COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, N.D. Lagaros, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

SEISMIC FRAGILITY ANALYSIS OF A NUCLEAR PRIMARY CONTAINMENT STRUCTURE

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Keywords: Seismic Fragility, Concrete Damage Plasticity, Nuclear Power Plant, Primary Containment, Tensile Cracking.

Abstract. *A seismic fragility curve is an important tool in the probabilistic performance assessment of a structure when subjected to earthquake loads. The seismic fragility of a structure can be defined as the probability of exceeding certain limit state(s) of performance, given a specific level of seismic hazard. The current study deals with the seismic fragility estimation of a nuclear primary containment structure of a typical PHWR in India. The focus of this work is to estimate the seismic fragility of this structure considering a failure defined in terms of tensile cracking of the damaged concrete which leads to leakage. The structure is modelled using 3D layered shell elements with detailed damaged plasticity model for concrete. Seismic responses in terms of equivalent plastic tensile strain are studied for a set of earthquake records at different intensity levels through a series of nonlinear response-history analyses. Using regression, the ground acceleration capacity of the structure are obtained for each record. The seismic fragility curve of the structure is derived using a lognormal model using the sample capacity data. This seismic fragility curve is compared to the conventional fragility curve based on an interstorey drift ratio based failure. The results show that tensile cracking based fragilities are more severe to the drift based fragilities.*

1 INTRODUCTION TO SEISMIC FRAGILITY ANALYSIS

Seismic fragility is the probability that a geotechnical, structural or a non-structural system violates at least one limit state when subjected to a seismic event of a specified intensity. It is the conditional probability of failure of the system given a specific intensity of the seismic hazard. The necessity of conducting seismic fragility assessments for important structures – such as nuclear power plants, large dams and bridges – is acknowledged in the engineering community worldwide. The objective of seismic probabilistic safety assessments (PSA) for nuclear power plants (NPP) is to examine the existence of vulnerabilities against postulated earthquake hazards [1]. Seismic fragility analysis of a NPP is conducted both at its component levels and at the system level. The pioneering work on the development of a systematic methodology for seismic fragility analysis of a nuclear power plant and its components was conducted by Kennedy et al. [2]. This method was modified later to a more systematic approach by Kennedy and Ravindra [3]. They obtained the fragility of a structure, using a lognormal model, as

$$F_r = \Phi\left[\frac{\ln(x/m_a) + \beta_U \Phi^{-1}(Q)}{\beta_R}\right] \tag{1}$$

$\Phi(.)$ is the standard normal CDF operator; x is the seismic intensity (typically, PGA) at which the fragility is evaluated, m_a is the median ground acceleration capacity, β_R and β_U respectively measure the randomness (aleatory) and uncertainty (epistemic) associated with the estimation of ground acceleration capacity, and Q is the non-exceedance probability level. The ground acceleration capacity (a) is expressed in terms of its median capacity (m_a) and associated uncertainties:

$$\begin{aligned} a &= m_a \epsilon_R \epsilon_U \\ &= a_{\text{DBE}} \bar{F} \epsilon_R \epsilon_U \\ &= a_{\text{DBE}} (F_S F_\mu F_R) \epsilon_R \epsilon_U \end{aligned} \tag{2}$$

where ϵ_R and ϵ_U follow lognormal distributions with a median equal to one and lognormal and standard deviations β_R and β_U, respectively. a_{DBE} is the intensity of the design basis earthquake, $\bar{F}$ is the median factor of safety, which is composed of three different factors of safety. The details of these factors of safety and the associated uncertainties were discussed in depth in a recent paper by Pisharady and Basu [4]. Due to the lack of necessary tools and data, the quantification of these uncertainties depended significantly on engineering judgement. Although this method originally used response spectrum based linear elastic analyses, Pisharady and Basu [4] showed that modern analysis techniques, such as the nonlinear static pushover analysis can easily be incorporated in this framework. The two-parameter lognormal model of Equation 1, in its various forms, has now been widely accepted for seismic fragility analysis of structures. For example, Ellingwood et al. [5] used this model for fragility analysis of building structures in the following form:

$$F_r = \Phi\left[\frac{\ln\{c(S_a)^d/\theta_{\text{LS}}\}}{\beta_R}\right] \tag{3}$$

where, the failure was defined by the maximum interstorey drift ratio ($\theta_{\max}$) exceeding its limiting value θ_{LS}. The median value of $\theta_{\max}$ was expressed as a continuous function of the intensity measure S_a, using nonlinear regression:

$$m_\theta = c(S_a)^d \tag{4}$$

where, c and d were regression parameters and S_a was the pseudo-spectral acceleration corresponding to the fundamental mode of vibration of the structure. The regression was based on the earthquake response data from a series of nonlinear response-history analysis (NLRHA).

2 OBJECTIVE

Conforming to these basic formats of seismic fragility estimation, the work presented here shifts focus to the performance limit state that is used to define the failure of a structure. Initial research works in this area defined performance in terms of force capacities or ground acceleration capacities. Later, seismic structural engineering moved into performance-based seismic design and performance-based seismic evaluation, where structural ‘failure’ is defined by multiple performance limits described by displacement-based limits. For example FEMA-356 [6] defined performance limits for building structures in terms of maximum interstorey drift ratio values. This damage measure is able to represent both the structural and non-structural damages in a seismic event more efficiently than any force-based demand parameter. Although drift-based limit states have been suggested for containment structures in NPP, this demand/response parameter is not a good indicator of the associated risks.

The primary containment structure is the last barrier to radioactive leakage, and is therefore considered to be the most important civil/structural engineering component in a NPP. From this understanding, the limit state of failure is defined here by the possibility of any leakage through the inner/primary containment. Such leakages can only take place if the seismic event leaves a through crack in the concrete shell. Assuming that tensile cracking in concrete takes place much earlier compared to crushing under compression, the limit state of failure in this work is defined by a positive equivalent plastic tensile strain ($\tilde{\varepsilon}_t^{pl} > 0$) in concrete, anywhere in the containment structure.

3 MODELLING OF TENSILE CRACKING IN CONCRETE

Available literature shows a variety of concrete material modelling approaches that can incorporate the inelastic behaviour of concrete, including tensile cracking. Since the study structure is analysed using the general purpose finite element package Abaqus [7], the discussion here is limited to the concrete inelastic material modelling options available with this software. These are

1. Brittle cracking model (Section 19.6.2 of [7]),
2. Concrete smeared cracking (Section 19.6.1 of [7]), and
3. Concrete damaged plasticity (Section 19.6.3 of [7]).

Although the brittle cracking model is assumed to be good for modelling tensile cracking, it suffers from the fact that concrete under compression is always assumed to behave as a linear elastic material. The smeared cracking approach is based on damaged elasticity concepts. This model does not track individual cracks but distributes/‘smears’ the overall effect of these cracks. The cracking effect is incorporated by specifying a reduction in the shear modulus as a function of the opening strain across the crack. A reduced shear modulus for closed cracks can also be specified. However, its application should be limited to cases where concrete is subjected to essentially monotonic straining at low confining pressure.

The concrete damaged plasticity model combines isotropic damaged elasticity in the elastic domain with isotropic tensile and compressive plasticity to represent the inelastic behaviour of concrete. It can be used for cyclic and dynamic loading conditions, such as earthquake excitations. This modelling approach is selected for fragility estimations in this work. Two hardening variables, equivalent plastic tensile strain ($\tilde{\varepsilon}_t^{pl}$) and equivalent plastic compressive strain ($\tilde{\varepsilon}_c^{pl}$) control the change in the yield surface. $\tilde{\varepsilon}_t^{pl}$ also represents the existence of a crack in concrete because it is the residual or plastic equivalent strain. Figure 1 shows this quantity for the uniaxial stress-strain plot for the damaged plasticity concrete model. E_0 is the linear elastic undamaged modulus in tension and ε_t^{el} is the recoverable or elastic tensile strain. Following Lubliner et al. [8], it is assumed that cracking initiates at points where the equivalent plastic tensile strain is greater than zero.

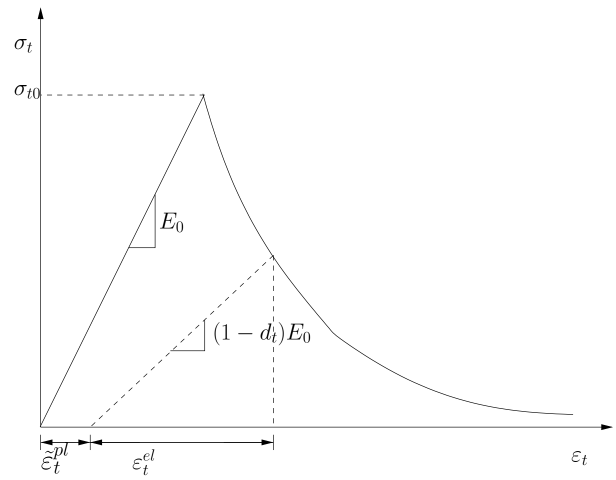

Figure 1: Concrete damaged plasticity under uniaxial tension.

4 STUDY STRUCTURE AND ITS FINITE ELEMENT ANALYSIS

The primary/inner containment (PC/IC) structure of the 700 MWe Indian PHWR is considered for this case study. The PC is a prestressed concrete cylindrical wall capped by a segmental prestressed concrete dome through a massive ring beam (Figure 2). At the bottom of the PC there is a base raft. The cylindrical wall has two rectangular access holes, and the dome has two circular steam generator openings. The containment wall is thickened near the base raft, near the ring beam and around the openings.

This structure is modelled in Abaqus [7] using 3D linear shell elements, S4R (and some S3R). The cross-section is defined as a layered shell, so that reinforcement layers can be modelled suitably along the thickness of the containment. Prestressing effects are not included based on the simplifying assumption, that prestressing effects will be nullified by the presence of internal pressure (and also prestress loss over time). The containment is assumed to be fixed at the base. The meshing of the study structure, with 10503 layered shell elements and 10515 nodes, is also shown in Figure 2. The uniaxial compression and tension behaviours of concrete are modelled using damaged plasticity and the Drucker-Prager flow rule following the work by Jankowiak and Lodygowski [9], after normalising the behaviour to M45 grade concrete as per IS:456 [10]. The Fe500 grade reinforcements are modelled to have elastic-curvilinear plastic behaviour as per IS:456.

This structural model is subjected to uni-directional horizontal ground acceleration time-histories and nonlinear response-history analyses (NLRHA) are performed. For running these

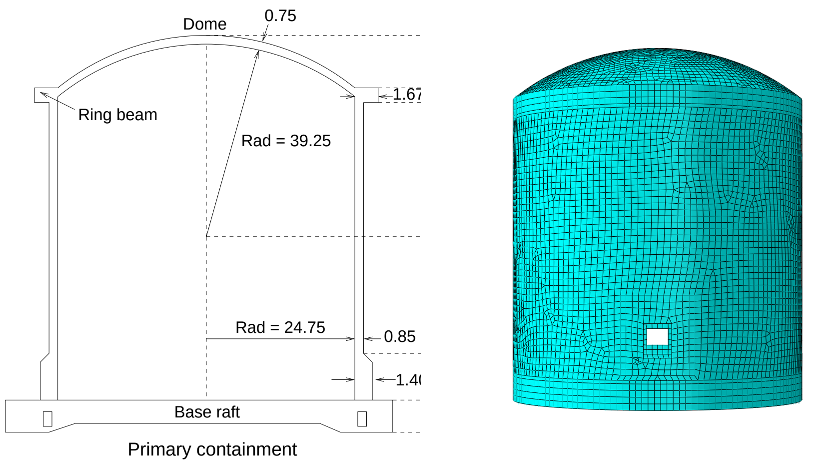

Figure 2: Schematic of the structure and its finite element model.

NLRHA, the implicit dynamic mode is used in Abaqus to ensure greater accuracy of results. From the output data, $\tilde{\varepsilon}_t^{pl}$ is recorded over all the shell elements.

5 FRAGILITY ANALYSIS OF THE CONTAINMENT STRUCTURE

The containment is assumed to be located in the peninsular India, and 13 recorded ground motions are selected from similar seismo-geological sources for the NLRHA. For each record, NLRHA are conducted at increasing intensity levels – similar to an incremental dynamic analysis (IDA) approach – and $\tilde{\varepsilon}_t^{pl}$ values are noted. Since, each NLRHA is a significantly computation intensive effort, instead of performing this at a large number of intensity scale factors, we try to narrow down to the intensity scale factor that indicates the upcrossing $\tilde{\varepsilon}_t^{pl} > 0$. This needs six-eight trials for each earthquake record. However, for most of these ground motion records, intensity measures (in terms of the peak ground acceleration or PGA) are identified corresponding to very low positive values of $\tilde{\varepsilon}_t^{pl}$ (of the order 10^{-6}). Figure 3 shows the $\tilde{\varepsilon}_t^{pl}$ contours ('PEEQT' in Abaqus), for the whole containment, as well as a magnified view around the rectangular hole where the cracking takes place.

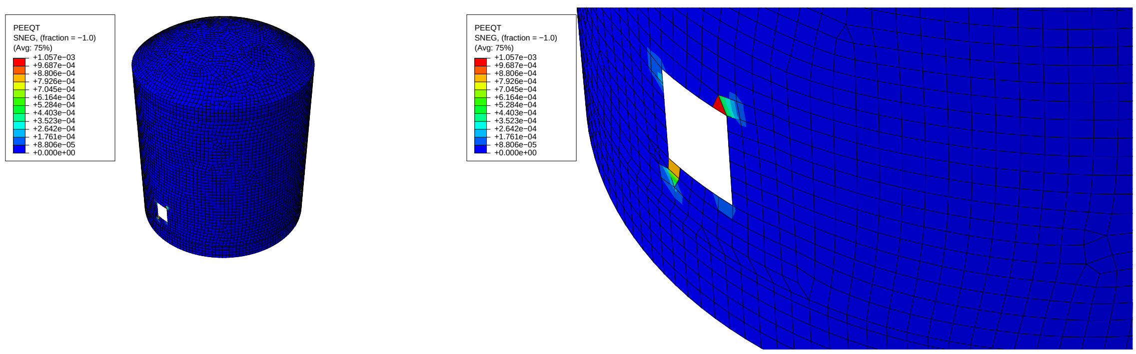

Figure 3: $\tilde{\varepsilon}_t^{pl}$ contours for a sample earthquake record at a selected intensity level.

To find the lowest PGA level at which $\tilde{\varepsilon}_t^{pl}$ has a positive value, all data points for an earthquake in a PGA vs. $\tilde{\varepsilon}_t^{pl}$ plot are fitted with an approximating curve. For each earthquake, results from only those NLRHA can be used that give a positive equivalent plastic tensile strain. The regression curves are selected in the following the general form

$$\tilde{\varepsilon}_t^{pl} = Ax^2 + Bx + C \tag{5}$$

at a PGA of x. A, B and C are regression coefficients. This form is selected so that $\tilde{\varepsilon}_t^{pl}$ is a monotonically increasing function of the intensity level x. Although IDA curves with displacement-based damage parameters sometimes show rebounds, it is safer to make the simplistic assumption that equivalent plastic tensile strain values do not decrease with increasing seismic intensity levels. Values of the regression coefficients are provided in Table 1 and the fitted PGA level vs. $\tilde{\varepsilon}_t^{pl}$ curves are shown in Figure 4. The intercept of these curves at $\tilde{\varepsilon}_t^{pl} = 0$ gives the ground acceleration capacity of the structure for each of these records. These limiting PGA values are also provided in Table 1.

Record	A	B	C	PGA at $\tilde{\varepsilon}_t^{pl} = 0$ (g)
GM-12	0.08469	-0.02465	0.001729	0.1732
GM-13	0.1398	-0.07274	0.009387	0.2833
GM-14	1.245	-0.2218	0.009807	0.09666
GM-15	-0.006588	0.001804	0	0
GM-16	0.1219	-0.02798	0.001602	0.1202
GM-17	0.005737	-0.001112	0.4000	0.1938
GM-18	1.742	-1.158	0.1923	0.3415
GM-19	1.655	-0.4375	0.02886	0.1379
GM-20	18.88	-2.845	0.1068	0.07981
GM-21	0.00191	-0.0002237	-9.536$\times 10^{-6}$	0.1503
GM-22	-0.014	0.004044	0	0
GM-23	0.01783	-0.00367	0.0001883	0.1085
GM-24	0.01407	-0.002843	0	0.2021

Table 1: Regression coefficients for the selected records.

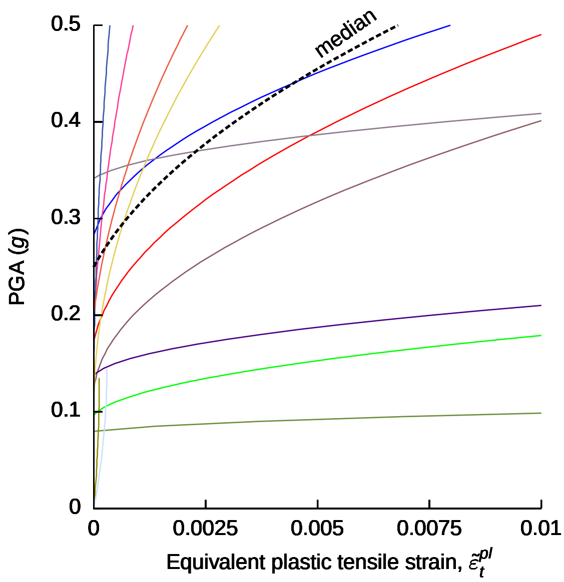

Figure 4: Regressed intensity vs. damage curves for selected records.

The variation in ground acceleration capacities (as listed in Table 1) is modelled using a two-parameter lognormal distribution, with median $m_a = 0.1732g$ and lognormal standard deviation $\beta_R = 0.4461g$. A Kolmogorov-Smironov goodness-of-fit test gives a $D^n_{\max}$ value of 0.1010, which is well within the limiting D^n_α values for all acceptable significance levels. These m_a and β_R vales are used to obtain the fragility curve based on tensile cracking, using Equation 1. The epistemic uncertainties are neglected in this fragility estimation in comparison with the (aleatory) randomness in the earthquakes. The fragility curve based on tensile cracking is plotted in Figure 5. For a comparison, the fragilities are also obtained considering a maximum interstorey drift ratio based failure. For this, the 'immediate occupancy' (IO) limit state defined in FEMA-356 is considered: $\theta_{\rm LS} = 0.004$ [6]. This fragility estimation is based on the method used by Pisharady and Basu [4] using the results of a nonlinear static pushover analysis. This fragility curve is also presented in Figure 5.

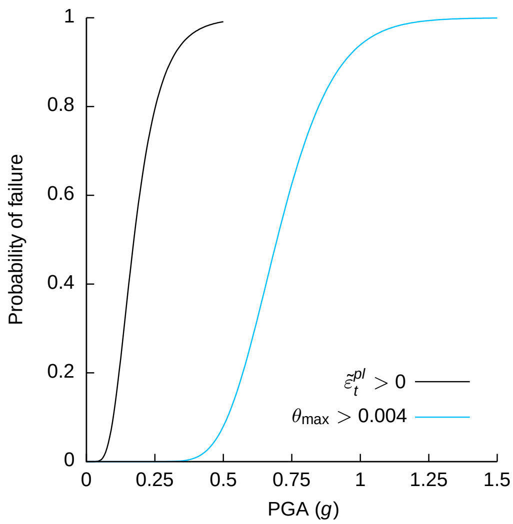

Figure 5: Fragility curves based on tensile cracking and drift based failure.

6 CONCLUDING REMARKS

This paper presents a preliminary attempt in obtaining fragility estimates for the primary containment structure of a NPP in terms of the possibility of leakage through the damaged concrete due to seismic events. The first results show that tensile cracking based fragilities are much higher compared to drift ratio based fragilities, even when the drift limit is defined at the immediate occupancy performance level. It points to the fact that crack/leakage-based performance levels should be considered for this type of structures.

The primary challenge in this task has been in the amount of computation involved in non-linear response-history analysis while adopting a reasonably accurate damage model for the containment. This has lead to several simplifying assumptions, which should be avoided. For example, the number of ground motion records may be considered to be less than adequate, and so are the number of data points with positive $\tilde{\varepsilon}_t^{pl}$ values for each earthquake. Similarly, the validity of the assumption that prestressing effects have not been present needs to be checked. This work needs to be verified again after these issues are properly addressed using better data and probabilistic analysis tools.

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