, lognormal distribution

1. INTRODUCTION

The safety of nuclear power plant (NPP) depends upon a number of factors, intrinsic and external to the plant. For evaluating the safety of the plant, the seismic ground motion is one of the important considerations. The possibility of occurrence of stronger earthquake motions which was not predicted in designing cannot be denied.

The purpose of evaluating seismic probabilistic safety assessments, is vulnerability estimation of existing structures against earthquake hazards. It involves numerically assessing the plant safety in a probabilistic framework, so that appropriate measures can be taken to reach a NPPs safety level. One of the major components in the seismic PSA of a NPP, is the seismic fragility evaluation. The seismic fragility of a structure is defined as its conditional probability of failure given a specific intensity of the hazard. The fragility of a system is illustrated by a conditional probability of failure against seismic hazard curve. The probability of failure can be defined for any limit state that is selected in computations. For example, recent works in structural earthquake engineering commonly defined displacement-based limit states in terms of inter-story drift ratio, however force-based ground acceleration capacity estimated by traditional fragility.

The seismic hazard usually describe by the pseudo spectral acceleration corresponding to the mainMode (S_a), or by the peak ground acceleration (PGA) while obtaining fragilities curve.

2. The fragility analysis of structures a brief review

(Yeom, 2006) used two methods, finite element and stick model, for determining dynamic properties of power plant building. Abaqus platform is used for this goal. Results represented that the frequency and effective mass are close to each other in these two methods. It was concluded that the dynamic properties of power plant which is evaluated from LMS are approximately the same with FEM, on the other hand run time of the FEM is more than LMS. So, it's better to use LMS instead of FEM for determining dynamic properties of complex structures such as containment. Finite element method and lumped mass stick model is used for enhancing fragility curve of power plant building by Naohiro Nakamura et al, 2010. It was concluded that lumped mass stick model estimated more damage than finite element model.

(Pujari, 2013), used lumped mass stick model for determining fragility curve of nuclear power plant building. For this aim, incremental dynamic analysis (IDA) done by Opensees platform. Nonlinear Beam

Column element, Fiber Section, concrete02 and steel01 element are used for modeling of structure, cross-section, concrete and steel materials, respectively. They suggested a method for rehabilitation of fragility curves, and then they obtained that the results of suggested method is closer than conditional approximately method to the results of IDA curves.

Shell elements were used in modeling of nuclear power plant by Abaqus platform. In this study the IDA curve was conducted by an equation which was obtained from regression method. They obtained that the vulnerability of structure based on cracking tension is more than the vulnerability of structure based on inter story drift ratio.

Fragility curves of containment obtained based on the displacement responses from inelastic nonlinear seismic analyses for near fault ground motion and design ground motion. The conclusion of this research represented that displacement response which obtained by design ground motion is greater than near-fault ground motion. So, the near fault ground motion has no effect on stiff structures such as nuclear power plant building.

(Lee H. R., 2012) proposed the frequency adaptive LMS model, which evaluated the amount and locations of lumped masses by using the dynamic characteristics of the structures, such as eigenvector, mode

shapes, and modal participation masses. In addition, applied the modeling approach to a single nuclear power plant building and compared the dynamic responses to those of the solid element based FE model.

In this study, nuclear power plant modeled by FEM and LMS model for evaluation of fragility analysis. In order to incremental dynamic analysis (IDA) for these models performed in the Abaqus and Open sees platform, respectively. Then multi-IDA curves obtain from these models. Finally, fragility curves obtain from these IDA curves for different limit state.

3. Analysis model

3.1. 3D FEM model

Fig.1.a shows the nuclear containment structure of the U.S Washington considered for this case study as an assumption for analysis and table 1 shows the property of materials and geometry of this building.

Fig.1.b shows the three dimensional finite element model which is used for this structure. This structure is modelled in Abaqus by using 3D linear shell elements. Layered shell is used for defining the cross-section, so that reinforcement layers can be modelled along the thickness of the containment. The mode shapes or eigen values are obtained by an eigenvector analysis for the finite element (FE) modeling. The fundamental frequency of the containment building obtained by the 3-

D FE (Finite Element) model, the results of this analysis represented that the value of this frequency is 5.45Hz. The mode shapes and frequency of this structure are illustrated in fig. 2.

The uniaxial compression and tension behaviors of concrete are modelled by using of concrete damage plasticity behavior and also the uniaxial compression and tension behaviors of reinforcement are modeled by using of bilinear behavior. Nonlinear response history analysis is used for this containment and the explicit dynamic mode is used for FEM analysis. In this study the

soil-structure interaction was neglected, because the containment building constructed on the rock and the containment is assumed to be fixed at the base.

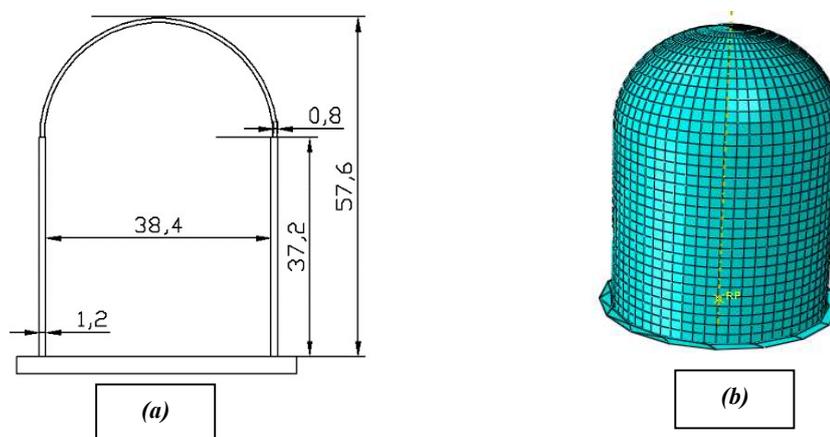


Figure 1: a) Primary containment b) FEM

Table 1: Property of material and structure

material	(N/m2)		(kg/m3)	E (N/m2)	(N/m2)		S(cm)
concrete	3.6e7	0.17	2400	2.74e10
Steel	0.3	7850	2.1e11	4.2e8	40	8

** Where, ϕ and S are respectively diameter of reinforcement and distance of transverse reinforcement.

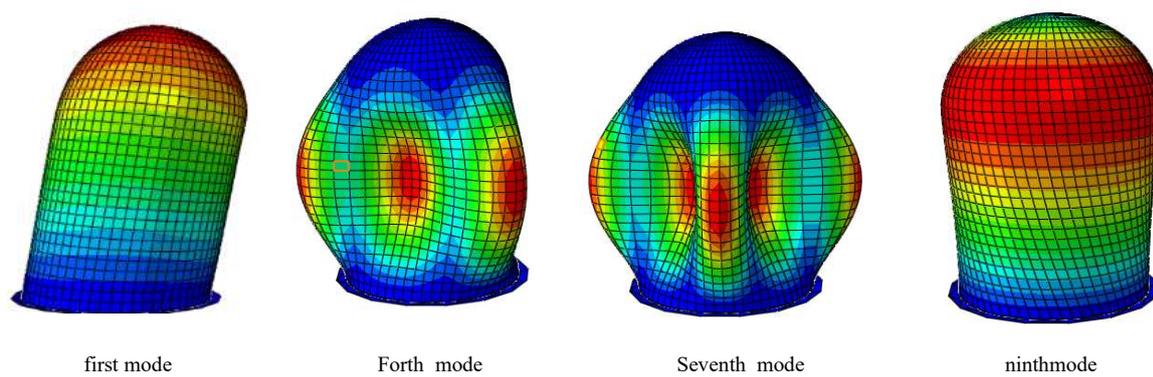


Figure 2: mode shape for actual structure

3.2 Lumped Mass stick model

For complex column typed-structures, such as, nuclear containment buildings and electric post bushings the lumped-mass stick model is usually accepted as a simplified model. An actual structure is discretized with a series of column its modal participation mass ratio, is normally above 90%. If more accurate responses are required, the number of the lumped mass locations can be increased higher than 90%. The eigenvalues or mode shapes are obtained for the finite element (FE) modeling of a main structure through a modal analysis. The fundamental frequency of the nuclear power plant obtained by the 3-D Finite Element model and the lumped mass model were 5.45Hz and 5.52Hz, respectively. The results of FEM and LMS exhibited well agreement

elements for the lumped-mass stick model. The lumped mass at each node is determined from the portion of the weight which can reasonably be assigned to each node. The number of the lumped mass locations is the same as the number of modes of actual structure which for the higher modes. The participation mass ratio in ninth mode are above 90 %, but for increasing the accuracy of nonlinear response history thirteen modes are defined. The modal mass of the fundamental mode is more than 85%. This result represented that for nonlinear dynamic time history analysis, the lumped mass model can be used as an approximate method instead of the 3-D FE model. The lumped mass stick model for the containment building is shown in Fig. 3

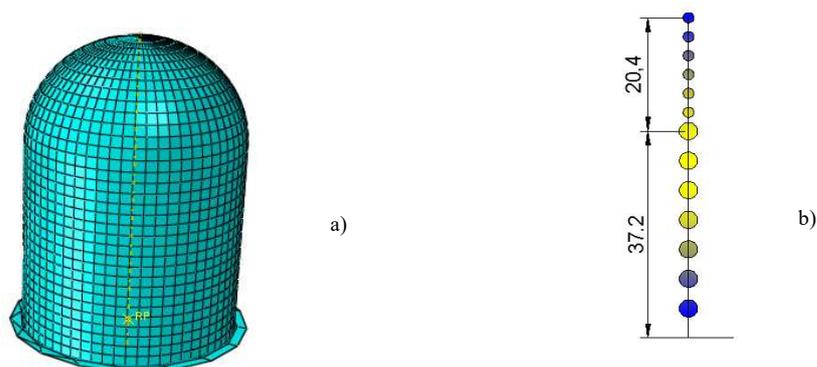


Figure 3: a) FEM b) lumped mass stick model (LMS)

4. Incremental dynamic analysis

Multi-IDA curves, are performed in OpenSees and Abaqus platform for LMS and FEM, respectively. In the Opensees platform the *nonlinear Beam Columnelement* is used for structure modeling, which is fixed by raft foundation at the base. FiberSection, concrete02 and steel01 element are used for modeling of structure, cross-section, concrete and steel materials, respectively. The shear deformation behavior is modelled using the section Aggregator approach.(Opensees Wikipedia)

Nonlinear response-history analyses of the idealized 2D stick model and FEM is performed for 30 ground acceleration records. The details of these acceleration records are provided in Table 2. The artificial time histories that generated for the analyses were envelop the US NRC Regulatory Guide 1.60 spectrum. Fig. 4 shows this standard response spectrum.(Committee., 1973)

$$\theta_{max} = Ax^2 + Bx + c$$

Eq. (1)

The DM parameter which was selected in this study is the maximum interstory drift ratio (θ_{max}). Three limit states are considered for this damage parameter: LS1, LS2 and LS3, corresponding to the IO, LS and CP performance levels of US NRC Regulatory Guide 1.60, respectively. For these limit states $\theta_{max} = 0.4\%$, 0.6% and 0.75% are selected, respectively. For each earthquake, the results of NLRHA can be shown such a curve that PGA versus maximum interstory drift ratio (θ_{max}) which is known as IDA curve. PGA and θ_{max} can be depended to each other by an equation which is selected by Regression (Committee., 1973).

At a PGA of x . A, B and C are regression coefficients. Values of the regression coefficients obtain for each earthquake. The fitted PGA level vs. drift curves are shown in Fig.5 for FEM and LMS. The vertical lines named LS1, LS2 and LS3 illustrated the three limit states of performance.

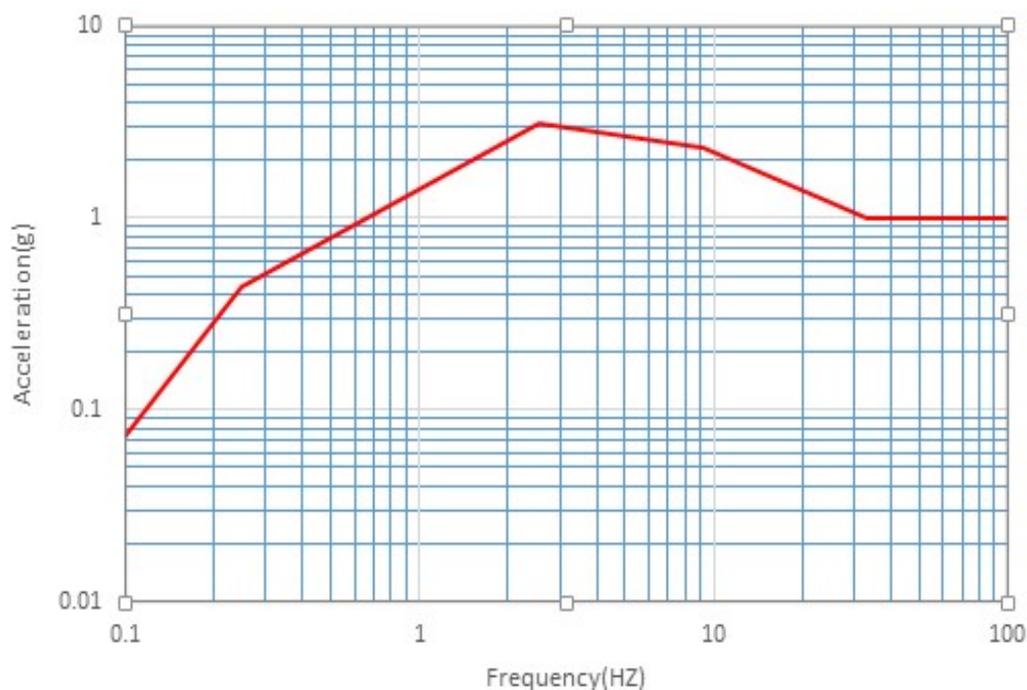


Figure 4: Regulatory Guide 1.60 Design Response Spectra -5% Damping

Table 2: Details of the ground motion data considered for the study.(PEER Website)

Record	Event	R(km)	PGA(g)	Mag
GM-01	Cape Medocino-1992	8	0.46	7.1
GM-02	Chalfant Valley, 1986	41	0.047	6.2
GM-03	Coalinga – 1983	11	0.2	5.8
GM-04	Coalinga – 1983	34	0.144	6.4
GM-05	Coyote Lake – 1979	3	0.4	5.7
GM-06	Friuti, Italy – 1976	13	0.13	6.1
GM-07	Gazli, USSR- 1976	3	0.86	6.8
GM-08	Helena- 1935	8	0.15	6.2
GM-09	Imperial Valley-1979	26	0.19	6.5
GM-10	Kobe, Japan- 1995	0.2	0.31	6.9
GM-11		49	0.08	
GM-12	Landers -1992	1.1	0.66	7.3
GM-13		42	0.55	
GM-14	Loma Prieta -1989	6	0.44	6.9
GM-15		21	0.46	
GM-16	Mammoth Lakes -1980	20	0.10	6
GM-17	Morgan Hill -1984	0.1	0.54	6.2
GM-18		16	0.08	
GM-19	Nahanni, Canada -1985	6	1.08	6.8
GM-20		16	0.13	
GM-21	Northridge -1994	8	1.07	6.7
GM-22		42	0.06	
GM-23	N. Palm Springs- 1986	26	0.065	6
GM-24	Parkfield -1966	10	0.35	6.1
GM-25	Santa Barbara -1978	37	0.068	6
GM-26	San Fernando- 1971	20	0.2	6.6
GM-27	Spitak, Armenia- 1988	30	0.17	6.8
GM-28	Victoria, Mexico -1980	35	0.1	6.1
GM-29	Whittier Narrows -1987	9	0.28	6
GM-30		21	0.13	

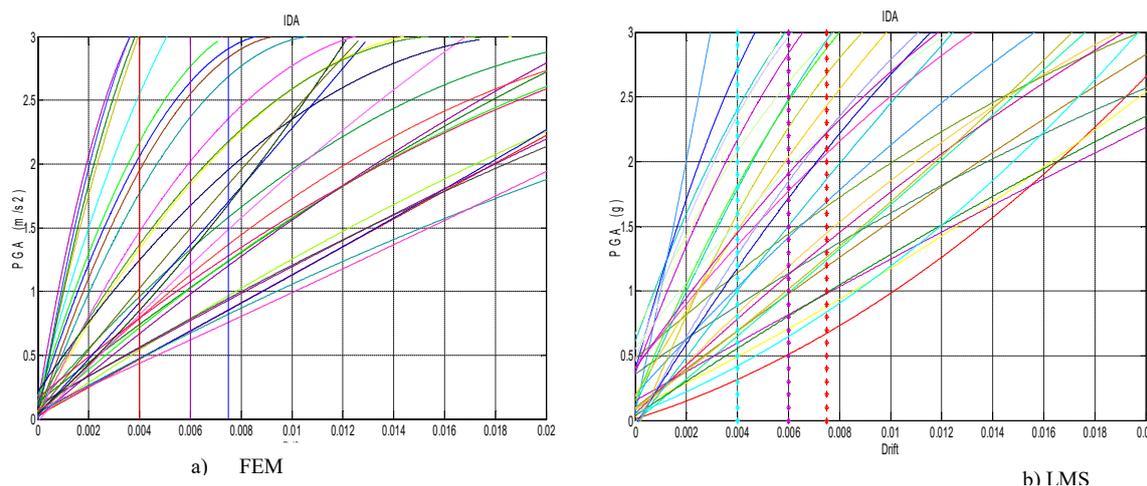


Figure 5: IDA plots for the selected 30 ground records.

5. Fragility curve

5.1 Evaluation of seismic fragility analysis by lognormal distribution method

Damages which are affected to a special structure divided to different and predefined conditions. Fragility function defined such as rising of probabilistic physical damages for known state which is represented in blow equation.(Keith Porter, 2015)

$$F_i(IM) = P(d_i > DI_i | IM) \text{ Eq. (2)}$$

Where

$F_i(IM)$ = a fragility function for damage state DI_i evaluated at x .

d_i = Uncertain damage state of a particular component

DI_i = A particular value of D , i.e., with no uncertainty

By assuming lognormal distribution for damage index, equation 3 can be written:

$$\text{Eq. (3)}$$

Standard lognormal cumulative distribution function can be written by numerical results:

$$\text{Eq. (4)}$$

Where

$P(D > d_i | IM)$ = standard normal cumulative distribution function.

IM = intensity parameter.

IM_i = median parameter for each damage estate d .

β_i = the standard deviation of the natural logarithm for each damage estate d .

5.2 Fragility Plots

Seismic fragility of a containment, which is the probability of a predefined damage measure (DM) exceeding certain predefined limit states (LS) for a given intensity measure (IM), is calculated as the fraction of IDA curves exceeding the LS at

the selected PGA. Fragility analysis results illustrated in a set of $P(DM > LS | IM)$ vs. IM plots. The fragility plots are obtained from discrete number of ground motion data which are used for multi-IDA.

Fragility plots which are based on different performance limit states as explained

before in incremental dynamic analysis are shown in fig. 6.

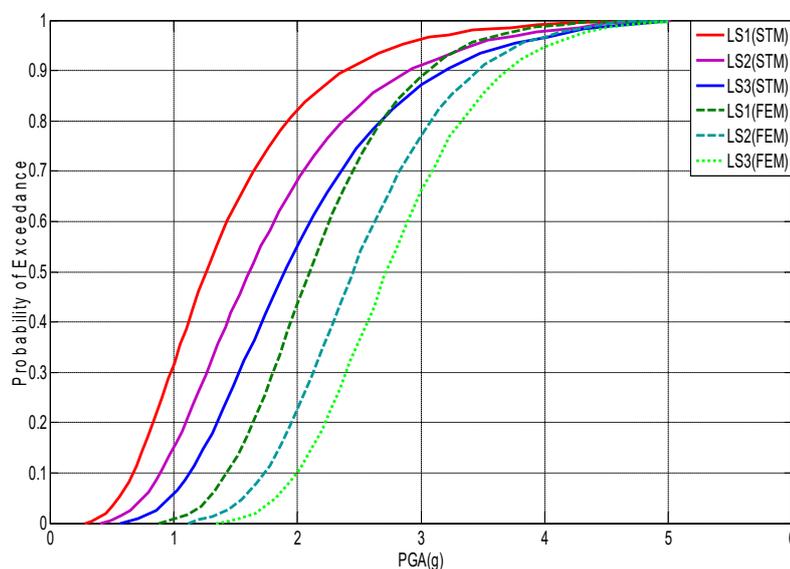


Figure 6: Fragility curves and IDA data for the three limit states.

The results of FEM model and LMS model are shown as fragility curves in fig. 6 and also these curves are compared with each other.

For the same input, the results that obtained from lumped mass model for the damage probability is overall larger than the FEM model results. On the other aspect of view, lumped mass model have a tendency to estimate the damage larger than the FEM model.

6. CONCLUSION

In this study, a nonlinear 3D FEM model and Lumped mass stick model were used to conduct seismic response analysis of a nuclear power plant building in an attempt to evaluate the seismic PSA building fragility.

- 1- Developed lumped-mass model and 3D shell element model have good agreement in the results of dynamic performance.
- 2- This result represented that for nonlinear dynamic time history analysis, the lumped mass model can be used as an approximate method instead of the 3-D FE model
- 3- Evaluation of fragility analysis in both methods (FEM&STM) represented that for the same input, the results that obtained from lumped mass model for the damage probability is overall larger than the FEM model results. One of the most important reasons of this performance is the cylindrical cross

section which is assumed for the dome in lumped mass stick model.

- 4- As mentioned above, the LSM estimated the rate of damage more than the FEM. So, the safety of lumped mass model is more than finite element model. On the other hand, LSM run time for assessing the acceptable results is less than FEM.
- 5- Fragility curves illustrated that in small PGAs, the differences of FEM and LMS is greater than the one which PGA is bigger.

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