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DISPLACEMENT BASED FRAGILITY CURVES FOR R.C.C. FRAME STRUCTURES IN CONTEXT OF DHAKA, BANGLADESH.

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ABSTRACT: In recent past, severe earthquakes have caused substantial physical losses and casualties in this sub-continent. At present Bangladesh is in high risk of attacked by earthquakes. Since a majority of the population is living in earthquake-prone areas, it is probable that such terrible events will take place again in the near future. Moreover, it is not easy to cope with the substantial direct and indirect economical losses after each devastating earthquake for a developing country like Bangladesh. Because in this country most of the reinforced concrete buildings are not designed according to the current building code, seismic behavior is not taken into consideration in the architectural design and during selection of the structural system and supervision in the construction phase is not adequate which in turn induces deficiencies like poor concrete quality, inadequate detailing of reinforcement etc. It is, therefore, vital to quantify the earthquake risk and to develop strategies for disaster mitigation. In order to achieve this goal, an extensive and inter-disciplinary study is required. Such a study is composed of two parts: hazard determination and vulnerability assessment. This study describes the methods by which it is possible to determine the vulnerability of existing engineering structures and building stock. The tool that is employed to assess the seismic performance of reinforced concrete frame structures is the fragility curve. By definition, fragility curves provide estimates for the probabilities of reaching or exceeding various limit states at given levels of ground shaking intensity for an individual structure or population of structures (MAE Report, 2003). A limit state; which is in the same terms as the response, usually represents a damage condition or a limitation of usage. The primary focus of this paper is to present a proper methodology that can be followed to construct fragility curves for R.C.C frame structures in Bangladesh and to generate fragility curves for some specific type of RCC frame structures using this methodology.

Keywords: Hazard Determination, Vulnerability, Fragility, Limit State.

1. INTRODUCTION

Bangladesh is an earthquake-prone country. Although, a large part of the urban building stock is engineered but the quality of the construction in some cases may not be satisfactory. Luckily, the recent major earthquakes have occurred away from the city. The history of the region indicates that there is a strong possibility of major earthquakes occurrence that could cause extreme devastation. In fact recent studies demonstrated that even moderate earthquakes could be fatal in populated, unplanned cities. General public and the engineering community are now becoming more and more aware of the situation. However, neither the possible extents of seismic damage of existing buildings are known nor there is any guideline for their strengthening measures. Even the performance of the engineered buildings under a seismic event is questionable, as enough work has not yet been done in this field. In this study the prime objective is to present an appropriate method to assess the seismic performance of RCC structures in Bangladesh in terms of fragility function.

The 1997 Chittagong Earthquake (Sabri, 2002) the 1999 Moheskhali Earthquake (Ansary et al., 2001) and the 2003 Rangamati Earthquake (Ansary et al., 2003) revealed the vulnerability of "non-earthquake proof" cities and villages in southern part of Bangladesh. In

1897, an earthquake of magnitude 8.0 caused serious damage to buildings in the northeastern part of India (including Bangladesh) and 1542 people were Killed. Recently Bilham et al. (2001) pointed out that there is high possibility that a huge earthquake will occur around the Himalayan region based on the difference between energy accumulation in this region and historical earthquake occurrence. The population increase around this region is at least 50 times than the population of 1897 and cities like Dhaka, Chittagong, Kathmandu have population exceeding several millions. It is a cause of great concern that the next great earthquake may occur in this region at any time. According to a report published by United Nations IDNDR-RADIUS Initiative, Dhaka and Tehran are the cities with the highest relative earthquake disaster risk (Cardona et al., 1999). Once a great earthquake occurs, Dhaka will suffer immense losses of life and property.

The primary focus of the present study is seismic damage estimation of the structures of Bangladesh. Damage estimation is a vital part of the seismic performance evaluation of buildings and other structures with respect to multiple performance objectives. In turn, the proper evaluation of seismic performance is essential for decision making involved in managing the risk to building, bridges, and other infrastructure in seismically active areas. Today, the earthquake engineering community faces new challenges that are brought about by the latest needs of the real estate development and management industries. The safety of buildings and other structures used to be the main concern of designers, owners, and regulators. The development of modern building codes has provided society with guidelines that serve well for achieving the required safety levels. However, nowadays other issues are becoming significant for owners and risk managers. Providing that safety requirements are met, the questions being asked now are “how much does it cost to repair?”, “how long it will be shut down in case of the earthquake?” etc. These questions relate to the economic aspect of the seismic performance of real estate. Given the multiple performance objectives, accurate damage estimation becomes more important than ever.

Fragility functions are the essential tools for seismic loss estimation in built environments. They represent the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation (Shinozuka et al., 1999). The damage limit states in fragilities may be defined as global drift ratio (maximum roof drift normalized by the building height), inter-story drift ratio (maximum lateral displacement between two consecutive stories normalized by the story height), maximum roof displacement or story shear force etc. The ground motion intensities in the fragility functions can be spectral quantities, peak ground motion values, modified Mercalli scale etc. In this respect, fragility curves involve uncertainties associated with structural capacity, damage limit state definition and variability of ground motion intensity. Thus from fragility functions the seismic performance of any structure can be examined and its level of serviceability during an earthquake can be evaluated.

2. FRAGILITY CURVE

As noted above, fragility (or vulnerability) can be described in terms of the conditional probability of a system reaching a prescribed limit state (LS) for a given system demand $D = d$, $P(LS|D = d)$. Limit states related to structural behavior range from unserviceability to various degrees of damage including incipient collapse. Demands can be in the form of maximum force, displacement caused by earthquake ground motions, or more generally a prescribed intensity measure of the ground motion, over a given period of time. Expressed in this general manner, the fragility (or vulnerability) is a function of the system capacity against each limit state as well as the uncertainty in the capacity. The capacity controls the central location of the Fragility Curve (FC) and the uncertainty in the capacity controls the shape (or dispersion) of the FC (Figure 1). For a deterministic system with no capacity uncertainty, the FC is a step function. Strictly speaking, FC is primarily a property of the system dependent on the limit state.

3. LIMIT STATE PROBABILITY

To tie the vulnerability of a given system to the seismicity of the region, the seismic hazard needs to be included in the consideration. The vulnerability needs to be described in terms of probability of a set of given limit states being reached of a system at a given location over a given period of time (0, t). Alternatively, the vulnerability can be stated in terms of occurrence rates of the prescribed limit states. In other words, a system of given capacity may be more vulnerable to earthquakes if it is located in a region of high seismicity than in a region of moderate or low seismicity. Knowing the fragility curve, the limit state (LS) probability over the time period (0, t) can be evaluated:

$$P_t(LS) = \int P(D|LS = d) f_D(d)\,dd \qquad (1)$$

in which $f_D(d)$ = the probability density function of the demand during (0, t), depending on the regional seismicity and ground excitation. In other words, through Eq. 2 the fragility curve and the probabilistic demand curve are combined.

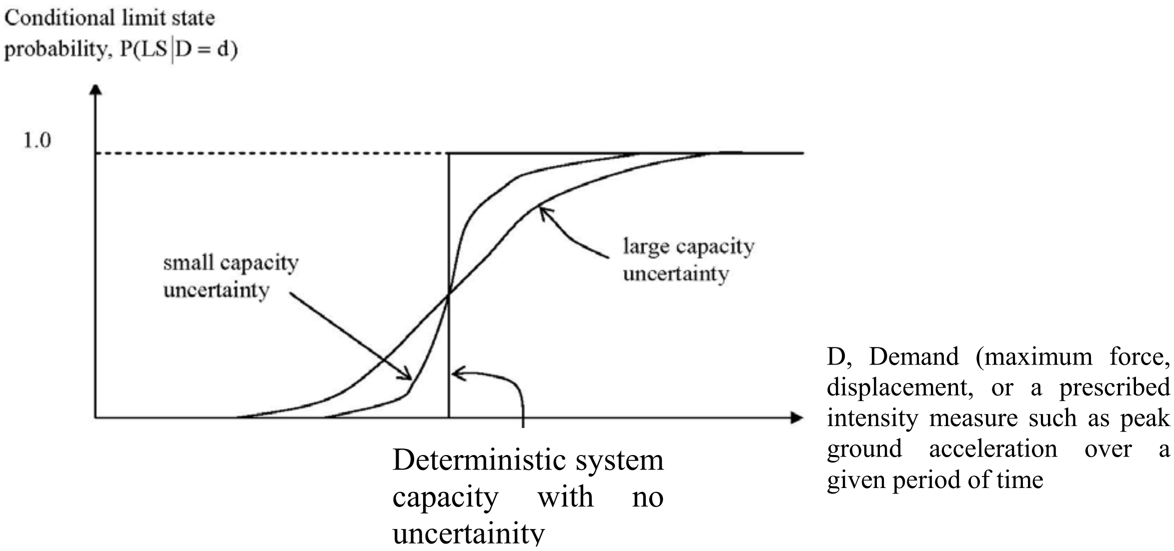

Fig 1: *Characteristic of a Fragility Curve*

4. METHODS TO DETERMINE SEISMIC FRAGILITY OF A STRUCTURE

To estimate the seismic fragility of a specific building type, two different approaches can be considered. In the first approach which is known as empirical method, the damage reports are usually utilized to establish the relation between the ground motion intensity and the damage state of each building. This method employs loss data from historical earthquakes. The seismic fragility function is created by examining the loss data by structure category and regressing loss against shaking intensity. To utilize this approach past earthquake and corresponding damage data of each building stock have to be available. The second approach which is known as analytical procedure is to conduct the fragility studies by performing structural analysis to estimate the structural response to a ground motion in terms of internal forces and deformations. The structural response is then input to fragility functions to determine the damage state of the building stock. Simple models and methods are employed in this approach. The advantage of this method is that it is simple and economically feasible. In addition, the nontechnical decision makers prefer such simple and rapid estimates of anticipated losses to develop the proper judgment to execute their mitigation plans. However, the obtained results will be crude and the limitations of the models or the methods should be carefully understood. The second approach can be considered as an appropriate way to estimate seismic fragility of the building stock in Bangladesh as past earthquake data are not available here.

5. DEVELOPMENT OR SELECTION OF REPRESENTATIVE MODELS

As mentioned earlier two different approaches are available to evaluate seismic vulnerability. For first approach there is no uncertainty in modeling as all the structures in the stock are analyzed separately. But for second approach statistical properties of the building stock are utilized. Therefore, selection and development of models are of immense importance. Various construction parameters of reinforced cement concrete (RCC) structures or buildings such as compressive strength of concrete ($f_c^{/}$), yield strength of steel (f_y), general trend in column and beam size etc. vary with different countries, even within the country. These parameters have predominant influence on lateral stiffness and strength of a structure. Thus site specific spectrum of these parameters should be selected during modeling to obtain fragility curves. Statistical properties of the building stock affecting lateral load deformation characteristics can be selected as follows:

(a) Random sampling can be done from a building database and then lateral load-deformation characteristics are examined. Similar process is followed by two research projects in Turkey. In one study (Yakut et al. 2004), 32 sample buildings are taken as random basis from 500 building database and then nonlinear static pushover analysis is conducted to determine lateral load-deformation characteristics. From pushover analysis the yield base shear co-efficient (V_y/W) and the yield global drift ratio (θ_y) are selected as random variables. The statistical properties of these two random variables are then determined. In the second study (Cullu, 2004) the random variables are taken as period (T) of the structures and the strength ratio (V_y/W). In this method proper field data of each building stock are necessary.

(b) Models can be developed using $f_c^{/}$, f_y, column size, beam size, bay length etc as random variables. These random variables should represent the site specific variation in order to construct the representative fragility curves. Similar method was employed in a study in University of Southern California (Shinozuka et al., 2001) where $fc^{/}$ and f_y are taken as random variables for developing models. In this study ten nominally identical but statistically different structures are created by simulating ten realizations of $fc^{/}$ and f_y according to respective probability distribution functions assumed. Then pushover analysis is done to determine statistical properties and limit state. In this method the trend in various construction parameters should be available or can be assumed on the basis of judgment and prior experience.

Fragility functions can also be developed by taking into consideration other variables such as the period (T), the post-elastic stiffness, ultimate strength as uncertain quantities. But a study by M.Altug Erberik, Department of Civil Engineering, Middle East Technical University, Ankara, Turkey has shown that the post-elastic stiffness has a very little effect on fragility curves.

6. IDENTIFICATION OF IMPORTANT LIMIT STATE

Performance levels or limit states for both structural and non-structural systems are defined as the point in which the system is no longer capable of satisfying a desired function. There are many types of performance levels in the field of earthquake engineering. In addition, performance levels can be identified by qualitative and quantitative approaches. Both methods are summarized below.

6.1 Traditional Qualitative Approaches

Qualitative approaches for identification of performance levels have traditionally been used in building codes. In particular, most building codes require designers to ensure life safety of the occupants during factored loading and serviceability or functionality during unfactored loading. FEMA 356 has the most comprehensive documentation on performance levels that are defined qualitatively and is briefly summarized below. FEMA 356 defines performance levels related to the structural system as:

(1) ***Immediate Occupancy (IO)*** - occupants are allowed immediate access into the structure following the earthquake and the pre-earthquake design strength and stiffness are retained;
(2) ***Life Safety (LS)*** - building occupants are protected from loss of life with a significant margin against the onset of partial or total structural collapse;
(3) ***Collapse Prevention (CP)*** – building continues to support gravity loading, but retains no margin against collapse.

6.2 Quantitative Approaches

Although current building codes and state-of-the-art publications have attempted to define the various performance levels for structural and non-structural systems, performance levels have only been identified qualitatively. Therefore, designers have to determine quantitative response limits that correspond to the qualitative code descriptions. Another approach for defining structural performance levels might be based on quantitative procedures using nonlinear pushover techniques (ATC-40, 1996). By this pushover technique customized values for different damage state such as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) can be evaluated.

7. SEISMIC PERFORMANCE EVALUATION BY NONLINEAR STATIC PROCEDURE

Two or three-dimensional models of each sample building can be prepared in the SAP2000 environment (Computers and Structures, 2000). Nonlinear static analysis is then conducted to determine the base shear versus roof displacement relationship (capacity curve). Flexural elements for beams, beam-column elements for columns, strut elements for infill walls and rigid diaphragms for floors can be employed for modeling the structural components of the buildings.

Nonlinear flexural characteristics of the individual frame members are defined by moment-rotation relationships of plastic hinges assigned at the member ends. Flexural moment capacities are based on the section and material properties of members. Column capacities are calculated from the axial force-bending moment interaction diagrams. A typical moment-rotation relationship for frame members is shown in Figure 2. The segment AB, representing initial linear behavior, is followed by the post-yield behavior BC. Point C corresponds to the ultimate strength, where a sudden loss of strength occurs when the associated plastic rotation level is exceeded. This drop from C to D represents the initiation of failure in the member.

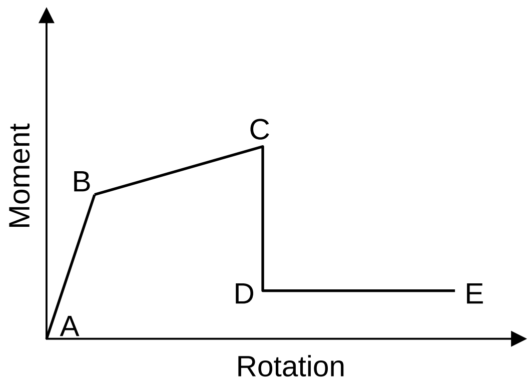

Fig 2: *Idealized moment-rotation relationship of a frame member-end*

8. DEVELOPMENT OF DIMENSIONLESS BILINEAR CAPACITY CURVE

The capacity curve of each model can be approximated with a bilinear curve using the guidelines given in FEMA-356 (ASCE, 2000). A typical idealization of a capacity curve is shown in Figure 3. It is required to specify the yield and ultimate strength capacities and their associated global drift values for constructing the approximate bilinear capacity curve. The global drifts can be used to represent the damage limit states of the buildings. The yield global drift ratio θ_y represents significant yielding of the system when the yield base shear capacity (V_y) of the building is attained whereas the ultimate global drift ratio θ_u corresponds to the state at which the building reaches its deformation capacity. The base shear coefficient $\eta = V_y/W$ in Figure 3 is the ratio of yield base shear capacity to the building weight.

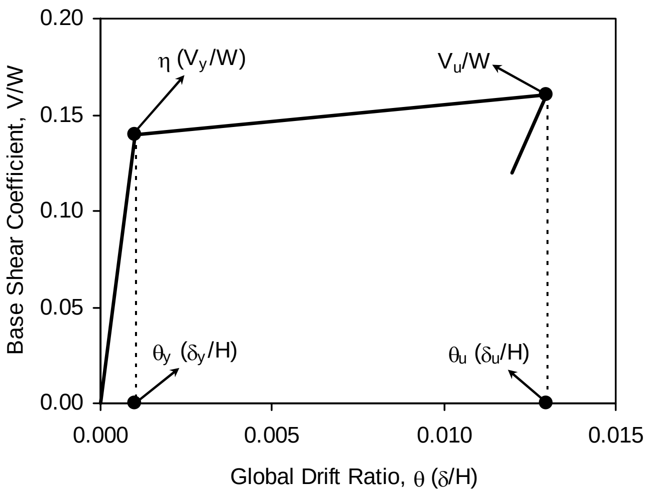

Fig 3. *A typical bilinear capacity curve.*

It should be noted that there is no universal consensus on how to approximate a capacity curve with a bilinear force-deformation representation. An initial stiffness targeting at the state of significant global yielding may lead to considerable variations in V_y and θ_y because there is no specific point on the capacity curve exactly describing significant yielding (Sullivan et al., 2004).

9. IDENTIFICATION OF QUANTITATIVE LIMIT STATE

Representative probability density functions of θ_y and θ_u can be determined in terms of mean, median and standard deviation. When global ductility capacities (θ_u/θ_y) are calculated both θ_y and θ_u can be utilized to determine deformation capacities. It is more appropriate to employ θ_u in assessing the deformation capacities of such buildings, which have infill walls or short span length (Yakut, 2004).

Three performance limits, immediate occupancy, life safety and collapse prevention that are specified in several other international guidelines are usually adopted in fragility studies. The collapse prevention performance limit θ_{CP} is taken as the 75 percent of the median θ_u computed considering the uncertainty in modeling and skewness of the ultimate drift probability function. The life safety performance is assigned as the 3 quartile or half of the suggested collapse prevention limit depending on the vulnerability of structure. The median θ_y computed for each story-based building group is accepted to be the limiting value for the immediate occupancy performance level. It is assumed that light, moderate and severe damage states are experienced when the immediate occupancy, life safety and collapse prevention drift limits are exceeded, respectively. The selected performance limits that are described qualitatively in Table 1 are conjectural and could be argued as subjective. (Yakut, 2004).

Table 1: *Assumed drift ratio limits for performance levels*

Performance Level	Limit State
Collapse Prevention	$\theta \le \theta_{CP}$
Life safety	$\theta \le ¾ \sim ½\theta_{CP}$
Immediate occupancy	$\theta \le \theta_y$

10. DEVELOPMENT OF REPRESENTATIVE MODELS IN CONTEXT OF DHAKA, BANGLADESH

To develop fragility curves of reinforced cement concrete frame structures of Dhaka, Bangladesh the models can be developed using $f_c^{/}$ as a random variable. Then numerous modeled can be constructed using the general trend in the variation in size of beams and columns and the assumed probability density function of $f_c^{/}$. The capacity curves are then obtained by performing nonlinear static analysis of the models. Bilinear capacity curves are then constructed as describe above. From the bilinear capacity curves the yield base shear co-efficient (V_y/W), the yield global drift ratio (θ_y) the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope (α) can be selected as random basis and the statistical properties of these three quantities (V_y/W, θ_y and α) are determined. From the bilinear capacity curves the ultimate global drift ratio θ_u are also calculated. The collapse prevention performance level is then estimated from the median values of θ_u. The other limit states are then selected as shown in Table 1.

To develop fragility curve for a particular RCC building again $f_c^{/}$ should be taken as a random variable as $f_c^{/}$ varies significantly in our country. As here fragility curve has to be developed for a single structure the capacity uncertainty will be small than constructing fragility curves for a locality. In our country as constructional quality control is always neglected there are always scope for variation in column and beam size and also in slab thickness than from the design dimension. These changes can have considerable effect on lateral strength properties of a reinforced cement concrete frame structure. But care should be taken that these variations have to be country specific otherwise they would not reflect the actual scenario. The random variables, statistical properties of these random variables and the quantitative limit states can be found as mentioned above.

11. GROUND MOTION DATA

The ground motion intensities in the fragility functions can be spectral quantities, peak ground motion values, modified Mercalli scale etc. Since recorded ground motions for Bangladesh are not available, synthetic ground motions can be developed. Various methods are available to simulate earthquake motion such as Hwang and Huo (1996), Wu and Wen (2001), Kanai-Tajimi (K-T) power spectra and Shinozuka-Sato (1967), MCEER Project (2004) etc.

The time history plotting of earthquake motions for Dhaka city can be developed using MCEER project in Cornell University, Ithaca, NY. With the help of this project it is possible to simulate the Gaussian/Non-Gaussian and Stationary/Non-stationary model of earthquake. The inputs are moment magnitude of the earthquake, source to sight distance and type of soil. The moment magnitude and source to site distance can be determined by Probabilistic Seismic Hazard Analysis (PSHA). The sources and their specifications are detailed in Noor, (2005) and also briefly described in literature review.

11.1 Input Parameters for Dhaka City

From Noor, (2005) following parameters of earthquake sources are found. From Table 3.2 and Table 3.3 it is found that the nearest earthquake source (A2) from Dhaka is at a distance of 91.779 km and the maximum magnitude of earthquake for this source is 7.0. Simulated earthquake can be produced using the parameters of this source. The fragility curves thus constructed should represent the fragility curves of RC buildings for this source. The soil type can be taken as Soil Type 4: NEHRP class D as the soil properties of Dhaka city have similarity with this NEHRP class of soil.

Thus using source distance 91.779 km, soil type 4 and changing magnitude a number of earthquakes can be generated of different PGA values. These time-history plots of earthquake are then utilized for to perform nonlinear time history analysis of the RCC buildings in order to construct fragility curves.

Table 2: *Parameters of the Earthquake sources.*

Source parameter	*Source 1 A1*	*Source 2 A2*	*Source 3 A3*	*Source 4 A4*	*Source 5 A5*	*Source 6 A6*	*Source 7 A7*
Area (sq km)	*21158*	*28494*	*76537*	*14434*	*105900*	*42227*	*57774*
No. of division	*18*	*33*	*70*	*15*	*112*	*40*	*56*
Area of each division (sqkm)	*1175.5*	*863.45*	*1093.4*	*962.27*	*945.55*	*1055.7*	*1031.7*
No. of Earthquake in database	*72*	*15*	*277*	*17*	*622*	*54*	*11*
Minimum magnitude (M_s)	*4.0*	*4.0*	*4.0*	*4.0*	*4.0*	*4.0*	*4.3*
Maximum magnitude (M_s)	*8.0*	*7.0*	*7.6*	*6.5*	*7.5*	*8.3*	*5.6*

Table 3: *Calculation of design ground motion parameter*

Source Name	***Database M_{max}***	***R_{min} (km)***	***PGA (g)***	
			McGuire (1978)	*Boore et al. (1993)*
A1	*8.0*	*113.47*	*0.117*	*0.061*
A2	*7.0*	*91.779*	*0.061*	*0.043*
A3	*7.6*	*104.05*	*0.090*	*0.053*
A7	*5.6*	*64.437*	*0.025*	*0.027*
Earthquake at Latitude 24 N Longitude 90.3 E	*5.7*	*25.75*	*0.054*	*0.045*

12. NONLINEAR DYNAMIC RESPONSE HISTORY ANALYSES AND COMPUTATION OF FRAGILITY CURVES

The set of earthquake records generated by the simulation procedures comprising the ground motion data is then used to compute the dynamic time-history response of the developed models. The SAP 2000 finite element code can be utilized in order to simulate the state of damage of each structure under ground acceleration time-history. The global drift ratio can be calculated by dividing the maximum value of the roof displacement, δ_{top} by average building height.

The maximum global drift values computed by the above procedure are then assumed to represent the seismic performance of the investigated concrete frames. Using the damage threshold levels defined in Table 1, the exceedance probabilities of that particular fragility curve were computed from the PGA/PGV/Spectral Acceleration versus maximum global drift scatters. The scatter diagrams were clustered for different PGA/PGV/Spectral Acceleration intervals and the global drift percentiles greater than a given damage threshold level were computed by using the normal distribution to estimate the exceedance probabilities of the fragility curves. A representative sketch for the above procedure is shown in Figure 4 (Cullu, 2004).

13. FRAGILITY CURVES FOR A PARTICULAR TYPE OF STRUCTURE

This section describes the generation of fragility curves for a specific type of RCC frame structure following the previously mentioned guidelines in context of Dhaka city. In this case fragility curves are developed for 3-story RCC buildings of Dhaka city. Selection of random variables, developments of models and generation of earthquake time-history plot has also been described. The main objective of this chapter is to establish the guidelines for developing fragility curves.

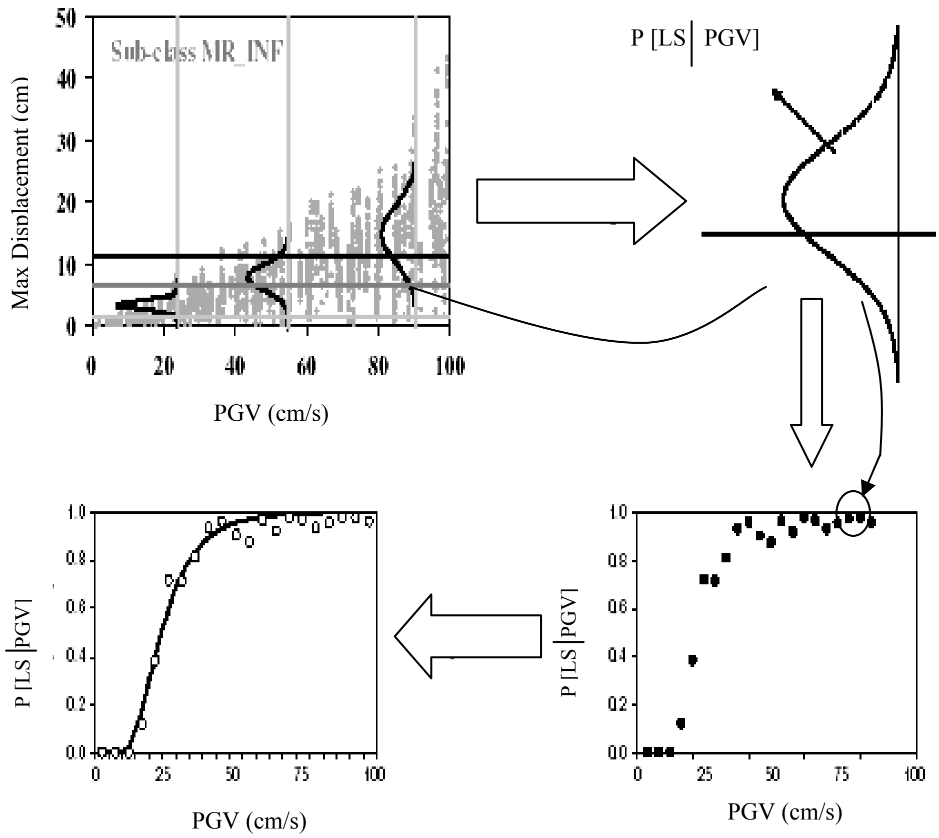

Fig 4: *The scatter diagram and generation of the corresponding fragility curves*

14. DEVELOPMENT OF MODELS

A particular structure type is considered in this study, namely 3-story existing reinforced concrete buildings, which generally do not comply with modern seismic resistant design and construction practice. Two dimensional models are created in SAP 2000 environment to perform nonlinear static analysis (pushover) and nonlinear time history analysis. As proper design data are not available in our country model have to be constructed using assumed probability density function and general trend of construction parameters. The random variables (yield base shear coefficient, yield global drift ratio and the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope) are then selected and statistical properties of these random variables in terms of mean and standard deviation are then determined. These statistical properties represent the group of building stock which seismic vulnerability will be reflected by the generated fragility curves by analyzing these models.

Simple two dimensional models are developed and for two dimensional models the construction parameters are the $f_c^{/}$, f_y, column size, beam size and bay length. In this work $fc^{/}$ and column size are taken as variable parameters. f_y, beam size and bay length are kept constant. Beam reinforcement is taken for gravity loads i.e. self wt, deal load and live load. In Bangladesh there is large variation in $f_c^{/}$ in different construction site but variation in f_y is not as much as in $f_c^{/}$. The beam size in 3 story building has little variation as most of them are designed for gravity load and span length are almost same that usually varies from 10 ft to 18 ft (PWD design section and RVS survey, 2005).The column size varies from 8 by 8 inches to 12 by 12 inches and this variation in column size has significant effect on lateral strength capacities of buildings. The reinforcement in column is taken as 2% ~ 3 % of the concrete gross section. Table 4 shows details about various construction parameters of developed models. Figure 5 shows the frequency distribution of $f_c^{/}$. 35 samples of $f_c^{/}$ are generated having mean value of 3000 psi, maximum value of 5000 psi and minimum value of 1700 psi. This assumed density of $f_c^{/}$ is based on BRTC test data of BUET.

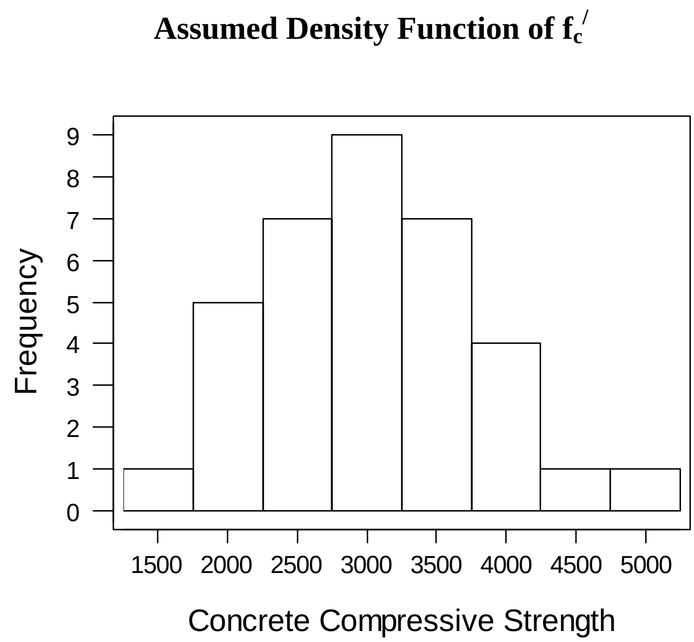

Fig 5: *Assumed Density Function of $f_c^{/}$*

Table 4: *Details of Construction Parameters*

Construction Parameter	Type	Dimension
Concrete Compressive Strength ($f_c^{/}$)	Variable	Mean = 3000 psi Standard Deviation = 800
Steel Yield Strength (f_y)	Constant	40 ksi
Column Size	Variable	8 by 8 inches ~ 12 by 12 inches Reinforcement: 2% ~ 3%
Beam Size	Constant	10 by 15 inches Reinforcement: For Gravity Load
Bay Length	Constant	15 ft

15. NON-LINEAR STATIC ANALYSIS AND IDENTIFICATION OF LIMIT STATE

Using the variable construction parameters shown in table 4.1 60 two dimensional models are developed in SAP 2000 environment having different $f_c^{/}$ and column size. Then nonlinear static analysis (pushover) is carried out to develop pushover curves for these 60 models. From these 60 developed models 30 samples are taken as random basis to construct fragility curves. The bilinear capacity curves are constructed for these 30 samples of 3-story structures. From these bilinear capacity curves the yield base shear co-efficient (V_y/W), the yield global drift ratio (θ_y) and the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope (α) are then selected as random variables and the statistical properties of these three quantities (V_y/W , θ_y and α) are determined.

The capacity curve (pushover curve) of each model is then approximated with a bilinear curve using the guidelines given in FEMA-356 (ASCE, 2000). From the 30 bilinear capacity curves probability density functions of θ_y and θ_u are determined in terms of mean, median and standard deviation. Three performance limits, immediate occupancy, life safety and collapse prevention that are specified in several other international guidelines are adopted in this fragility study.

From bilinear capacity curves θ_y and θ_u for thirty structures are determined. The collapse prevention performance limit θ_{CP} is taken as the 75 percent of the median θ_u computed considering the uncertainty in modeling. The life safety performance is assigned as the half of the suggested collapse prevention limit as most the structures in this region are not properly designed and detailed for seismicity. The median θ_y computed for each story-based building group is accepted to be the limiting value for the immediate occupancy performance level. It is assumed that light, moderate and severe damage states are experienced when the immediate occupancy, life safety and collapse prevention drift limits are exceeded, respectively. Table 5 shows the statistical properties of θ_y and θ_u. The damage thresholds are then obtained in terms of θ_y and θ_u and shown in table 6.

Table 5: *Statistical Properties of θ_y and θ_u*

Parameter	Mean	Median	Standard Deviation
θ_y	0.0011	0.0011	0.00014
θ_u	0.0081	0.0081	0.00025

Table 6: *Damage Threshold*

Limit State	Value
Immediate Occupancy (Light Damage)	0.0011
Life Safety (Moderate Damage)	0.0030
Collapse Prevention (Severe Damage)	0.0061

16. STATISTICAL PROPERTIES OF RANDOM VARIABLES

As mentioned earlier that the yield base shear co-efficient (V_y/W), the yield global drift ratio (θ_y) and the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope (α) are taken as random variables, the statistical properties of these random variables have to be determined. The statistical properties are measured in terms of mean, median and standard deviation. These statistical properties of selected random variables represent the range of 3-story structures for which fragility curves are constructed. Table 7 demonstrates statistical properties of selected random variables.

Table 7: *Statistical Properties of Selected Random Variables*

Random Variables	Mean	Median	Standard Deviation
Yield base shear coefficient (V_y/W)	0.34	0.33	0.076
Yield global drift ratio (θ_y)	0.011	0.0011	0.00014
Ratio of the post elastic slope to the elastic slope (α)	0.054	0.055	0.011

17. GENERATION OF GROUND MOTION

The time history plotting of earthquake motions for Dhaka city are developed using MCEER project in Cornell University, Ithaca, NY. Using source distance 91.779 km, soil type 4 and changing magnitude fourteen non-stationary earthquakes of different PGA values are generated. The various parameters of these fourteen earthquakes are described in table 8.

Table 8: *Parameters of Generated Earthquake*

Source Distance (km)	Soil Type	Magnitude	PGA (g)
91.779	4 (NEHRP class D)	6.00	0.10
91.779	4 (NEHRP class D)	7.00	0.20
91.779	4 (NEHRP class D)	7.50	0.30
91.779	4 (NEHRP class D)	7.80	0.40
91.779	4 (NEHRP class D)	8.30	0.50
91.779	4 (NEHRP class D)	8.50	0.60
91.779	4 (NEHRP class D)	8.85	0.70
91.779	4 (NEHRP class D)	9.00	0.80
91.779	4 (NEHRP class D)	9.20	0.90
91.779	4 (NEHRP class D)	9.50	1.00
91.779	4 (NEHRP class D)	9.55	1.10
91.779	4 (NEHRP class D)	9.70	1.20
91.779	4 (NEHRP class D)	9.85	1.30
91.779	4 (NEHRP class D)	9.95	1.40

18. DEVELOPMENT OF FRAGILITY CURVES FOR 3 STORY STRUCTURES

The set of earthquake records generated by the simulation procedures comprising the ground motion data is then utilized to compute the dynamic time-history response of the developed models. The SAP 2000 finite element code is used in order to simulate the state of damage of each structure under ground acceleration time-history. The global drift ratios are calculated by dividing the maximum value of the roof displacement, δ_{top} by average building height. In this case the average building height is 30 ft or 360 inches.

The maximum global drift values computed by the above procedure are then assumed to represent the seismic performance of the investigated concrete frames. Using the damage threshold levels defined in Table 4.3, the exceedance probabilities of that particular fragility curve were computed from the PGA versus maximum global drift scatters. The probability distribution function is the standard normal or lognormal distribution in most cases (Shinozuka et al., 2000; Kircher et al., 1997). From the central limit theorem it is known that if a random variable X is made of the sum of many small effects then X might be expected to be normally distributed. The scatter diagrams are clustered for different PGA intervals and it is found that global drift ratios have standard normal probability distribution for each and every PGA intervals. The global drift percentiles greater than a given damage threshold level are computed by using the normal distribution to estimate the exceedance probabilities of the fragility curves. Figure 6 shows the fragility curves for three damage states i.e. Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) for 3 story two dimensional structures in Dhaka City for earthquake source at 91.779 km from Dhaka.

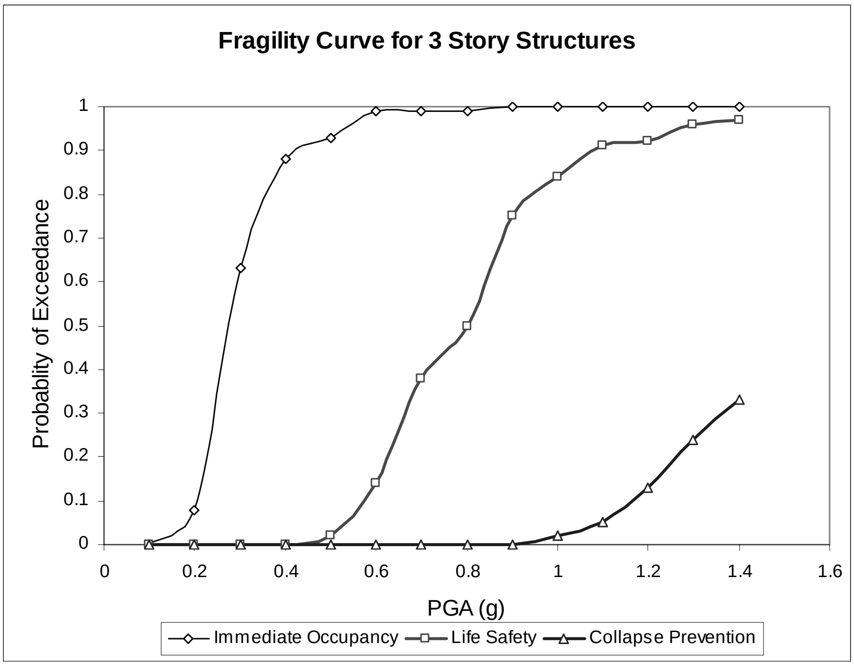

Fig 6: *Fragility Curves for 3 Story Structures*

19. CONCLUSION

The procedure described above is one of the ways to develop fragility curves analytically. In this case fragility functions are derived from the data of groups of existing building stocks. The curves that are produced in the above mentioned method can be used for regional loss estimation studies in different seismic prone zones of Bangladesh. The parameters that are considered as uncertain in the analysis are the construction parameters, yield strength, ground motion intensity and global drift that is used to identify the damage limit states.

There are different approaches that can be followed in construction of fragility curves. But it is imperative that during the construction of the fragility curves for building structures, it is necessary to consider the country-specific characteristics of the building stock. The reason is that the construction practice may differ substantially in different countries and since the differences in the country specific characteristics of building structures are directly reflected in the fragility curves, this may lead to erroneous estimates in terms of earthquake damage and loss.

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