Development of Fragility Curves for an RC Frame

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INDIA

CERTIFICATE

This is to certify that the thesis entitled "**Development of Fragility Curves for an RC Frame"** submitted by **Sanju J Thachampuram** in partial fulfilment of the requirement for the award of **Master of Technology** degree in **Civil Engineering** with specialization in **Structural Engineering** to the National Institute of Technology, Rourkela is an authentic record of research work carried out by her under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

Project Guide

Prof. ROBIN DAVIS P

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Abstract

Keywords: *HDMR, Fragility Curve, Cornell's Method, Latin Hypercube Sampling, MCS, Probability of exceedance*

Fragility curves provide the conditional probability of structural response when subjected to earthquake loads as a function of ground motion intensity or other design parameters. Seismic fragility curves are used mainly by decision makers for the assessment of seismic losses both for pre-earthquake disaster planning as well as post-earthquake recovery programs. Generation of fragility curves in conventional methods involves development of large number of computational models that represent the inherent variation in the material properties of particular building type and its earthquake time history analyses to obtain an accurate and reliable estimate of the probability of exceedance of the chosen damage parameter. There are many Response surface methods available in the literature that is capable of representing the limit state surface depending on the problem type. High Dimensional Model Representation (HDMR) method is a type of response surface method that can express input-output relations of complex computational models. This input-output relation can reduce the number of iterations of expensive computations especially in problems like fragility curve development. Unnikrishnan *et al.* (2012) applied this technique in fragility evaluation for the first time and demonstrated its computational efficiency compared to computationally intensive Monte Carlo method. In this study, fragility curve of an RC frame is developed using HDMR response surface method. There are also other simplified approaches which are computationally easy for fragility curve development. Cornell *et.al.* (2002) proposed such a simplified method which assumes a power law model between the damage parameter and intensity measure of earthquake. This study presents Fragility curves evaluated using HDMR and its computational efficiency with reference to the one using the method suggested by Cornell *et al* (2002).

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABBREVIATIONS

PGA	Peak Ground Acceleration
RC	Reinforced Concrete
IS	Indian Standard
FEMA	Federal Emergency Management Agency
HDMR	High Dimensional Model Representation
ATC	Applied Technology Council
MCS	Monte Carlo Simulation
LHS	Latin Hypercube Sampling
EDP	Engineering Demand Parameter
IO	Immediate Occupancy
LS	Life Safety
CP	Collapse Prevention

NOTATIONS

a & b	Regression constant
σ	Standard deviation
µ	Mean value
f_c	Characteristic strength of concrete
E_c	Young's Modulus of Concrete
f_y	Yield Strength of Steel
COV	Coefficient of variation
g	Acceleration due to gravity

1
INTRODUCTION

CHAPTER-1

INTRODUCTION

1.1 FRAGILITY CURVES

Former to an earthquake, vulnerability evaluations of buildings are normally carried out for judging the requirement for strengthening vital facilities and buildings against later earthquakes. The best way to accomplish such assessments is Fragility curves. Fragility curves epitomise the conditional probability that a response of a particular structure may exceed the performance limit at a given ground motion intensity. These curves are valuable tools for the valuation of probability of structural damage due to earthquakes as a function of ground motion indices otherwise design parameters.

Fragility curves - show the probability of failure verse us peak ground acceleration. Fig 1.1 shows a typical fragility curve with PGA along the x-axis and probability of failure along y-axis. A point in the curve represents the probability of exceedance of the damage parameter, which can be lateral drift, storey drift, base shear etc., over the limiting value mentioned, at a given ground motion intensity parameter.

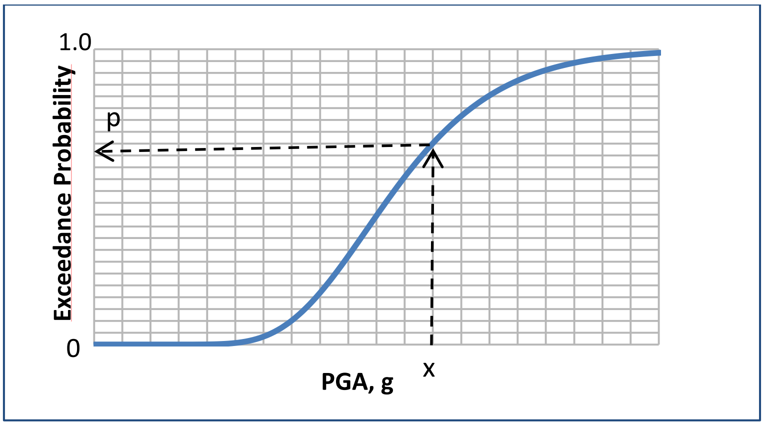

Figure 1.1 Typical Fragility Curve showing PGA vs. Probability of exceedance

For a PGA of say ‘x’, the fragility curve gives the corresponding probability of exceedance of limiting damage parameter as ‘p%’. It can be interpreted as if 100 earthquakes of PGA ‘x’ occur, ‘p’ times the damage parameter will exceed the limiting value for which the fragility curve is plotted.

Earthquake engineering has evolved over the years and it is now moving towards Performance-based methods rather than the existing force based approaches. The concept of design for the force is now changing towards design for a particular performance objective required by the stake holders. The engineers are familiar with the performance measures such as strain, drift, acceleration etc. but the stakeholders may be more familiar with cost involved for design making. To convert the performance of a particular structure to a format involving repair cost in a systematic way there are many factors to consider. Probabilistic seismic hazard (Probability of earthquake with certain intensity), Response analysis (Exceedance probability of a demand parameter of structure for a specific intensity measure of earthquake), Damage analysis (Damage of structure given a particular demand parameter), Loss analysis (Cost involved for a particular damage) are the four components of the a performance based earthquake engineering frame work introduced by Moehle and Deierlein (2004). Figure 1.2 shows the components involved in performance-based earthquake engineering frame work. The second component in this frame work is the development of fragility curves.

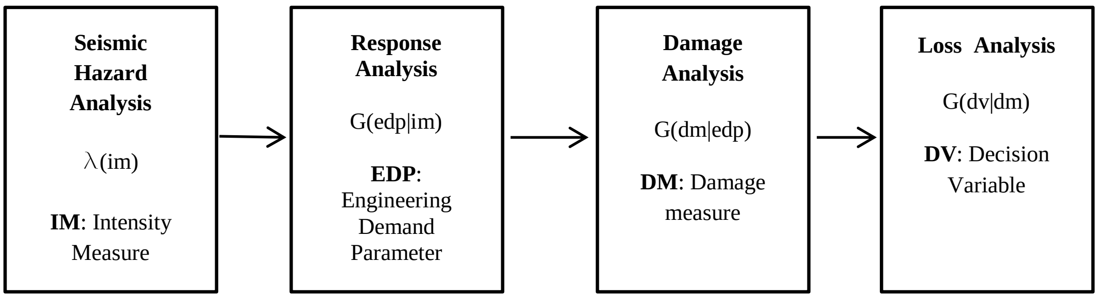

Figure 1.2 Performance-based earthquake engineering framework

Fig 1.3 shows typical fragility curves for different limiting values for damage parameter. The intensity measure here is the spectral displacement of the earthquake. As the limiting value increases the curve shifts towards right and becomes more flat. From the figure it can be seen that at weak shaking the probability of exceedance for the limit state corresponding to slight damage is high. For strong earthquakes probability of exceedance is 100% for the first curve, which means slight damage is sure, moderate and extensive damages are likely to occur. But probability that complete damage will occur is low. Regions of various damage states such as slight, moderate, Extensive and complete damages are marked between each fragility curves. With the severity of damage, the parameter defining the limit state of damage increases, and the exceedance probability decreases.

For an earthquake with spectral intensity corresponding to weak shaking, the exceedance probability for the slight damage is quite high and the levels defined by higher damage states such as moderate, Extensive, complete are very negligible. Whereas if there is an earthquake of strong intensity the building is more likely to be crossed the damage states of slight and moderate. The exceedance probability for the extensive damage state is more than that of complete damage state.

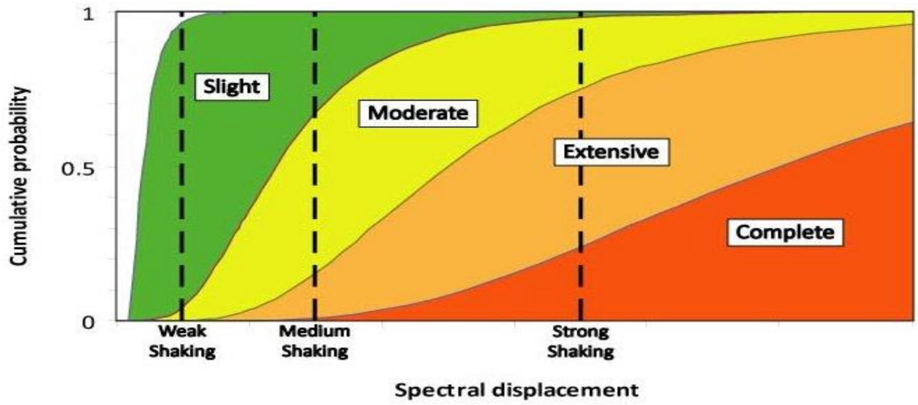

Figure 1.3 Fragility curves for 4 different limit states (Tobas and Lobo 2008)

1.2 METHODS OF DEVELOPMENTS OF FRGILITY CURVES

Conventional methods for computing building fragilities are:

- Monte Carlo simulation (MCS)
- Cornell *et al.* (2002)
- Response Surface Method
- ATC-63

The Monte Carlo simulation is a statistical simulation procedure that provides reasonably accurate solutions to problems expressed mathematically. It employs a sequence of random numbers to execute the simulation. This tactic requires fairly large number of models to obtain a satisfactorily reliable evaluation of fragilities which makes it computationally expensive and also time consuming.

The Latin hypercube sampling method is a competent sampling technique which makes sure that the complete ranges of input variables are sampled. Metamodels are a more advanced approach for fragility analysis, which is a statistical estimate of the complex and implicit occurrences, expressed by the use of response surface methods. Response is evaluated in a closed-form function of input variables thus reducing the computational effort. One of the most common metamodel used is the response surface methodology. This methodology states not simply to the use of a response surface as a multivariate function, but also to the determination of polynomial coefficients. A response surface equation is simply a polynomial representation to a data set. The process of obtaining the polynomial is more accurate by using a large data set.

Cornell *et al.* (2002) proposed a methodology to characterize the fragility function as the probability of exceedance of the designated Engineering Demand Parameter (EDP) for a selected physical limit state (DS) for a particular ground motion intensity quota (IM). Fragility curve reaching a specified damage state or more is represented as a function of that particular demand. More detailed explanation of this method is given in Chapter 2.

1.3 MOTIVATION OF THE PRESENT STUDY

Generation of fragility curves in conventional methods involves development of large number of computational models that represent the inherent variation in the material properties of particular building type and its earthquake time history analyses to obtain an accurate and reliable estimate of the probability of exceedance of the chosen damage parameter. There are many Response surface methods available in the literature that is capable of representing the limit state surface depending on the problem type. High Dimensional Model Representation (HDMR) method (more explanation of this method is provided in Chapter 2) is a type of response surface method that can express input-output relations of complex computational models. This input-output relation can reduce the number of iterations of expensive computations especially in problems like fragility curve development. Unnikrishnan *et al.* (2012) applied this technique in fragility evaluation for the first time and demonstrated its computational efficiency compared to computationally intensive Monte Carlo method. This study is an attempt to develop fragility function of an RC frame using HDMR response surface method and to verify its comparison with that developed using another approximate method proposed by Cornell *et al.* (2003).

1.4 OBJECTIVES OF THE STUDY

Based on the preceding discussions, the main objectives of the current study has been quoted as follows

- Develop fragility function using high dimensional model representation (HDMR) response surface method for a typical RC frame.
- Develop fragility function as per the method suggested by Cornell *et al.* (2002) for the same frame.
- Study of Fragility curves developed using HDMR and its computational efficiency with reference to the one using the method suggested by Cornell *et al* (2002).

1.5 SCOPE OF WORK

The present study is limited a single RC plane frame without shear wall, basement, and plinth beam. The stiffness and strength of Infill walls is not considered. The Soil structure interface effects are not taken into account in the study. The flexibility of floor diaphragms is ignored and is considered as stiff diaphragm. The column bases are assumed to be fixed in the study. OpenSees platform (McKenna *et al.*, 2000) is used in the present study to implement the simulation of large number of computational models for fragility evaluation. The nonlinearity in the material properties are modeled using fiber models available in OpenSees platform.

1.6 ORGANISATION OF THE THESIS

Following this introductory chapter, the organisation of further Chapters is done as explained below.

i. A review of literature conducted on Fragility Evaluation of buildings, and use of HDMR in various fields are provided in Chapter 2.
ii. Development of fragility curve for the RC frame using HDMR is explained in Chapter 3.
iii. Cornell's method of fragility function development is explained in Chapter 4
iv. Finally in Chapter 5, discussion of results, limitations of the work and future scope of this study is dealt with.

2

REVIEW OF LITERATURE

CHAPTER 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

As the present study deals with fragility curve development, a detailed literature review has been conducted on various conventional methods involved in fragility curve development like Monte Carlo Simulation, method proposed by Cornell *et al.* (2002), Latin Hypercube Sampling, Response Surface Method etc.. In the later part a general review of High Dimensional Model Representation technique and its application in Fragility curve evaluation are discussed.

2.2 FRAGILITY ANALYSIS

A reinforced concrete 25 story moment resisting structure with three-bays was considered by Tantala and Deodatis (2002). Fragility curves are developed for wide series of ground motion intensities. Time histories demonstrated by stochastic procedures were used. The nonlinear analysis was done by taking into account the P-Δ effects and ignoring soil-structure collaboration. The nonlinearity in the material properties in the model was achieved with nonlinear rotational springs. Monte Carlo simulation method is used for the simulation of the ground motion. The simulation for durations of strong ground motions was done at 2, 7 and 12 seconds labels to observe the effects. Stochastic process was adopted for modelling. The analyses were done by using DRAIN-2D as a dynamic analysis with inelastic time histories data. The arbitrary material strengths for every beam and column were simulated using Latin Hypercube sampling.

Schotanus (2002) applied a general and urbane method for seismic fragility analysis of systems previously proposed by Veneziano *et.al* (1983) to a reinforced concrete frame. Response surface was used to switch the capacity part in an analytical limit-state function (g- function), with a categorical functional relationship which fits a second order polynomial, and is used as input for SORM analysis. Such an explicit function highly reduces the number of costly numerical analyses needed compared to classical methods that determine the failure domain.

A numerical approach was suggested by Franchin *et al.* (2004) for seismic reliability problems that can be applied in the assessment of an RC frame structure. The procedure determined a response surface, characterised by a statistical model of the mixed type, to represent the seismic capacity in an analytical limit-state function. FORM investigation was used to calculate the fragility function of the system, with the constructed empirical limit-state function as input. The application focused on the clarification of implementation issues, and confirmed the versatility of the method in realistic problems.

Murat and Polat *et al.* (2006) established the fragility curves for mid-rise RC frame buildings located in Istanbul, which were designed according to the 1975 version of the Turkish seismic design code that was based on numerical replication with respect to the number of stories of the buildings. Buildings of 3, 5 and 7 story were designed according to the Turkish seismic design code. To investigate the effect due to the number of stories of the building on fragility constraints, regression analysis was carried out between fragility parameters and the number of stories of the building. It was found that fragility parameters change widely due to the number of stories of the building. Finally, the maximum allowable inter-story drift ratio and spectral displacement values that satisfy the ‘immediate occupancy’ and ‘collapse prevention’ performance level requirements were estimated using obtained fragility curves and statistical methods,.

Craig *et al.* (2007) labelled the results of research to develop a methodology to rapidly assess the fragility of structures and geostructures over a specified region by developing a procedure based on the use of computationally efficient metamodels to represent the overall structural conduct of the collection. In particular, response surface metamodels were developed using a Design of trials approach to select the most influential parameters. Monte Carlo simulation was carried out using probability distributions for the parameters that are distinctive of the target collection of structures or geostructures, and the fragility of the collection is estimated from the computed responses.

Ellingwood (2001) assessed the earthquake risk calculation of the building by applying the probabilistic risk investigation tools. The work concentrated on the 3-probability grounded codified designed and reliability based condition valuation of existing structures. Weld connected steel frames were designed. To study the performance a nonlinear dynamic analysis was done in the prominence of inherent randomness and modelling uncertainty in the performance of the structures through fragility study. The ground motion from California strong ground motion network was considered for the seismic hazard investigation.

Ji *et al.* (2007) presented an analytical agenda and sample application for the seismic fragility assessment of reinforced-concrete tall buildings. A simple lumped-parameter prototype was presented for an existing skyscraper structure with dual core walls and a reinforced concrete frame. The exactness of the individual components of this model was compared with the estimates of more detailed analytical models and sample fragility curves were presented. The proposed framework was mostly applicable for developing fragility relationships for high-rise building structures with frames and cores or walls.

Guneyisi and Altay (2008) detected the behaviour of already existing R/C office structures through fragility plots considering the circumstances as before and after retrofitted by liquid viscous (VS) dampers The R/C building was modelled as a 3-dimensional analytical model and was established in ETABS version 7.2 Structural Analysis Program for the analysis. The seismic reaction of the buildings was obtained by the nonlinear dynamic analysis with pushover investigation by IDARC version 6.1 packages. The fragility curves were made for four damage conditions which are slight, moderate, major, and collapse states. The fragility curve produced for the structure are resolved that with the aid of retrofitting the chances of failure on building can be minimized.

Samoah (2012**)** studied the fragility performance of non-ductile RC frames in low and medium seismic zones. The structural capability of the structures was studied by inelastic pushover analysis and seismic demand is investigated by inelastic time history analysis followed by evaluation of fragility curves. Three non-ductile RC frames symmetrical and regular in plan and elevation were studied which were designed rendering to BS 8110 (1985). The buildings taken into account were a 3-storey 3-bay, a 4-storey 2-bay and a 6-storey 3 bay buildings to acquire an appreciable result. A macro-element package IDARC2D (1996) was established as the inelastic static and dynamic analysis of non-ductile RC frames. The modelling and analysis for the non-ductile RC frame buildings are done adequately on the basis of their structural properties.

Response Surface (RS) models were used by Buratti *et al.* (2010) with arbitrary block effects to evaluate seismic fragility curves in a rough way with good computational efficacy. The RS models were regulated through numerical data obtained by non-linear incremental dynamic analyses performed using various sets of ground-motions, strength allocations in frame elements, and values of the arbitrary variables taken to describe the ambiguities in the structural behaviour. The work was largely focused on the problem of obtaining a reasonable compromise between result soundness and computational effort. With reference to a three storey frame structure, a series of numerical examinations were presented. Different simulation tactics, defined following the theory of Design of Experiments (DOE), and abridged polynomial RS models were employed. The fragility curves obtained by different approaches were compared, using the results from full Monte Carlo simulation as the reference solution.

The fragility analysis for an irregular RC frame under bi-directional earthquake burdening has premeditated by Jeong and Elnashai (2006). For the contemplation of the anomalies in structure, the torsion and bidirectional response were employed as 3-Dimensional structural response attributes to represent the damage states of the structure irregularities is bestowed through a reference derivation. A three story RC plane frame was taken which is asymmetric in plan with thickness of slab 150 mm and depth of beam 500 mm to learn the damage assessments. Generation of fragility curves were by computing the damage measure by spatial damage index with statistical manipulation methods and lognormal distributions for response variables. Earthquake records consist of 2 orthogonal components (Longitudinal and Transverse) of horizontal accelerations and were modified from the natural records to be compatible with a smooth code spectrum. PGAs were taken from a range of 0.05 to 0.4g with a step of 0.05g. Planar decomposition method was used for accurate damage assessment of buildings exhibiting torsion, where the building was decomposed into planar frame and analysed. The parameters such as roof displacement, inter-story drift or a damage index were found out from numerical simulation results. For the planar frames the total damage index was calculated from the backbone envelope curve as a combination of damage due to in-plane monotonic shift and strength reduction.

Fragility curves were developed by Bakhshi and Asadi (2012) in order to assess various probability parameters such as, PGA, importance factor (I) and typical over-strength and global ductility capacity (R). These illustrations were utilized to show when a coefficient or a number of parameters were used to improve the performance capacity of a structure. The results showing that by increasing the R, the probability of damage exceedance is dwindled; however, an increase in I for hospital buildings versus office buildings, cannot pledge a decrease in the chances of damage exceedance. The PGA randomness outcomes revealed that, considering PGA uncertainty does not mean that the probability of damage exceedance will be increased in general cases.

Towashiraporn (2004) suggested an alternative methodology for carrying out the structural simulation. The use of Response Surface Methodology in connection with the Monte Carlo simulations abridges the process of fragility computation. The usefulness of the response surface metamodels becomes more apparent for promptly deriving fragility curves for buildings in a portfolio. After metamodels applicable for building inventory in a geographical expanse are developed, they can be used for analysis of any portfolio of interest, located within the same region. The ability for quick estimation of fragility relation for a discrete building in a target portfolio was a noteworthy step toward more accurate seismic loss estimation.

Cornell *et al.* (2002) investigated a recognized probabilistic framework for seismic design and assessment of structures and its solicitation to steel moment-resisting frame buildings based on the 2000 SAC, Federal Emergency Management Agency (FEMA) steel moment frame guidelines. The framework was based on recognizing a performance objective expressed as the probability of exceedance for a specified performance level, that related to ‘‘demand’’ and ‘‘capacity’’ of which were described by the nonlinear dynamic displacements of the structure. To describe the randomness and improbability in the structural demand given the ground motion level and the structural capacity probabilistic model distributions were used. A customary probabilistic tool, the total probability theorem was used to convolve the probability distributions for demand, capacity, and ground motion intensity hazard. An analytical

expression was delivered for the probability of exceeding the performance level as the primary product of the development of framework. Consideration of uncertainty in the probabilistic modelling of demand and capacity allowed for the definition of confidence statements for the likelihood performance objective being achieved. This method is termed as Cornell's method in this study.
As this method is used in this study, this method is explained in detail.

2.2.1 Cornell's Method in Detail

According to this technique a fragility function denotes the probability of exceedance of the nominated Engineering Demand Parameter (EDP) for a selected structural limit state (DS) for a specific ground motion intensity measure (IM). These curves are cumulative probability distributions that specify the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular ultimatum. Fragility curve damaged to a given damage state or a more severe one, as a function of a particular demand.Fragility curve can be obtained for each damage state and can be conveyed in closed form as using Eq.2.1

$$P(C\_D \leq 0|IM) = \Phi\left(\frac{\ln\frac{S_d}{S_c}}{\sqrt{\beta_{d|IM}^2+\beta_c^2}}\right) \tag{2.1}$$

Where, C is the drift capacity, D is the drift demand, S_d is the median of the demand and S_c is the median of the chosen damage state (DS). $\beta_{d/IM}$ and β_c are dispersion in the intensity measure and capacities respectively. Eq. 2.1 can be redrafted as Eq. 2.2 for component fragilities (Nielson, 2005) as,

$$P(DS|IM) = \Phi\left(\frac{\ln IM - \ln IM_m}{\beta_{comp}}\right) \tag{2.2}$$

Where, $IM_m = \exp\left(\frac{\ln S_c - \ln a}{b}\right)$, a and b are the regression coefficients of the probabilistic Seismic Demand Model (PSDM) and the dispersion component, β_{comp} is given by

$$\beta_{comp} = \sqrt{\frac{\beta_{d|IM}^2+\beta_c^2}{b}} \tag{2.3}$$

$$\beta_{d|IM} \cong \sqrt{\frac{\sum(\ln(d_i)-\ln(aIM^b))^2}{N-2}} \tag{2.3b}$$

It has been suggested by Cornell *et al.* (2002) that the estimate of the average engineering demand parameter (EDP) can be represented by a power law model, which is called a Probabilistic Seismic Demand Model (PSDM) as specified in Eq. 2.4.

$$\widehat{EDP} = a(IM)^b \tag{2.4}$$

In this study, roof drift is taken as the engineering damage parameter (EDP) and peak ground acceleration (PGA) as the intensity measure (IM).

2.3 HIGH DIMENSIONAL MODEL REPRESENTATION

Two types of HDMRs were demonstrated by Rabitz H *et al.* (1999): ANOVA-HDMR which is the same as the analysis of variance (ANOVA) decomposition used in statistics, and cut-HDMR which was be shown to be computationally more efficient than the ANOVA decomposition. Application of the HDMR tools affectedly reduced the computational struggle needed in representing the input–output relationships of a physical system.

Alis and Rabitz (2001) illustrated the application of Random-sample High Dimensional Model Representation (RS-HDMR) by captivating two examples, Sensitivity analysis and an inverse problem in dynamical systems. RS-HDMR was shown to be computationally very efficient to compute sensitivity catalogues with high accuracy, and as such this method can be used to construct a data-generating dynamical system.

An illustration of High Dimensional Model Representation was carried out by Li *et al.* (2001) in financial instruments whose value derives from the value of other merchandises. They also suggested the application of this method in industrial plant or economic system performance under conditions of constrained resources, and other similar mathematical problems.

Rajib *et al.* (2009) proposed a new computational tool for forecasting failure probability of structural/mechanical systems subject to random loads, material properties, and geometry. The method involved high-dimensional model representation (HDMR) that facilitates lower-dimensional approximation of the original high dimensional implicit limit state/performance function, response surface generation of HDMR constituent functions and Monte Carlo simulation. Results of nine numerical examples which involved mathematical functions and structural mechanics problems showed that the proposed method provides accurate and computationally efficient estimates of the probability of failure.

2.3.1 HDMR Method in General

HDMR is a method reputable for the expression of input-output relations of complex, computationally arduous models in terms of hierarchical interrelated function expansions. A reduced and accurate metamodel of the original complex and nonlinear system can be obtained by the use of HDMR approach. The uncertainty analysis of the computationally burdensome system or model can then be well approximated by the use of Monte Carlo simulation of the corresponding condensed model, at a much lower computational cost without negotiating the accuracy. The input variables can be the specified initial and boundary conditions, parameters and functions involved in the model, or field control variables and the output variables would be the solutions to the model or observed system response.

The N-dimensional vector $x= \{x_1, x_2, \ldots\ldots, x_N\}$ with N ranging up to the order of $10^2 - 10^3$ or more; denote the input variables of the model or system under consideration, and the output variable is given by *f(x).* As the effect of the input variables on the output variable can be independent and/or cooperative, HDMR expresses the output *f(x)* as a hierarchical correlated function expansion in terms of the input variables as:

$$f(x) = f_0 + \sum_{i=1}^{N} f_i(x_i) + \sum_{1<i_1<i_2<N} f_{i_1 i_2}(x_{i_1}, x_{i_2}) + \sum_{1<i_1<i_2<i_3<N} f_{i_1 i_2 i_3}(x_{i_1}, x_{i_2}, x_{i_3}) + \ldots + f_{i_1 i_2 \ldots i_N}(x_{i_1}, x_{i_2}, \ldots, x_{i_N}) \tag{2.5}$$

Where,

- f_0 symbolizes the response *f(x)* at a selected reference point generally the mean point, which is a constant.
- The function $f_i(x_i)$ is the first order term representing the individual contribution of the variable x_i upon the output.

- The function $f_{i_1 i_2}(x_{i_1}, x_{i_2})$ is the second order term, which describes the cooperative effects of the variables $x_{i_1}$and x_{i_2} together upon the output.
- The higher order terms gives the collaborative effects of increasing numbers of input variables acting mutually to influence the output.
- The last term $f_{i_1 i_2 \ldots i_N}(x_{i_1}, x_{i_2}, \ldots, x_{i_N})$ contains any residual dependence of all the input variables locked together in a cooperative way to influence the output *f(x).*

There are two particular HDMR expansions reliant on the method adopted to determine the component functions in Equation 2.5; analysis of variance –HDMR (ANOVA-HDMR) and Cut-HDMR. ANOVA-HDMR is used for measuring the contributions of the variance of individual component functions on the overall variance of the output. In Cut-HDMR, a reference argument is looked-for to determine HDMR functions. Cut-HDMR gives exact result along the lines, planes, volumes etc., through and around the reference point, in the other case ANOVA-HDMR returns a good average value universally.

In Cut-HDMR a reference point $c = \{c_1, c_2, c_3, \ldots \ldots \ldots c_N\}$ is first demarcated in the variable space which is mostly at the mean values of the input variables. The expansion functions are determined by estimating the input-output response of the system relative to the reference point c along associated lines, surfaces, subvolumes, etc. in the input variable space. The component functions in Equation (2.5) will be reduced to the following relationships.

$$f_0 = f(c) \tag{2.6}$$

$$f_i(x_i) = f(x_i, c^i) - f_0 \tag{2.7}$$

$$f_{i_1 i_2}(x_{i_1}, x_{i_2}) = f(x_{i_1}, x_{i_2}, c^{i_1 i_2}) - f_{i_1}(x_{i_1}) - f_{i_2}(x_{i_2}) - f_0 \tag{2.8}$$

In Equation (2.7) $f(x_i, c^i) = f(c_1, c_2, \ldots . c_{i_1 - 1}, x_i, c_{i_1 + 1}, \ldots \ldots c_N)$ denotes that all the input variables except x_i are at their reference points, and the function given by $f(x_{i_1}, x_{i_2}, c^{i_1 i_2}) = f(c_1, c_2, \ldots . c_{i_1 - 1}, x_{i_1}, c_{i_1 + 1}, \ldots \ldots c_{i_2 - 1}, x_{i_2}, c_{i_2 + 1}, \ldots . . c_N)$ denotes that the input variables except (x_{i_1}, x_{i_2}) are at their reference point values. The f_0

term is the output response of the system at the reference point c. The higher order terms are evaluated as cuts in the input variable space around the reference point. The process of subtracting off the lower order expansion functions removes their dependency, to provide a unique contribution from the new expansion function. Although second-order HDMR is more accurate than first-order HDMR, it is reasonably expensive computationally when compared to first-order. In this study first-order Cut-HDMR is used.

2.4 HDMR TECHNIQUE FOR SEISMIC FRAGILITY EVALUATION

Unnikrishnan *et al.* (2012) applied HDMR-based response surface method for the generation of the seismic fragility curves for an RC frame structure for the first time. Advantage of using this method was the reduction in computational effort and time when compared with other existing methods. Two simple case studies were taken – Spring-Mass system and RC plane frame. The results were validated using conventional methods like LHS (Latin hypercube sampling)

2.4.1 Methodology of HDMR in Fragility Evaluation

The principal step in the computation of the seismic fragility curves using HDMR is the definition of the input and output variables. Seismic intensity parameter is also defined and used as an input variable. To recognize the damage states, depending upon the type of structure being considered, Base Shear, Maximum Roof displacement, Peak interstorey drift, Damage indices, Ductility ratio and Energy dissipation capacity can be used.

In the above study by Unnikrishnan *et al.* (2012) two cases namely Spring-Mass system and RC plane frame was considered and fragility curves were obtained. In the Spring-Mass system stiffness, Mass and Sa (spectral acceleration) were considered as input variables. For the RC plane frame Compressive strength and Modulus of elasticity of concrete, Yield strength of steel and Sa were the input variables.

Various combinations of input variables were generated, which represents different earthquake-structure circumstances and the sampling points of the HDMR.

Computational seismic analysis was performed on those structural models using Scaled earthquake records (20 in number) as the loading inputs. Mean and standard deviation of the response from the analysis using 20 earthquake records for each combination of input variables were calculated. Metamodels, which are polynomial functions representing the mean and standard deviation of the responses, were framed by applying HDMR technique. Metamodels are polynomial functions representing the mean and standard deviation. The two metamodels are combined to form the overall metamodel as specified in Equation (2.9).

$$y = y_{\mu} + N[0, y_{\sigma}] \quad (2.9)$$

Where y_{μ} and y_{σ} are the mean and standard deviation metamodels of the responses respectively, N is the normal distribution. Monte Carlo techniques with a large number of simulations were carried out on the overall metamodel using probability density functions for the input variables. The probability of the chosen response exceeding certain damage limit states were taken out from the simulation outcomes. This probability value corresponds to one specific earthquake intensity and characterizes one point in the fragility curve. The process was repeated for different levels of earthquake intensity and fragility curve is plotted.

2.5 SUMMARY

The review of the study indicates that there have been numerous research efforts found on the seismic behaviour of RC buildings, Fragility analysis and nonlinear analysis. Also with regard to use of High Dimensional Model Representation in Fragility Analysis, there were very few studies conducted. The main objective is to study comparison of fragility curve developed by two approximate methods such as recently introduced HDMR method and Cornell’s method. The first part the present study will attempt to conduct Fragility analysis using HDMR. In the second part, Cornell’s method will be used for the same, and is compared with Fragility curve obtained using HDMR.

In the present study the work done by Unnikrishnan *et al.* is implemented in OpenSees platform with a different set of scaled earthquake records (44 in number), for the 6 storey 3 bay RC plane frame they have used and the fragility curves are plotted.

3

DEVELOPMENT OF FRAGILITY CURVES USING HDMR

CHAPTER 3

DEVELOPMENT OF FRAGILITY CURVES USING HDMR

3.1 GENERAL

This chapter is based on the development of the fragility curves using HDMR technique. The frame considered, uncertainties in material properties and ground motion data, limit states and finally fragility evaluation is detailed here. For the study, the peak ground acceleration is taken as the seismic intensity measure and the roof displacement is considered as the engineering demand parameter for generation of fragility curves for different performance levels

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3.2 DESCRIPTION OF THE STRUCTURE

In this study an RC frame having six stories and three bays is considered. The frame is designed according to IS 456-2000 using M20 concrete and Fe415 steel. The details of the building elevation and reinforcement details of beams and columns are shown in Figure 3.1. The frame is having a storey height of 3.6 m and bay width of 5 m. The base of the frame is considered as fixed. In addition to self-weights of beams and columns, the dead load (due to slabs and infill walls) and live loads prescribed for all beams are 35 kN/m and 15 kN/m respectively.

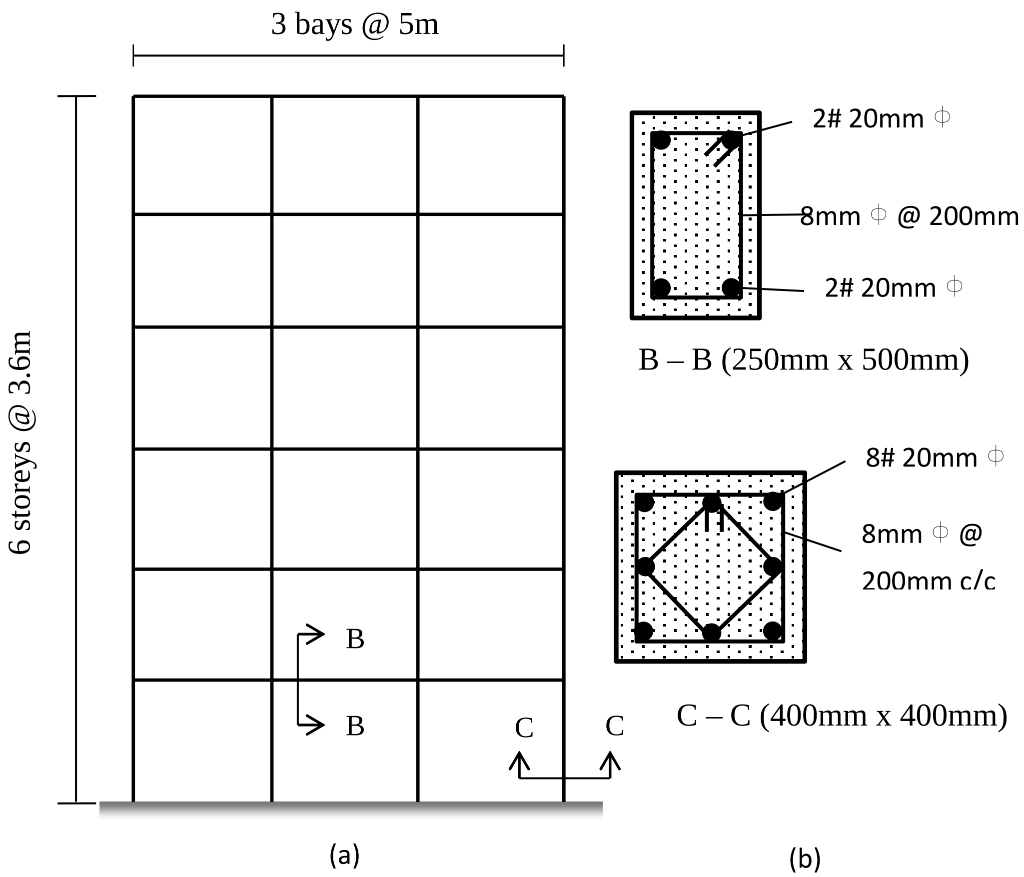

Figure 3.1(a) Elevation of the RC Frame and (b) Reinforcement details of beams and columns

3.3 MODELLING OF RC MEMBERS FOR NONLINEAR DYNAMIC ANALYSIS

A modified model of Mander *et al.*(1988b) is used to define section stress strain relation. The modelling of the structure is done in OpenSees (Open System for Earthquake Engineering Simulation) which is an object oriented open-source software framework used to model structural and geotechnical systems and simulate their earthquake response. OpenSees is primarily written in C++ and uses some FORTRAN and C numerical libraries for linear equation solving, and material and element customs. OpenSees has progressive capabilities for modelling and analysing the nonlinear response of systems using a wide range of material models, elements, and solution algorithms. It is an open-source; the website provides information about the software architecture, access to the source code, and the development process.

The open-source movement allows earthquake engineering researchers and users to build upon each other's accomplishments using OpenSees as community-based software. Another advantage of using OpenSees is that modelling frames with different sets of input variables can be done with the help of loops, whereas in conventional softwares each case will have to be modelled separately.

Concrete behaviour is modelled by a uniaxial Kent‑Scott‑Park model with degrading, linear, unloading/reloading stiffness no tensile strength. For the confined concrete, the strain and strength values are increased according to the formulae developed by Mander *et.al* (1988). Steel behaviour is represented by a uniaxial Giuffre‑Menegotto‑Pinto model. Fiber Section modelling of element is done according to Spacone *et.al*, 1996. A proportional damping model called Rayleigh Damping is used in this study. In Rayleigh Damping it is assumed that the damping matrix is proportional to the mass and stiffness matrices.

3.4 MODELLING OF UNCERTAINTIES

The uncertainties in the material properties are unavoidable in reality. The uncertainty in the material properties are modelled by considering the parameters defining the materials as random variables. Some of studies (Rajeev and Tesfamariam, 1999, Únnikrishnan *et al.,* 2012) conducted shows that the major random variables to be considered in fragility study are compressive strength of concrete (f_c), yield strength of steel (f_y) and Young's modulus of concrete (E_c). The distribution characteristics and the values used in this work is taken from Ranganathan (1990) and these are specified in Table 3.1.

Table 3.1 Statistics of Random Variables'

Material	Variable	Mean (Mpa)	COV (%)	Distribution
Concrete	f_c	19.54	21.0	Normal
Concrete	E_c	34100	20.6	Normal
Steel	f_y	469	10	Normal

3.5 EARTHQUAKE GROUND MOTIONS

Randomness in ground motion is taken into account by using 44 scaled earthquake records. The ground motion data is taken from the work done by Haselton *et al.(*2007). In this research and related work, a general far-field ground motion set was established for use in structural analyses and performance valuation. 22 pairs of motions that cover the FEMA P695 (ATC-63) far-field ground motion set details of which are given in Table 3.2. This 22 pairs (44 components) of ground motions are used in this study.

Table 3.2 Details of Earthquake records considered as per FEMA P695 (ATC-63)

Sl No	Magnitude	Year	Event	Fault type	Station name	Vs_30 (m/s)
1	6.7	1994	Northridge	Blind thrust	Beverly Hills - 14145 Mulhol	356
2	6.7	1994	Northridge	Blind thrust	Canyon Country - W Lost Cany	309
3	7.1	1999	Duzce, Turkey	Strike-slip	Bolu	326
4	7.1	1999	Hector Mine	Strike-slip	Hector	685
5	6.5	1979	Imperial Valley	Strike-slip	Delta	275
6	6.5	1979	Imperial Valley	Strike-slip	El Centro Array #11	196
7	6.9	1995	Kobe, Japan	Strike-slip	Nishi-Akashi	609
8	6.9	1995	Kobe, Japan	Strike-slip	Shin-Osaka	256
9	7.5	1999	Kocaeli, Turkey	Strike-slip	Duzce	276
10	7.5	1999	Kocaeli, Turkey	Strike-slip	Arcelik	523
11	7.3	1992	Landers	Strike-slip	Yermo Fire Station	354
12	7.3	1992	Landers	Strike-slip	Coolwater	271
13	6.9	1989	Loma Prieta	Strike-slip	Capitola	289
14	6.9	1989	Loma Prieta	Strike-slip	Gilroy Array #3	350
15	7.4	1990	Manjil, Iran	Strike-slip	Abbar	724
16	6.5	1987	Superstition Hills	Strike-slip	El Centro Imp. Co. Cent	192
17	6.5	1987	Superstition Hills	Strike-slip	Poe Road (temp)	208
18	7	1992	Cape Mendocino	Thrust	Rio Dell Overpass - FF	312
19	7.6	1999	Chi-Chi, Taiwan	Thrust	CHY101	259
20	7.6	1999	Chi-Chi, Taiwan	Thrust	TCU045	705
21	6.6	1971	San Fernando	Thrust	LA - Hollywood Stor FF	316
22	6.5	1976	Friuli, Italy	Thrust-part blind	Tolmezzo	425

3.6 FAILURE CRITERIA AND PERFORMANCE LIMITS

In this study roof displacement as often preferred by many researchers is taken as the failure criteria because of the ease and convenience allied with its estimation. The limit states considered are according to Federal Emergency Management Agency (FEMA) 356. The limit states associated with various performance levels of reinforced concrete frames is given in Figure 3.2 and Table 3.3 (FEMA 356, 2000).

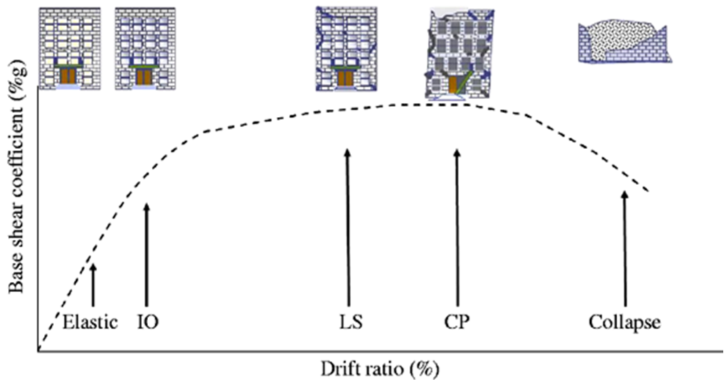

Figure 3.2 Damage states of a representative building pushed to failure (FEMA356)

Table 3.3 Limits associated with various structural performance levels

Structural performance level	**Permissible top storey drift**
Immediate Occupancy (IO)	1%
Life Safety (LS)	2%
Collapse Prevention (CP)	4%

3.7 METAMODEL FORMULATION USING HDMR

The metamodel, which is the polynomial relationship between the structural response (y) and the random variables (f_c, E_c, and f_y) that define the structure and intensity measure (PGA). To arrive at the metamodel, the computational models are developed at selected values of input variables and nonlinear dynamic analysis of each model for 22 pairs of ground motions is conducted. Three point sampling method as per the HDMR method is followed for the selection of values for each input variables. Supposing there are only two random variables, the three point sampling procedure can be explained with the grid lines as shown Fig 53824. It can be seen that the vertical line shows the random variable fy and horizontal line shows the random variable fc. The centre point, μ is the mean point, which means that the computational model is developed for both the random variables at their mean values.

The sampling is done such that only one variable is taken as arbitrary and assessed at a given time, while the other random variables are kept at their reference mean points (μ). To find the values of the random variables for the next point, the fy is kept at mean and fc is to be the point shown as $\mu + 2\ \sigma_{fc}$. Similarly the same procedure is repeated for all the grid points. If there are more than two random variables considered, the grid will have that many numbers of dimensions in space. Table 3.4 presents the values of each random variables considered for three point sampling.

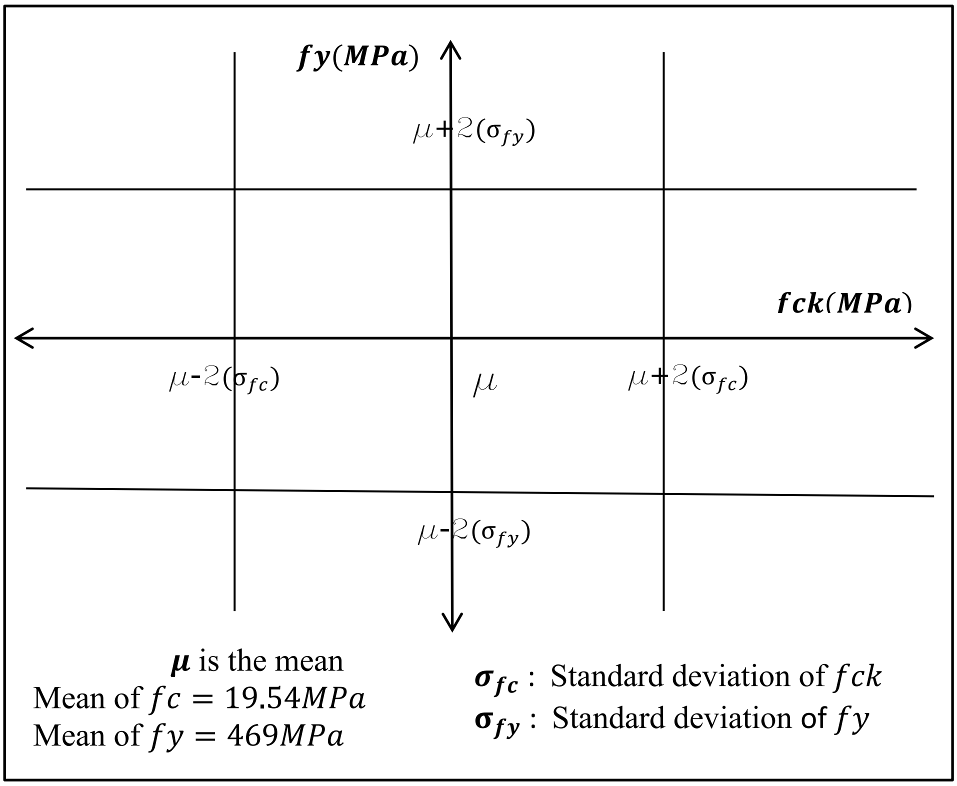

Figure 3.3 Two dimensional representation of sampling points taken in HDMR metamodel evaluation

Table 3.4 Range of input variables used in 3-point sampling in HDMR

Variable	$\mu - 2\sigma$	μ	$\mu + 2\sigma$
$f_c(Mpa)$	11.34	19.54	27.74
$E_c(Mpa)$	20050.8	34100	48149.2
$f_y(Mpa)$	375.2	469	562.8
$PGA(g)$	0.1	0.55	1.0

Nonlinear dynamic analysis of the computational models developed at the grid points (9 sets) for 44 scaled ground acceleration records are conducted. The maximum response (roof displacement) obtained from the nonlinear time history analysis at all grid points are shown in Table 3.5.

Table 3.5 Set of input variables used in 3-point sampling in HDMR

Sl No.	$f_c(Mpa)$	$E_c(Mpa)$	$f_y(Mpa)$	$PGA(g)$	y_μ **(mm)**	y_σ **(mm)**
1	19.54	34100	469	0.55	434.3231	160.8021
2	27.75	34100	469	0.55	418.0808	146.925
3	11.33	34100	469	0.55	478.5732	175.4779
4	19.54	48149.2	469	0.55	434.2786	160.7731
5	19.54	20050.8	469	0.55	434.543	160.8849
6	19.54	34100	562.8	0.55	425.3222	148.2063
7	19.54	34100	375.2	0.55	492.9165	193.2207
8	19.54	34100	469	1.0	995.9649	395.8648
9	19.54	34100	469	0.1	78.5418	24.1256

As per the HDMR equation given Equation 2.5, the metamodel for mean and standard deviation can be expressed as

$$y_\mu = f_0 + f(f_c) + f(E_c) + f(f_y) + f(PGA) \tag{3.1}$$

$$y_\sigma = f_0 + f(f_c) + f(E_c) + f(f_y) + f(PGA) \tag{3.2}$$

The functions $f(f_c), f(E_c), f(f_y)$ & $f(PGA)$ in equation 3.1 and 3.2 can be assumed as a quadratic equation of the form

$$f(z) = a_0 + a_1 z + a_2 z^2$$

The values of the coefficients a_0, a_1 and a_2 can be found out by considering the input and output combinations corresponding to the points where all the random variables except the considered random variable (z) at their mean values.

The functions for each random variable are found out as given by Equations 3.3 to 3.6. The coefficient of each random variable in each function represents the dependence of response on that variable. More the value of the coefficient more will be the dependence. In equation 3.7, the coefficients in the function of variable PGA are more than that for the others. This means that the response is more dependent on PGA rather than other variables. The metamodels for mean and standard deviation are derived as given by Equations 3.7 and 3.8

$$f(f_c) = 151.312007820296 - 11.8033430310619 f_c + 0.207760358791231 f_c^2 \quad (3.3)$$

$$f(E_c) = 0.83753417 - (3.9712450199281 * 10^5) E_c + (4.4432055339552 * 10^{10}) E_c^2 \quad (3.4)$$

$$f(f_y) = 788.892 - 3.0038342217484 f_y + 0.00281825528161811 f_y^2 \quad (3.5)$$

$$f(PGA) = -406.88744691358 + 460.231716049383 PGA + 508.297530864197 PGA^2 \quad (3.6)$$

$$y_\mu = 968.477195075817 - (11.8033430310619) f_c + (0.207760358791231) {f_c}^2 - (0.000039712450199281) E_c + (0.00000000044432055339552) {E_c}^2 - (3.0038342217484) f_y + (0.00281825528161811) {f_y}^2 + (460.231716049383) PGA + (508.297530864197) PGA^2 \quad (3.7)$$

$$y_\sigma = 403.969710111506 - (1.97044768048235)f_c + (0.00592471377853873){f_c}^2 - (0.0000132735333166179)E_c + (0.00000000000136285323675492){E_c}^2 - (1.2966012793177)f_y + (0.0011264951514132){f_y}^2 + (145.821777777778)PGA + (242.928888888889)PGA^2 \quad (3.8)$$

The overall metamodel is formulated based on Equation 2.9. Monte Carlo simulation is to be performed successively on the overall metamodel. In order to find out the number of simulations a convergence study is conducted.

3.7.1 Convergence Study

The determination of optimum number of simulations to yield a reasonably accurate probability of failure in MCS is carried out by estimating the probability of exceedance for various numbers of simulations ranging from 10 to 100000. This procedure is repeated for arbitrary PGA values such as 0.2g, 0.55g and 1g. The variation of number of simulations and the probability of exceedance for these cases are shown in Figures 3.4a, 3.4b and 3.4c. It is found that 10000 simulations is appropriate for the convergence.

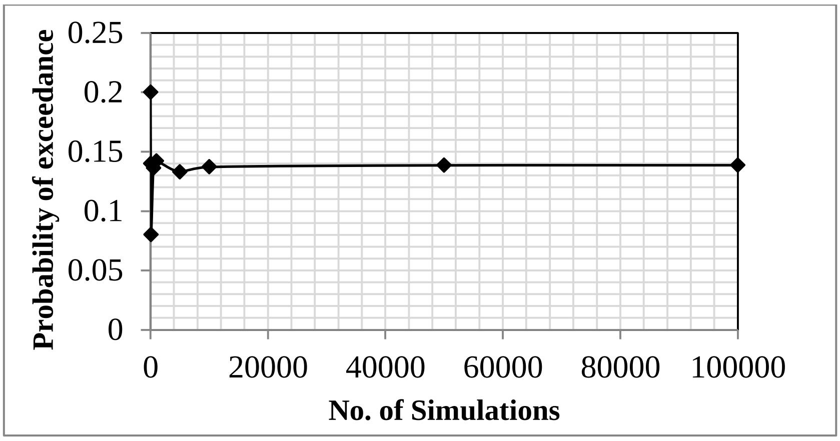

Figure 3.4a Convergence of Probability of Exceedance (PGA 0.2 at IO limit state)

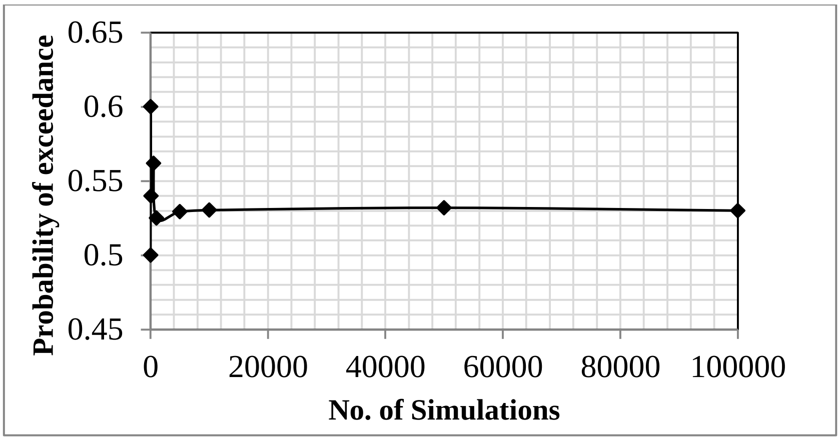

Figure 3.4b Convergence of Probability of Exceedance (PGA 0.55 at LS limit state)

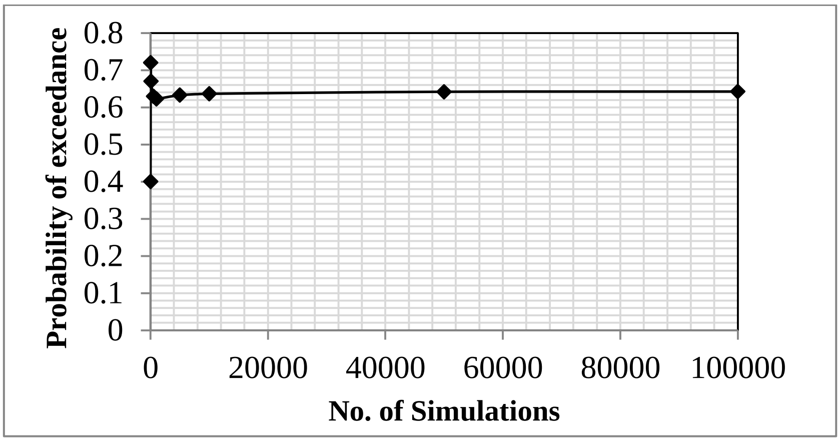

Figure 3.4c Convergence of Probability of Exceedance (PGA 1.0 at CP limit state)

3.7.2 Monte Carlo Simulation of the Metamodel

Monte Carlo simulation is performed successively on the overall metamodel by arbitrarily generating 10000 values for input variables and the corresponding response (roof displacement) is calculated. Probability of exceedance for each PGA is calculated by dividing the number of cases exceeding the limiting response value by the total number of simulations (10000). The fragility curve is obtained by joining the points represented by probability of exceedance for each PGA. This procedure is repeated for all the limit states values given in Table 3.2. The obtained Fragility curves for each limit sta7es are shown in Figure 3.5.

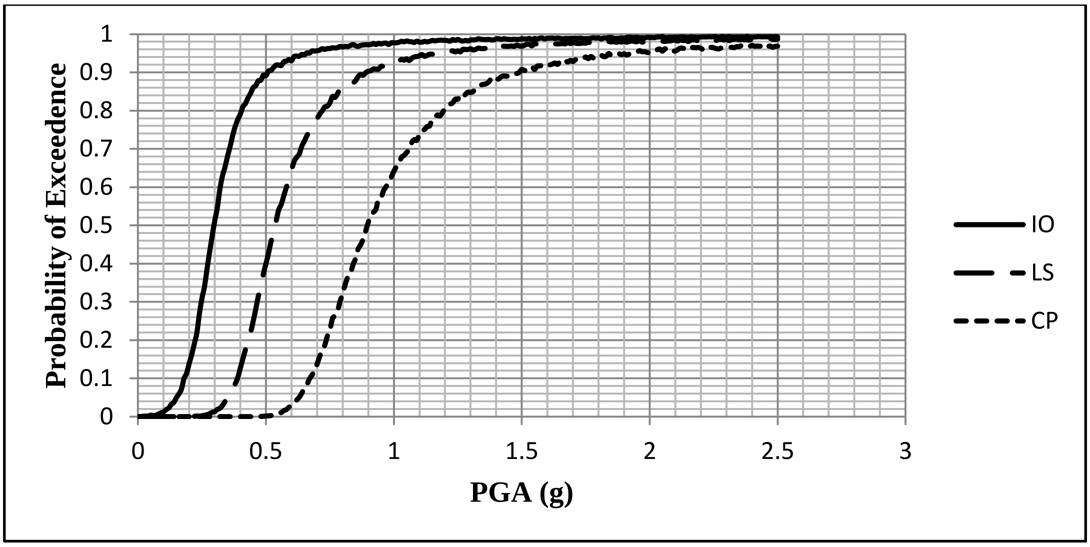

Figure 3.5 Fragility Curve using HDMR

3.7.3 Reading Fragility Curve

From the Fragility curve obtained we get probability of exceedance of the designated limit state at a particular Peak Ground Acceleration. An illustration of how to understand or read a fragility curve is explained below using Figure 3.6.

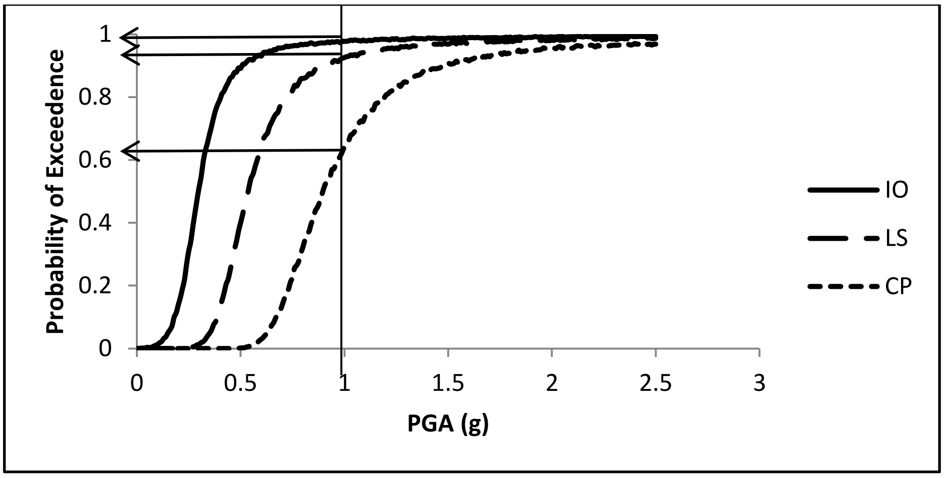

Figure 3.6 Reading Fragility Curves

In Figure 3.6 for a PGA of 1g the probability of exceedance for IO, LS and CP limit states are about 97.73%, 92.57% and 64.16% respectively as reported in Table 3.6.

Table 3.6 Probability of Exceedence for different limit states at PGA 1g

Limit States			Probability of Exceedence for PGA 1g
Immediate Occupancy	1%	216 mm	97.73%
Life Safety	2%	432 mm	92.57%
Collapse Prevention	4%	864 mm	64.16%

The inference of the Fragility curve obtained can be explained as, if an earthquake of PGA 1g occurs 97.73 % the roof displacement of the frame will exceed the limit 216 mm.

3.8 DISCUSSIONS

From the study conducted it is evident that HDMR is a computationally easy and cost effective method that can be used for fragility evaluation. The accurate method prescribed for fragility analysis is the Monte Carlo technique which takes exponentially long time to complete when compared to HDMR. In this work, to develop the fragility curve only 9 sets of input variables were taken, each of which underwent time history analysis with 44 scaled ground motion intensities, and metamodel is obtained. Each set took an approximate of 5 hours to complete. Conducting MCS on the metamodel by generating 10000 values takes only few minutes. For conducting overall Monte Carlo simulation for Fragility evaluation 10,000 to 100,000 sets of input variables are to be taken. The total computational time for all these analysis is about 2 days for HDMR. MCS for generating the same fragility curve requires about 2100 days. The computational efficiency in this problem is approximately 99%.

The metamodel is the representation of how the output variable (in this study roof displacement) varies with each of the input parameters. The coefficients of each variable show the dependency of the roof drift on the corresponding variable. In this particular case the roof displacement depends primarily on PGA followed by compressive strength of concrete (f_c), yield strength of steel (f_y) and is least dependent on the Young's modulus of concrete (E_c).

In this study 3 point sampling of HDMR is used to obtain the metamodel. The sampling points considered are μ, μ +2σ and μ -2σ. The use of 5 point sampling which considers μ, μ + σ, μ - σ, μ +2σ and μ -2σ for sampling is expected to be more accurate when compared to 3 point sampling. The number of terms used from the HDMR equation, as given in Equation 2.5, for the development of metamodel also can contribute to the accuracy. In this work only the first two terms, that is the response at the mean point, which is a constant and the first order term representing the individual contribution of the variables is considered. Considering further terms may increase the precision of the metamodel.

4

FRAGILITY EVALUATION USING CORNELL'S METHOD

CHAPTER 4

FRAGILITY EVALUATION USING CORNELL'S METHOD

4.1 GENERAL

In this chapter a conventional method for development of fragility curve is used. The method is termed Cornell's method in this study which was developed by Cornell *et al.* in the year 2002. The detailed description of the method has been explained in Chapter 2. This method assumes power law to represent the input (PGA) and output (roof displacement) relation. This method uses Latin Hypercube sampling to generate the input sets. The same uncertainties in materials and ground motion, as taken in HDMR method conducted in Chapter 3, are used in this method also.

4.2 CORNELL'S METHOD

Cornell's method suggests Latin hypercube sampling of the random variables, compressive strength (f_c) and Young's modulus of concrete (E_c) and steel yield strength (f_y).

Latin Hypercube sampling is a sampling technique designed to accurately produce the input distribution through sampling in fewer repetitions when compared with the Monte Carlo method. The fundamental to Latin Hypercube sampling is stratification of the input probability distributions. Stratification divides the cumulative curve into equal interims on the cumulative probability scale (0 to 1.0). A model is then randomly taken from each interval or stratification of the input distribution. Sampling is enforced to represent values in each interval, and thus, is forced to recreate the input probability distribution. A sample is taken from every stratification. However, once a sample is drawn from stratification, this stratification is not sampled from again — its value is already represented in the sampled set. This conserves randomness and independence and avoids unwanted correlation between variables. 30 input variable sets for each random variable is generated using LHS method and the generated samples are given in Table 4.1.

Computational models of the frame are developed for the 30 sets of random variables. PGA values, which are used to scale the ground motion intensities, are uniformly distributed in the range of 0.1g to 1.0g to 30 values. For each set, time history analysis is done with the 44 earthquake records, scaled using the PGA values, and mean of maximum roof displacement obtained from each set is taken. The maximum roof displacements are also specified in Table 4.1.

Table 4.1 Set of input variables for Cornell's Method of Fragility evaluation

Sl No.	$f_c(Mpa)$	$E_c(Mpa)$	$f_y(Mpa)$	$PGA(g)$	y (mm)
1	1.60E+01	4.90E+04	5.06E+02	0.13	108.0182
2	2.00E+01	4.04E+04	4.79E+02	0.16	124.9727
3	2.20E+01	4.57E+04	4.75E+02	0.19	145.0203
4	2.80E+01	2.86E+04	4.37E+02	0.22	193.1908
5	1.80E+01	3.20E+04	5.25E+02	0.25	197.3111
6	2.10E+01	3.50E+04	4.67E+02	0.28	210.4379
7	2.00E+01	4.25E+04	4.42E+02	0.31	231.1819
8	1.40E+01	3.44E+04	5.34E+02	0.34	270.5515
9	1.90E+01	3.96E+04	4.59E+02	0.37	277.4043
10	1.50E+01	1.92E+04	4.47E+02	0.4	311.4825
11	2.30E+01	3.26E+04	4.55E+02	0.43	317.1274
12	1.70E+01	3.07E+04	4.32E+02	0.46	358.5697
13	2.00E+01	2.57E+04	4.20E+02	0.49	383.3279
14	1.80E+01	3.88E+04	5.69E+02	0.52	404.1237
15	2.10E+01	3.75E+04	5.46E+02	0.55	422.5428
16	1.80E+01	3.68E+04	4.87E+02	0.58	469.1865
17	2.20E+01	2.68E+04	4.13E+02	0.61	520.29
18	2.30E+01	4.38E+04	4.27E+02	0.64	543.7953
19	2.60E+01	3.81E+04	4.63E+02	0.67	585.1315
20	1.90E+01	3.32E+04	5.11E+02	0.7	593.7053
21	1.10E+01	3.62E+04	4.04E+02	0.73	723.4289
22	1.70E+01	3.56E+04	3.69E+02	0.76	782.3677
23	2.40E+01	2.44E+04	3.92E+02	0.79	761.3428
24	1.60E+01	3.38E+04	4.91E+02	0.82	764.4065
25	2.40E+01	3.01E+04	4.96E+02	0.85	771.214
26	2.10E+01	2.94E+04	5.01E+02	0.88	816.4072
27	2.50E+01	2.78E+04	4.71E+02	0.91	864.1148
28	1.30E+01	3.14E+04	4.83E+02	0.94	937.8056
29	1.90E+01	2.25E+04	4.51E+02	0.97	970.2053
30	1.50E+01	4.14E+04	5.18E+02	1	986.3147

Probabilistic Seismic Demand Model (PSDM)

Probabilistic seismic demand model is the relationship between maximum displacement (EDP) and the PGA (IM). Cornell (2002) assume power law model for PSDM as given by Equation 2.4. In order to find the parameters of the PSDM model, the maximum roof displacement (*y*) and the corresponding PGA from the set 1 to 30 is expressed in a logarithmic graph. The parameters of the power law model (*a*, *b*) are found out by regression method for the frame to form the PSDM model. Figure 4.1 shows the plot of maximum roof displacement (*y*) and the corresponding PGA values in logarithmic graph. The straight line is the fitted curve and the parameters of the PSDM model are obtained as a = 928.75 and b = 1.1261 which is also shown in Figure 4.1.

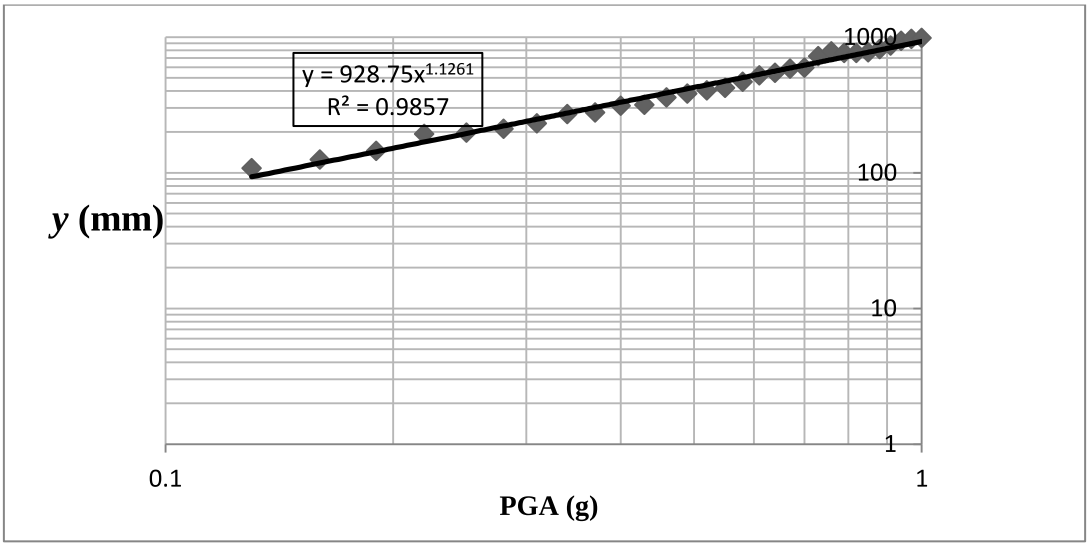

Figure 4.1 Probabilistic Seismic Demand Model (a=928.75, b= 1.1261)

The PSDM model obtained in this case is,

$$y = 928.75\ (PGA)^{1.1261} \tag{4.1}$$

Fragility Curve

The dispersion in capacity, β_c is reliant on the building type and construction excellence. For β_c, ATC 58 50% draft suggests 0.10, 0.25 and 0.40 depending on the quality of construction. In this study, dispersion in capacity has been assumed as 0.25.

$\beta_{d|IM}$ is the dispersion in the demand for given IM is found out using the equation 2.3b. The Fragility curves evaluated using the equation 2.2 for all limit states namely Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) and are shown in Figure 4.2.

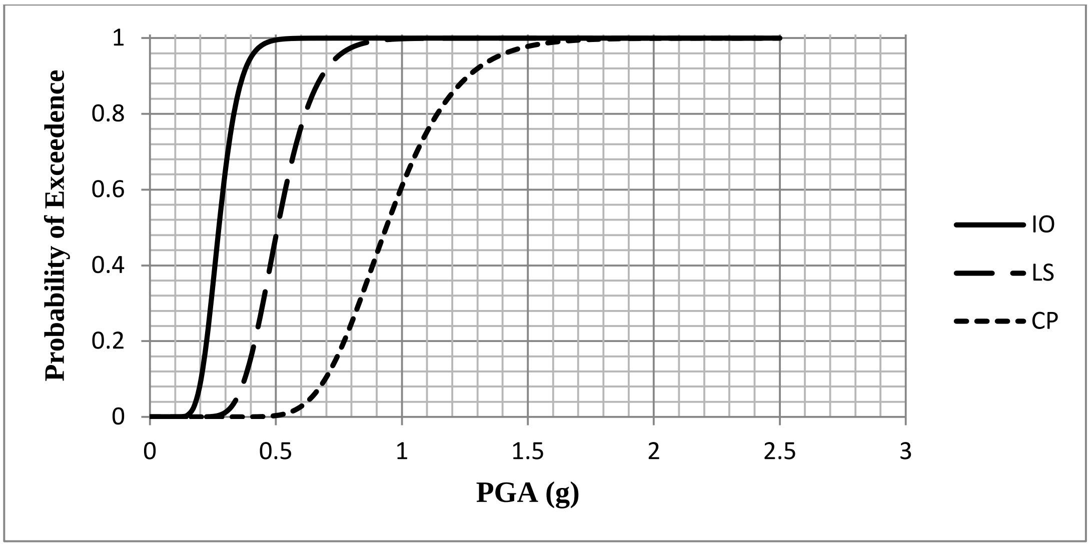

Figure 4.2 Fragility curve using Cornell's Method

4.3 COMPARISON OF FRAGILITY CURVES OBTAINED USING HDMR AND CORNELL'S METHOD

In this section the fragility curves developed using HDMR technique and Cornell's method is compared. Plots showing fragility curves using both the methods, taking into account each limit states, is shown in different figures. Figure 4.3 shows both curves for Immediate Occupancy, Figure 4.4 and 4.5 shows the comparison of the curves for Life Safety and Collapse Prevention limit states respectively. From the graphs showing the comparison of both methods the initial part seems to be same but the later part of the curve shows slight difference. The error in the fragility curve developed by HDMR method compared to that of Cornell method can be estimated using an error index proposed by Menjivar (2004). The error index is calculated for all the three cases and presented in the Table 3.6. This can be due to the various assumptions and approximations of the two approaches.

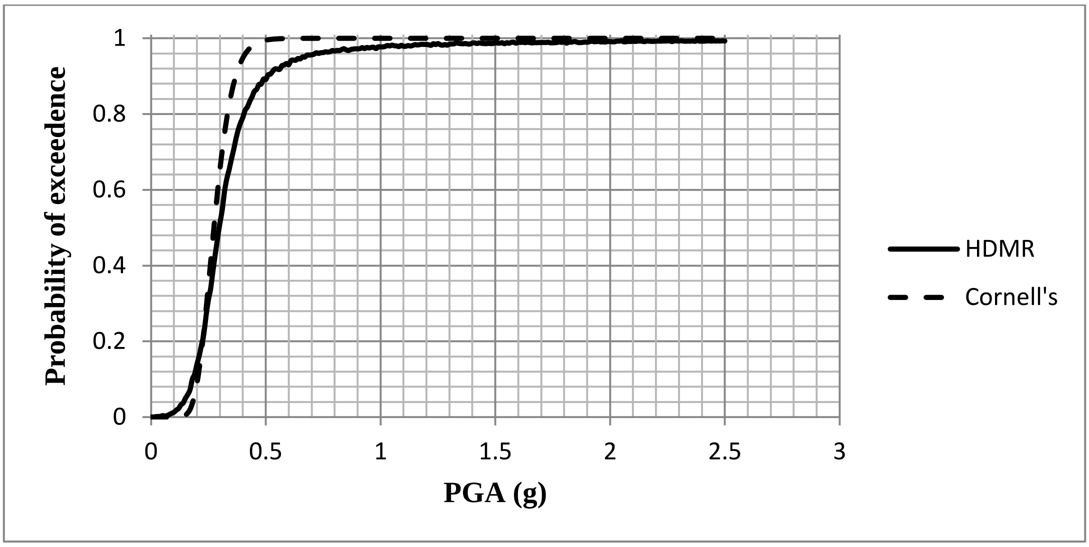

Figure 4.3 Comparison of fragility curve developed using HDMR and Cornell's method for the limit state Immediate Occupancy (IO)

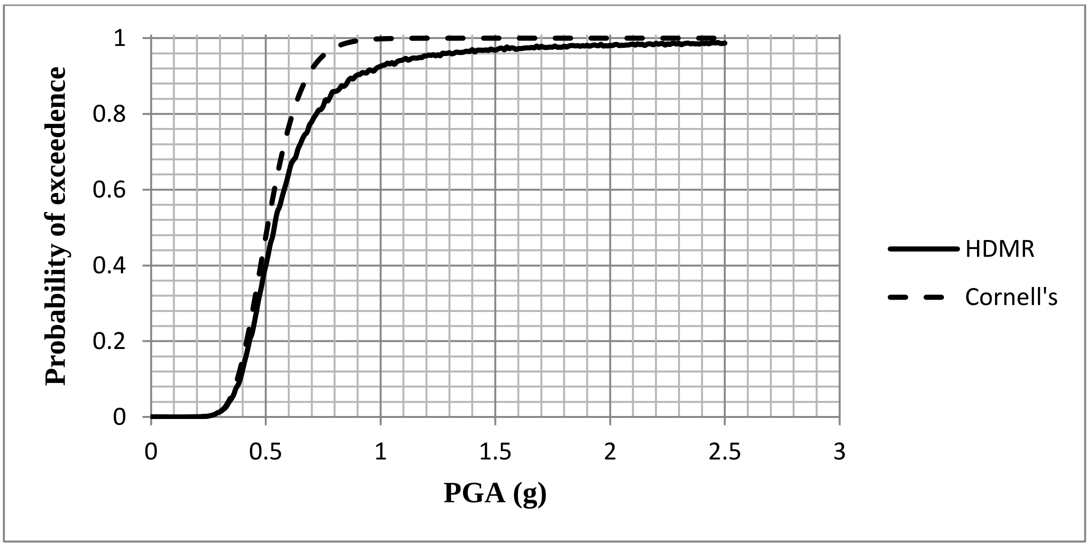

Figure 4.4 Comparison of fragility curve developed using HDMR and Cornell's method for the limit state Life Safety (LS)

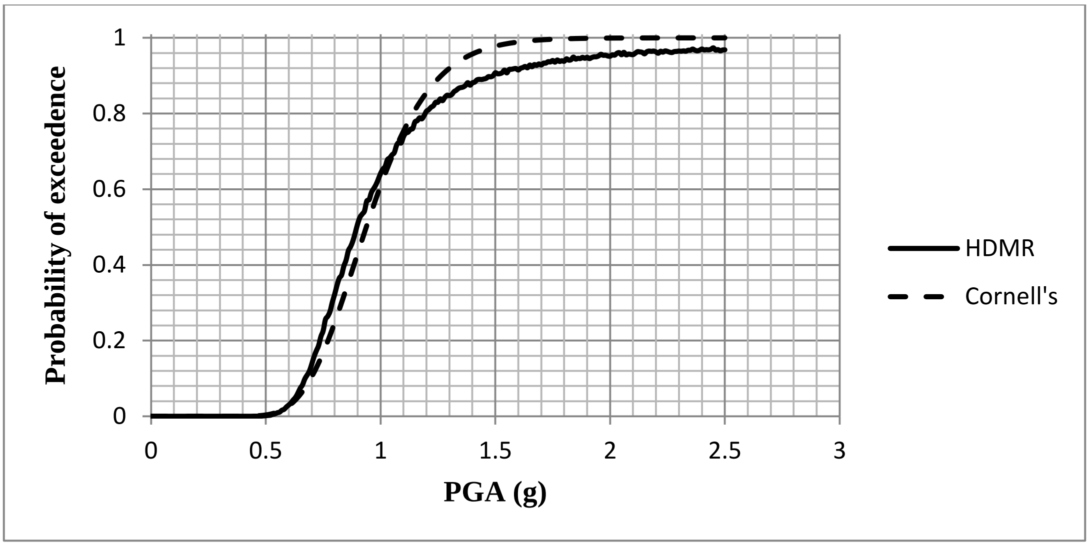

Figure 4.5 Comparison of fragility curve developed using HDMR and Cornell's method for the limit state Collapse Prevention (CP)

Table 4.2 Error index of fragility curve (HDMR) with reference to that developed using Cornell (2002)

Sl No	Limit state	Error Index in %
1	IO	9.8 %
2	LS	11.7 %
3	CP	15.7 %

4.4 COMPARISON OF COMPUTATIONAL EFFICIENCY

A comparison of computational efficiency between HDMR and Cornell's method is given in Table 3.7. To have a comparison with the Monte Carlo Simulation (accurate) the expected computational requirement for the same is also tabulated. Time taken for single analysis (computational model developed using a set of input variables and its time history analysis for 44 scaled earthquake records) is about 5 hours. From the table it is evident that HDMR method is fairly efficient in the computational time when compared to MCS.

Table 4.3 Computational requirements of different methods of Fragility Evaluation

Method	Number of analysis required	Estimated time if done using a single system
MCS (expected)	10000 minimum	70 months
Cornell's	30	7 days
HDMR 3-point	9	2 days
HDMR 5-point(expected)	17	4 days

4.5 DISCUSSIONS

In this chapter Cornell's method was successfully used for development of fragility curves. But the number of sampling points taken in this study is only 30 due to the factors of time. The fragility curves may differ from the accurate one due several kind of assumptions and approximations. By increasing the number of sampling sets the curve can be made more accurate and close to the one that will be obtained using Monte Carlo Simulation which is out of scope of this study.

The comparative study between HDMR and Cornell's method show slight variation in the graphs obtained. This can be due to the approximations in Cornell's method, like number of sampling points, or the limited study conducted on HDMR. In the present study, only first two terms in the HDMR equation (Equation 2.5) are considered and also a 3 point sampling is used. Reducing these approximations may provide more accurate results.

5

SUMMARY AND CONCLUSIONS

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 SUMMARY

Fragility curves are useful representation of conditional probability of structural response when subjected to earthquake loads as a function of ground motion intensity. Fragility curve has an important role in the present scenario in the pre-and post-earthquake damage and loss estimation to the design makers. Generation of fragility curves in conventional methods involves earthquake simulation of large number of computational models that represent the inherent variation in the material properties of a particular building type to obtain an accurate and reliable estimate of the probability of exceedance of the chosen damage parameter. High Dimensional Model Representation (HDMR) method is a type of response surface method that can express input-output relations of complex computational models. This input-output relation can reduce the number of iterations of expensive computations especially in problems like fragility curve development. HDMR method was implemented in fragility curve development for the first time by Unnikrishnan *et al.* (2011). In this study, fragility curve of an RC frame is developed using three point sampling HDMR response surface method considering the first two terms of the generalized HDMR input output relation. A method proposed by Cornell *et.al.* (2002) is one of the popular and simplified approaches for fragility curve development. This method assumes a power law model between the damage parameter and intensity measure of earthquake. The objective of the present study was to develop the fragility curve for an RC frame applying HDMR expansion and study the relative computational efficiency and accuracy with reference to the one proposed by Cornell (2002). The conclusions obtained from the study, limitations of the present work and the future scopes of this research are quoted in this chapter.

5.2 CONCLUSIONS

The following are the major conclusions that are reached from the studies conducted:

5.2.1 HDMR method of Fragility Evaluation

- Computational efficiency with reference to Monte Carlo Simulation
  - Time History analysis of one model for 44 earthquake data takes about 5 hour for the considered plane frame.
  - If Monte Carlo simulation is used for the evaluation of fragility curve a minimum of 10,000 time history analysis is to be performed.
  - In HDMR 3-point sampling method, only 9 Time History analysis was done to obtain the metamodel, on which Monte Carlo simulation was done using the metamodel (generating 10,000 random values for the input variables), which takes only few minutes.
  - The time consumption is reduced by about 99.9% compared to MCS when HDMR is used.
- The metamodel is a polynomial function that relates roof displacement (damage parameter) with the random variables defining the frame. The metamodel provides the dependency of damage parameter on each of the random variables. Higher the value of the coefficient of the random variable higher will be the dependency on it. The metamodel developed for the RC frame in the present study show that the order of dependency of each random variables on the Metamodel is as follows

$$E_c < f_y < f_c < PGA$$

5.2.2 Fragility Evaluation using Cornell's Method

- Computational efficiency of this method when compared to MCS is about 99.6%.

- Comparative study between fragility curves obtained by Cornell's method and HDMR shows that the initial part of the curve is almost same but in the further section of the curve (at higher PGA values), slight difference is observed which can be attributed to the approximations and assumptions followed by both methods.

5.3 LIMITATIONS OF THE STUDY AND SCOPE OF FUTURE WORK

The limitations of the present study are summarised below.

- The present study considered only one plane frame for fragility evaluation. More number of frames which may include 3-D fames can be used for the same and effectiveness of HDMR can be studied.
- Uncertainties in modelling are considered only for compressive strength and Young's modulus of concrete and yield strength of steel.

HDMR Method

- Only 3-point sampling is used while conducting the fragility evaluation using HDMR. The use of 5-point sampling may give more accurate results. In addition to 3-point sampling 5-point sampling can also be utilized.
- In the development of metamodel only the first two terms of HDMR equation (Equation 2.5) is used. The use of further terms in the HDMR equation can be incorporated in future works.

CORNELL'S Method

- In the present study only 30 input sets are considered for development of Fragility curve using this method. Use of more input sets may lead to higher accuracy.
- Other methods like response surface methodology can be used for fragility assessment.
- Monte Carlo Simulation technique for fragility evaluation, even though it may take long time, can be used by the help of High Performance Computational Facilities for getting the most accurate fragility curves, and the correctness of other methods can be studied.

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