APPLICATION OF PROBABILISTIC METHODOLOGY FOR DAMAGE ESTIMATION OF MID-RISE RC BUILDING DESIGNED IN LIMA

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ABSTRACT:

This study is oriented to implement the probabilistic approach and outline a procedure for the calculation of structural damage in one case study: an eight stories RC framed building which is taken as a representative structure of the mid-rise RC buildings currently designed in Lima under the Peruvian Seismic Code. The probabilistic methodology is developed in order to assess the seismic performance under a new generation of performance based earthquake engineering. Three of the four steps involved in the above methodology are included in this study. Hazard Analysis, represented by the peak ground acceleration as intensity measure; Structural Analysis, with lateral drift as engineering demand parameter; and Damage Analysis, where the damage is measured by the probability of exceeding certain damage state. The uncertainties are involved in each step as follows: In the hazard analysis, actual records are selected and scaled to the intensity target values. These records include the actual uncertainties in some parameters such as duration and frequency contents. For the Structural Analysis, the uncertainty is introduced by means of a random variable for the yielding point in the capacity curve calculated for the structure; which is used as envelope in the nonlinear single degree of freedom analysis for the frame. The uncertainty is produced by simulation on the material capacity. Finally, the damage uncertainty is given by the fragility functions, which relate the structural demand parameter (lateral drift) with the structural damage states for the building. The damage probabilities are integrated for all the selected records and the values of yielding strength. The final result is given in terms of the probability of being in each damage state given a PGA value and plotted in fragility functions.

KEYWORDS: probabilistic assessment, structural damage, fragility functions, Peruvian seismic code.

1. INTRODUCTION

1.1 Development on Loss Estimation Methodologies

The prediction of Seismic Loss for buildings is currently a matter of research in most of the countries located in earthquake prone areas. Both safety and functionality are intended to be fulfilled since the first design stages. Every study follows a similar flow during the loss calculation, from intensity measurements, structural response, damage estimation and finally the loss assessment un terms of suitable parameters which should be indicators not only of the structural damage, but also take into account nonstructural elements and contains. Besides the direct seismic loss produced in the structure, additional indirect loss can be further calculated by means of functions which estimate interruption of activities, casualties, injuries, or even the consequent social impact.

1.2 Probabilistic Approach Methodology

The Pacific Earthquake Engineering Research Center (PEER) is currently working on the development of a new generation of Performance Based Earthquake Engineering. In this sense, the ATC-58, is a project aimed to develop guidelines in order to design buildings with performance objectives that are both predictable by the design professional and meaningful and useful for the decision makers who select or approve the performance objectives. PEER divided the performance assessment process into logical elements that can be studied and solved in a rigorous and consistent manner (Moehle, J, Deierlein, G, 2004). Four stages are defined in the process which defines, in a probabilistic manner, the features of the involved parameters. The outcome of each step is mathematically characterized by four variables: Intensity Measure (IM), Engineering Demand Parameter (EDP), Damage Measure (DM), and Decision Variable (DV):

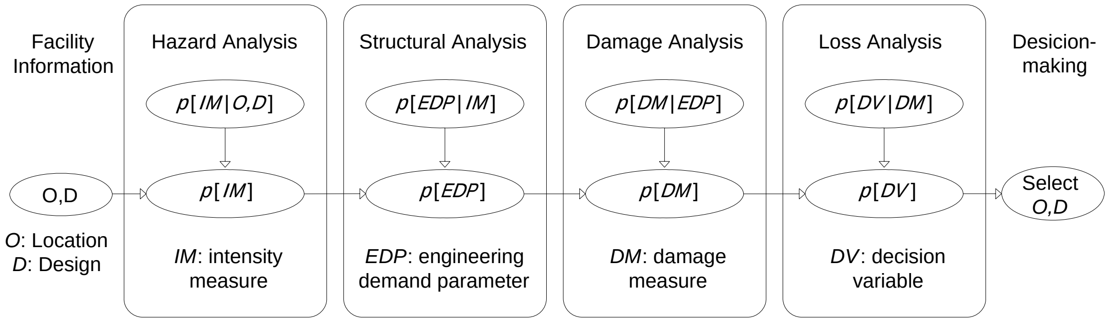

Figure 1. Underlying Probabilistic Framework (ATC-58)

The procedure of this study includes three of the mentioned steps to estimate the structural damage of the building. They can be summarized as follows:

1.3 Ground Motion Processing

In this step the seismic motion is characterized by means of actual accelerograms recorded in Lima. Every accelerogram has been recorded in firm soil, which is the most common soil type in the city. These records will contain the uncertainties of motions typical for Lima. The accelerograms are scaled in order to have records with peak response acceleration as needed for the damage estimation. The target value to match is the response acceleration for the fundamental period of the structure, taken from the PGA uniform hazard curve. The response of every original record at the fundamental period is set to this target value by means of a factor, which scales the record. The output of this step is a set of records scaled to the target PGA value

1.4 Structural Response

This response is calculated by means of nonlinear dynamic analysis on SDoF systems. Force restoring envelope relationship is taken as an approximated bilinear curve from the capacity curve for the structure, which is calculated from pushover analysis. The uncertainty considered in the structural response is introduced by the variation of the yielding force capacity produced by material properties uncertainty. This variation is represented by a set of realizations fitting a probability density function (PDF) for the yielding force capacity. The PDF is previously calculated from the results of simulation, considering the material properties variation. The output of this step is a set of peak displacement values for SDoF systems under the previously scaled accelerograms and the probability related to these values.

1.5 Structural Damage Calculation

The structural damage is calculated from fragility relationships whose threshold values are set from the capacity curve of the building representing damage states ranging from No-damage to Collapse. The probability of being in every damage state is calculated having as input parameter the value of peak displacement from analysis. The probability of being in each damage state is summed up for every peak displacement value considering the probability in relation with the previous step. The final result is the probability of being in each damage state for the structure, which is subjected to a given PGA. The fragility can then be calculated performing the process as many times as needed to cover the range of the selected PGA. Figure 2 explains the process for damage estimation implemented in this study.

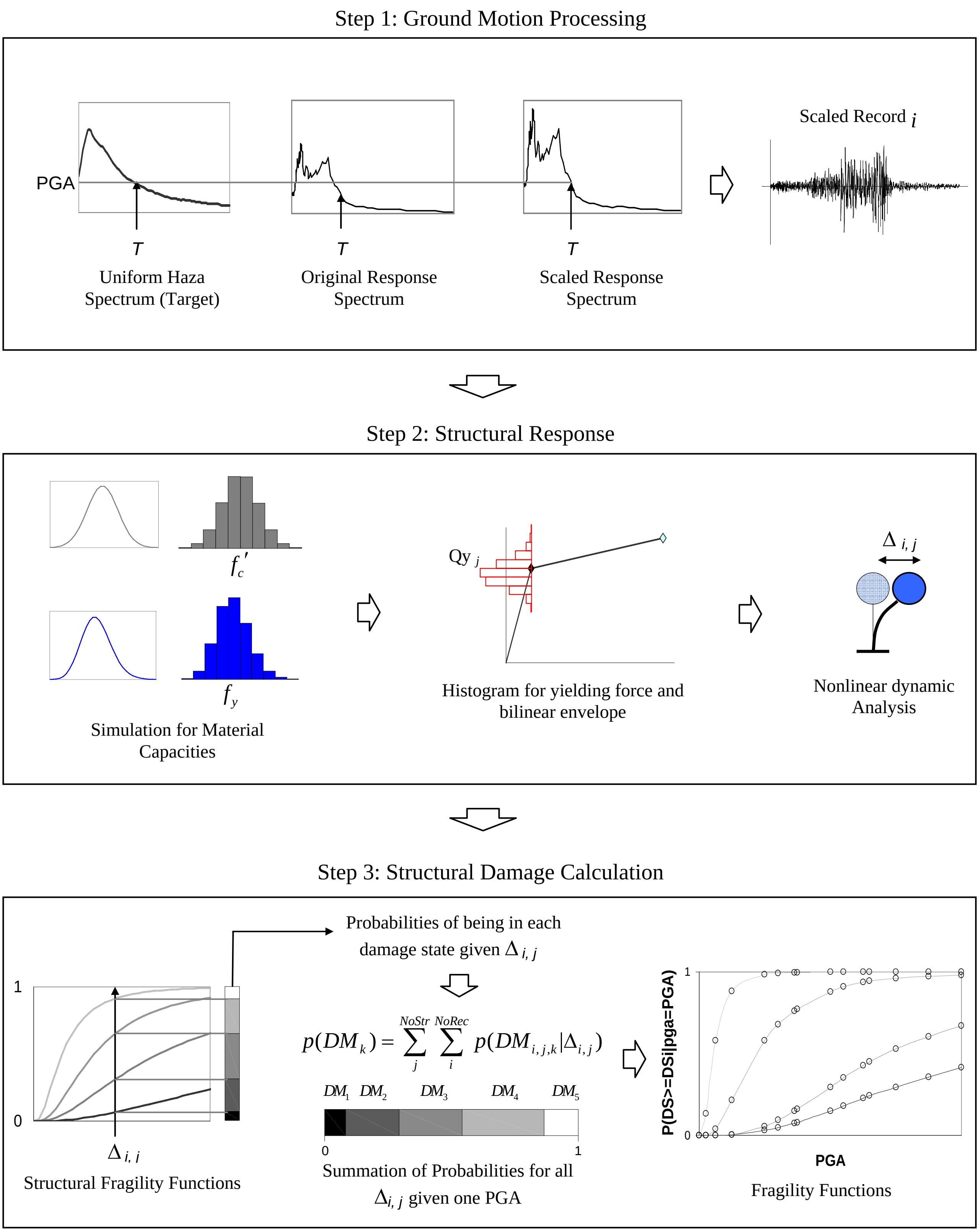

Figure 2. Scheme of Damage Estimation Procedure

2. EARTHQUAKE GROUND MOTIONS

2.1 Selection of Ground Motion Records

Twenty accelerograms from *CISMID - Strong Motion Accelerograph National Network* in Lima were selected. The records information is shown in Table 2.1.

Table 2.1 Selected Ground Motion

Code	Date	Coordinates		Magnitude	PGA (gals)		Site
		Latitude S	Longitude W		EW	NS	
PRQ-5101311139F	10/31/1951	12	78	5.5	60	45.7	Dense Gravel
PRQ-6610171641F	10/17/1966	10.7	78.7	6.3	180.6	269.3	Dense Gravel
PRQ-7005311523F	05/31/1970	9.2	78.8	6.6	104.8	97.7	Dense Gravel
PRQ-7111291514F	11/29/1971	11.2	77.8	5.3	53.5	86.5	Dense Gravel
PRQ-7410030921F	10/03/1974	12.3	77.8	6.6	192.5	179	Dense Gravel
SCO-7410030921F	10/03/1974	12.3	77.8	6.6	192.3	207.1	Dense Gravel
UNI290491	04/29/1991	11.26	77.67	5.7	53.6	49.1	Dense Gravel
PRQ290491	04/29/1991	11.26	77.67	5.7	35.2	27.5	Dense Gravel
UNI180493	04/18/1993	11.75	76.62	5.8	129	94.2	Dense Gravel
CMS-0305281626F	05/28/2003	12.51	77.19	5.1	143	118	Dense Gravel

2.2 Scaling the Set of Accelerograms to the Sa Target Values

The accelerograms are scaled by means of a scale factor (*SF*), a numerical value, that modifies the acceleration history of the record in order to match the spectral response ordinates of each record to the target values from the uniform hazard spectrum, both corresponding to certain period. The scale factor is calculated as the quotient of the acceleration taken from the hazard spectrum ordinate and the spectral ordinate calculated from the original accelerogram (Shome and Cornell, 1999):

$$SF = \left(\frac{(Sa/g)_{TR}}{(Sa/g)_{Sp}} \right) \tag{2.1}$$

The target period is taken as the fundamental period of the elastic structure. The available information for Lima includes uniform hazard spectrum for the 475 years return period and 5% damping (Monroy et al, 2005), calculated from seismological information.

3. STRUCTURAL MODEL OF BUILDING

3.1 Characterization of a Representative Peruvian Building

The selected structure that represents a typical and regular Peruvian building was taken as an eight story building with 3 bays@6m in X direction and 3 bays@5m in Y direction. The inter-story height is 3m for every story. Reinforced concrete frames form the lateral resistant system in both directions. For the analysis, one of the interior frames in X direction was considered as shown in figure 5. The building is assumed as a residential facility, founded in firm soil.

3.1.1 Gravity Loads

The gravity loads were considered as distributed dead load from slab weight ($3000 N/m^2$), non-structural partitions ($1000 N/m^2$), and floor finishing ($1000 N/m^2$). The total dead weight is $5000 N/m^2$ plus the structure selfweight. The live load was considered as $2500 N/m^2$.

3.1.2 Seismic Load
The seismic load for design is taken in accordance with the Peruvian Code of Earthquake Resistant Design. The spectral shape for the design spectrum considering the building structural properties is shown in Figure 3.

3.1.3 Design
Square sections of 0.60x0.60 m^2 were selected for both exterior and interior columns and for every story. The beams were taken as rectangular 0.30x055 m^2 for every story. The steel ratio in every case is provided according to forces demand and provisions of Peruvian Regulation for Construction for earthquake design of reinforced concrete buildings.

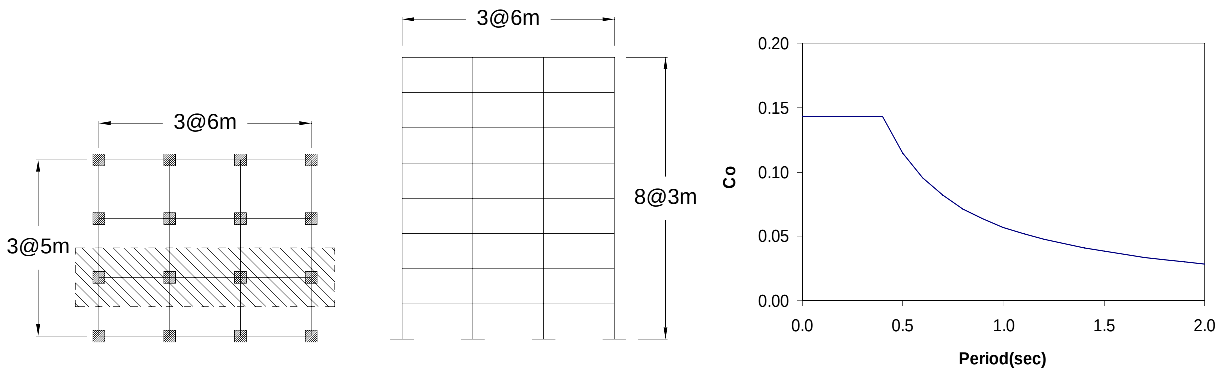

Figure 3. Frame Plan and Elevation and Design Spectrum according to Peruvian Design Code

3.2 Nonlinear Structural Analysis

3.2.1 Pushover Analysis
The pushover curve is taken as the envelope for a further SDoF nonlinear dynamic analysis. This procedure has been already performed to determine displacement based fragility functions (Akkar et al, 2005). The program used for the analysis is IDARC2D V6.1 (Reinhorn, 2006). The material properties for concrete and steel reinforcement considered in the analysis are assumed as shown in table 3.1; where for the steel: f_y is the yield strength, f_u is the ultimate strength, E_s is the Young's Modulus, E_{sh} is the Modulus of strength hardening and ε_{sh} is the strain at start of hardening. For concrete: f'_c is the compressive strength, E_c is initial Young's Modulus, ε_0 the strain at maximum strength, and f_r is the stress at tension cracking.

Table 3.1 Material Properties for Nonlinear Analysis

Steel		
f_y	441	Mpa
f_u	662	Mpa
E_s	196000	Mpa
E_{sh}	3489	Mpa
ε_{sh}	0.03	
Concrete		
f'_c	24.5	Mpa
E_c	19600	Mpa
ε_0	0.002	
f_r	4.41	Mpa

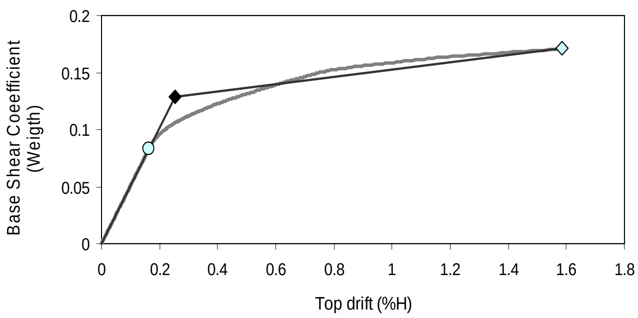

Figure 4. Bilinear Curve

As can be seen in figure 4, the bilinear curve has the same initial stiffness as the pushover curve and the collapse point was set on the ultimate drift (approx. 1.6%). The capacity curve is transformed into a SDoF system by taking into account the dynamic characteristics of the first mode of vibration.

3.2.2 Structural Uncertainty and Dynamic Analysis

The structural strength (Yielding) of the building is modeled as variable, keeping the stiffness as a constant for every strength value (random strength – constant stiffness approach) with constant value of post-yielding stiffness as calculated from the bilinear curve. The yielding strength was considered as a random variable with log-normal PDF, which is calculated by fitting the PDF to the results of 100 realizations of pushover analysis considering variation on the f'_c and f_y stress values. Normal distribution with coefficient of variation of 0.175 and lognormal distribution with coefficient of variation of 0.1 were considered for f'c and fy random variables, respectively (Lee and Mosalam, 2005). The simulation results showed a coefficient of variation of 0.08 for the yielding force value, which is similar to values obtained by previous studies for behavior analysis of RC flexural members (Ellingwood et al, 1980, Porter, 2002). The lateral drift is chosen as the structural response parameter for the calculation of the structural damage.

4. STRUCTURAL DAMAGE CALCULATION

As a probabilistic approach procedure, the structural damage is assessed by means of fragility functions, which relate the probability of being in a specific level of damage, given a demand structural parameter. In this study, the fragility functions are developed from the building capacity curve, where the thresholds are related to stages of structural behavior. Five damage states are recognized in the history of loading from the pushover curve:

- No damage: Before cracking.
- Light damage: After initiation of cracking.
- Moderate damage: Yielding of steel reinforcement for beam elements.
- Severe damage: Long post-yielding deformation.
- Near Collapse: Failure mechanism initiation and structural instability.

Table 4.1 Drift Threshold for Damage States

State	Drift Threshold (%H)
Light	0.06
Moderate	0.26
Severe	0.92
Near Collapse	1.59

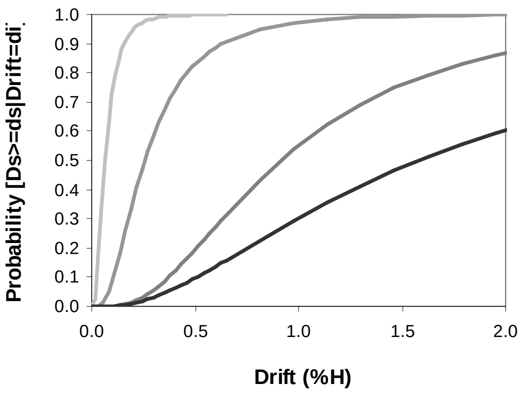

Figure 5. Fragility functions for structural damage

The fragility functions are assumed as log-normal functions with median and lognormal standard deviation as parameters. The median are taken as the drift threshold (from pushover analysis) and the log-normal standard deviation are typically taken from the set of structural fragilities developed in HAZUS (FEMA, 1999) after choosing fragilities for buildings with similar characteristics (Structural Type and Strength Design Level). The values for the S.D. are: $\beta = 0.7$ for light, moderate and severe damage levels and $\beta = 0.89$ for the near collapse threshold. Assuming these parameters for the log-normal functions, the structural fragility curves are constructed (Figure 5).

5. CALCULATION AND RESULTS

The whole process of damage assessment by the probabilistic approach requires large number of calculations. The probability of being in a damage state for each realization is defined as:

$$p(DM_{i,j,k}) = p(DM_k)\,p(EDP_j)\,p(IM_i) \tag{5.1}$$

where:

$p(IM_i)$: Probability of occurrence of an intensity *i* (record).

$p(EDP_j)$: Probability of each value of strength *j* according PDF.

$p(DM_k)$: Probability of experience damage state *k* given a structural response (EDP).

The intensity IM is represented by the peak ground acceleration and the strong motion uncertainty is included in the actual records, then each realization is considered to have the same probability of occurrence given by the inverse of the number of records NoRec. Each value of EDP resulting from the nonlinear dynamic analysis for each record has the probability corresponding to the PDF for the strength value considered in such analysis; being NoStr the total number of strength values taken in the lognormal probability distribution. DM represents a damage state and the probability of experiencing such a state given a level of structural response is calculated from the fragility functions. The expected probability of being in each damage state (DM_k) is calculated as the sum of the damage probabilities over all the realizations (NoRec x NoStr):

$$p(DM_k) = \frac{1}{NoRec}\sum_{j=1}^{NoStr} p(DM_{j,k})\,p(EDP_j) \tag{5.2}$$

In order to evaluate representative values of intensity from hazard studies (Castillo, J, Alva, J, 1993) three values of peak ground acceleration corresponding to periods of return of 50, 475 and 970 years respectively for frequent, rare and very rare earthquakes are shown and the probabilities of damage are calculated.

Table 5.1 PGA for three levels of Hazard and Probability of being in each damage state (in percentages) for three levels of intensity.

Ground Motion	Return Period (years)	PGA (g)
Frequent	50	0.2
Rare	475	0.4
Very rare	970	0.5

PGA	0.2 g	0.4 g	0.5 g
No Damage	1.5	0.11	0.03
Light Damage	40.3	11.8	6.3
Moderate Damage	52.7	58.4	50.9
Severe Damage	2.5	14.5	19.9
Near Collapse	2.9	15.0	22.7

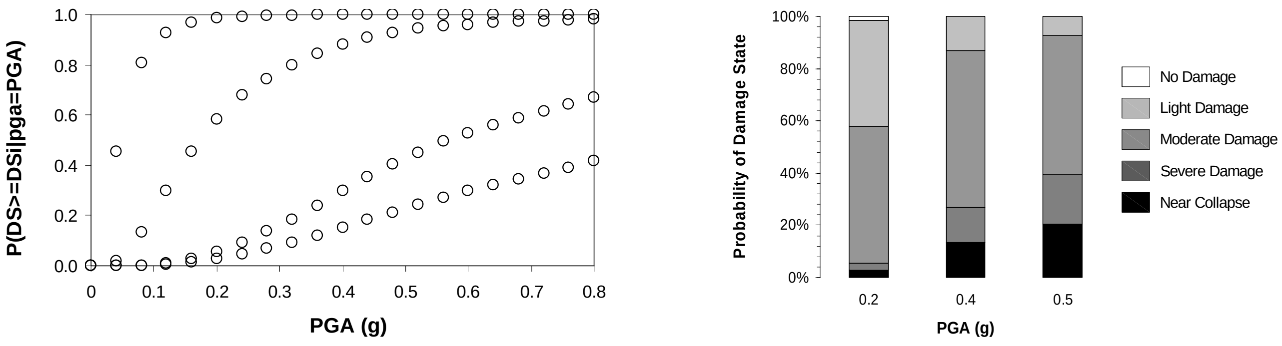

Figure 6 Fragility curves for structural damage and Damage Probabilities for PGA = 0.2, 0.4, 0.5 (g)

Results show that for the frequent earthquake, the expected damage is around 93% in light and moderate level. For the rare earthquake which corresponds to the design level motion, the building experiences 70% damage in light and moderate level. For the very rare earthquake with return period of 970 years, the damage is 71% in moderate and severe level.

6. CONCLUSIONS

The results show an expected response according to the design philosophy of the Peruvian code. The structure behaves practically with light and moderate damage for the 50 years earthquake. The damage for the rare and very rare earthquakes is equal or lower than moderate and the probability of collapse is kept under 23% for the 970 years earthquake.
In the structural response step, the model of pushover envelope - SDoF dynamic analysis may be changed into a lumped multi-mass system. This model which keeps the simplicity and allows taking into account some important features such as higher modes participation and story strength distribution, results in an efficient calculation saving computational effort and catching the overall response of the structure by means of parameters such as peak floor displacement or peak floor acceleration.
The outlined procedure applied in the study takes into account the uncertainties in the calculation process, from the seismic motion, the structural response and the damage evaluation; however, the analytical treatment of the damage estimation is based on analysis and the results should be calibrated and fit by real data in further studies.

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