RELIABILITY ESTIMATION OF INCREMENTAL RETROFITTED STRUCTURES CONSIDERING CUMULATIVE SEISMIC DAMAGES

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Abstract

Incremental retrofitting consists of two or more retrofitting stages scheduled discretely during the lifetime of the building in order to decrease initial costs and to avoid long period of activities interruption. Retrofitted building achieves limited or partial performance objectives in each incremental retrofitting. The objective of this paper is to assess the reliability of incremental retrofitted buildings in seismic zones considering the accumulated damages with simplified probabilistic method. In this work, we consider that structural damage is accumulated in a series of seismic ground motions until a damage level is reached or exceeded. The proposed method consists in 7 steps: (1) Definition of the global damage states (GDS) that results of the division of the capacity curve of the building in a non-damage state, obtained through a static nonlinear analysis (Pushover Analysis) according to Vision 2000, (2) Estimation of the modified moment-rotation curves of each damaged element of the building associated to each defined GDS with an approximate approach. This method is based on the evaluation of the plastic hinge rotation through the estimation of the moment-rotation curve for each structural element, (3) Estimation of the building capacity curve for each GDS considering the damage elements through their modified moment-rotation curves, (4) Elaboration of seismic fragility curves for each GDS with the capacity spectrum assessment method using the software FRACAS, which uses inelastic response spectra derived from earthquake ground motion accelerograms to construct fragility curves, (5) Determination of damage transition probability matrices for different seismic intensity levels using the values obtained in the previous step, (6) Implementation of a Markov Chains Model to obtain the probability distribution function of each GDS after a determinate number of earthquakes, and (7) Estimation of the structural reliability taking into account the damage accumulation process during probable future earthquakes assuming a Poisson process. This methodology is applied to compare the reliability of two retrofitted Peruvian school buildings, one with partial performance objectives and the other with full performance objectives. It is obtained that the reliability at the end of the service life of the first one is 20% less than the reliability of the second one. We recommend this methodology to assess reliability for life cycle cost benefit analysis of new and retrofitting projects.

Keywords: accumulated damage, nonlinear analysis, structural performance, FRACAS, Markov Chains, incremental retrofitting

1. Introduction

In 2014, the Ministry of Education (MINEDU) and the World Bank initiated a national plan to implement incremental retrofitting techniques for 780 PRE school buildings in order to reduce seismic risk [1]. These buildings, which represent approximately a quarter of the total number of schools in Peru, have suffered great damage in past earthquakes due to excessive flexibility in their longitudinal axis, because these buildings have been designed with the Peruvian Seismic Resistant Design Standard of 1977, which did not contemplate adequate displacement restrictions.

The Pontifical Catholic University of Peru (PUCP) and the National University of Engineering (UNI), in charge of the project, proposed three incremental retrofitting techniques in order to bring the structure to the same level of performance as an essential building. The structural response and damage states of present and retrofitted structures to occasional and rare earthquakes were determined through non-linear static analysis (Pushover analysis) and non-linear time-history analysis (IDA). In recent studies by Loa et al [2] it was found that with any type of retrofitting technique the structure could only reach a state of damage of operational and functional due to the occasional and rare earthquakes, respectively.

The present paper will evaluate the reliability of two school buildings with an incremental retrofitting technique, one with a partial performance objective and the other with a full performance objective, considering the accumulation of damage by earthquake with a simplified probabilistic method, in order to determine the state of damage of the buildings due to the effect of the occurrence of probable earthquakes throughout their service life.

2. Theoretical Framework

2.1 Incremental retrofitting

Incremental retrofitting appears as an alternative to treat buildings with high seismic risk. This is the technique that reduces or eliminates expenses due to the interruption of educational programs, being the optimal option for reinforcement in educational buildings. [3]

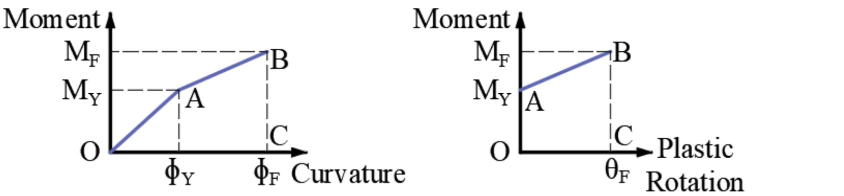
2.2 Moment curve - rotation of the structural element

The behavior of the elements is represented in moment-curvature relations, which allows to compare and identify the different stages that the section will suffer until failure, including over-strength and ductility. Fig. 1 shows a moment - curvature diagram in which the yield point (Φ_Y, M_Y) and the failure point (Φ_F, M_F) are distinguished. This simplified moment - curvature curve is transformed to a moment - rotation curve, plastic rotation θ, using the following expressions:

$$\theta = l_p * (\phi - \phi_Y) \qquad \phi_Y \le \phi \le \phi_F \tag{1}$$

$$\theta_F = l_p * (\phi_F - \phi_Y) \tag{2}$$

In the model, damage is not considered for the elastic range of the deformations, as shown in Fig. 1.b, where Φ_Y and Φ_F are the yield and fault curvatures respectively, l_P is the plastic hinge length which is considered constant at each instant and θ_F is the plastic rotation of failure.



(a) Moment - Curvature diagram (b) Moment - Rotation diagram

Fig. 1 – Models of the behavior of a structural element

2.3 Non-linear static analysis

The procedure of the non-linear static analysis (Pushover analysis), consists of elaborating a mathematical model of a structure, initially without plastic hinges, which is exposed to lateral forces that act at floor level until some elements reach their elastic limit, then the structure is modified to take into account the reduced resistance of elements where plastic hinges have been produced. A distribution of lateral forces is again applied until additional elements produce plastic hinges. This process is continued until the structure becomes unstable or until a predetermined limit is reached. [4]

3. Methodology

3.1 Definition of the Global Damage States (GDS)

The Global Damage States (GDS) represent a limit condition established according to the possible damages, threat on the security of the occupants and the functionality of the building after an earthquake. [5]

The GDS correspond to defined sectors of the structure's capacity curve, that is obtained through a non-linear static analysis. To divide the capacity curve, the effective yield displacement (Δe) and the inelastic displacement capacity (Δp) must first be defined. The effective yield displacement (Δe) corresponds to the instant in which a maximum of 50% of the inelastic incursions forming the failure mechanism have occurred, without the deformation in any section exceeding 150% of its yield deformation. The inelastic displacement capacity (Δp) corresponds to the lateral displacement of the structure from the effective yield point to the collapse. The inelastic section of the capacity curve is divided into four sectors defined by fractions of the inelastic displacement (Δp).[5]

As shown in Fig. 2, the defined GDS are: Operational (DS1), Functional (DS2), Life Safety (DS3), Near Collapse (DS4) and Collapse (DS5).

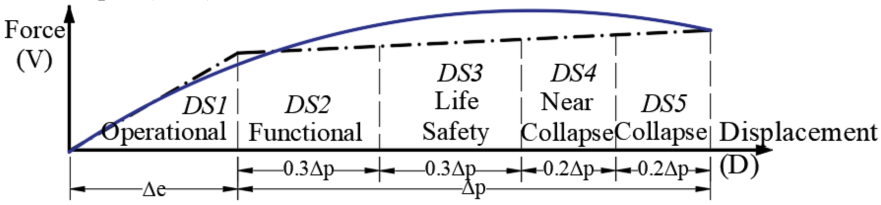


Fig. 2 – Division of the capacity curve

3.2 Estimation of the modified moment-rotation curve for each damaged element of the building associated to each defined GDS

The damage in the elements, is manifested as a decrease in the rigidity and the resistant capacity of the element, being appreciated a progressive formation of plastic hinge in the ends of the element. This simplified model is based on the hysteretic model proposed by Campos & Esteva [6], in which each loading and unloading cycle is associated to the formation of plastic rotation in the element.

The proposed model consists in that given a plastic rotation formation θ_i associated to a moment M_i, according to the moment – rotation diagram, the moment $M_{i'}$ is defined, which is associated to the loss by damage, using the following expression:

$$M_{i'} = (1-\varepsilon * D_i) * M_i \tag{3}$$

Where D is the accumulated damage parameter and ε is the damage index, defined with the following expressions:

$$D_i = \frac{\theta_i}{\theta_F} \tag{4}$$

$$\varepsilon = 1 - e^{-\alpha * D_i} \tag{5}$$

Being α a constant of adjustments, which due to several tests made by Campos & Esteva [6], is considered a value of α = 0.0671. The modified moment-rotation diagram is defined by the O'B'B curve, as shown in Fig. 3.

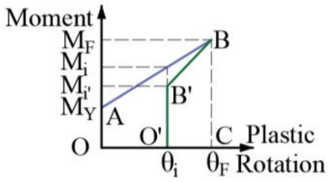


Fig. 3 – Modified moment – rotation curve of a structural element

3.3. Estimation of the building capacity curve for each GDS considering the accumulated damage in elements

For each GDS of the building, some elements must present damage (formation of plastic hinges), these damages are estimated as indicated in item 3.2. Then, considering the damage on the elements through the modified moment-rotation curve, the building capacity curve is determined through a non-linear static analysis for each defined GDS.

3.4 Elaboration of seismic fragility curves for each GDS with the capacity spectrum assessment method using the software FRACAS

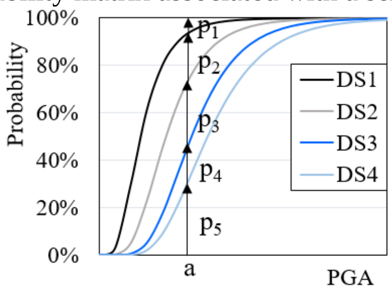
The capacity spectrum evaluation methodology, first developed by Rossetto & Elnashai [7], allows the generation of fragility curves directly using seismic records from which the elastic and inelastic demand spectra are calculated in order to find the performance point.

The proposed approach is highly efficient and allows the fragility curves to be derived from the analysis of a structure subjected to a series of seismic records with different characteristics. In this way, the method can explain the effect of variability in seismic demand and structural characteristics on the damage statistics simulated for the type of structure, and evaluate the associated uncertainty in predicting fragility.

Rossetto et al [8] developed the software FRACAS (FRAgility through CApacity Spectrum assessment) based on the described methodology, which allows sophisticated capacity curve idealizations, the use of several hysteric models for systems with a single degree of freedom in the inelastic calculation of demand and the construction of fragility functions by using several statistical model fitting techniques.

3.5 Determination of damage transition probability matrices for different seismic intensity levels

For each GDS, the fragility curves are obtained for each damage state. Given a seismic intensity, the probability of the damage states is obtained. This set of probabilities for each GDS, is a row of the damage transition probability matrix associated with a seismic intensity, as shown in Fig. 4.



PGA "a"

	DS1	DS2	DS3	DS4	DS5
GDS1					
….					
GDSi	**p_1**	**p_2**	**p_3**	**p_4**	**p_5**
….					

Fig. 4 – Elaboration of the damage transition probability matrix

3.6 Implementation of a Markov Chains Model to obtain the probability distribution function of each GDS after a determinate number of earthquakes

The failure of a structure occurs at the instant it passes from an "operational" state to a "non-operational" state. Assuming that the damage accumulated due to the earthquake can be modeled as a Markov process and that the structure presents "n" damage states. The damage state 1 (DS1) is the one in which the damage is zero and the nth damage state (DSn) is the failure, this is, when the structure stops operating, as shown in Fig. 5.

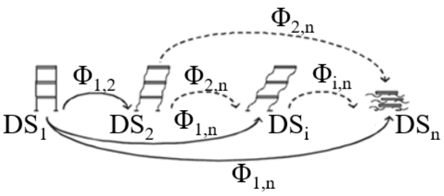


Fig. 5 – Damage states of a structure

Moreover, the reliability function can be obtained from the damage probability distribution. Heredia-Zavoni et al [9] propose a damage model that depends on the state of damage before an event and the maximum inelastic deformation reached in the inelastic response semicycles. Subsequently, Santa-Cruz & Heredia-Zavoni [10] use such a damage model and simulated accelererograms to find the conditional probability that the structure reaches the damage state d_j after the intensity event A = a given that the damage state before the event was d_i, hence:

$$\Phi^{a}_{i,j} = P[D^{a} = d_j \mid D^{a}_{0} = d_i] \tag{6}$$

ϕ^k is the transition matrix of the damage accumulation process for the kth event of intensity "a":

$$\phi^k(a) = \phi^k = \{\phi^{a}_{i,j}\} \qquad i, j: 1, 2, 3, \ldots, n \tag{7}$$

Also, being v_o the total number of events considered in the analysis per unit of time and v(a) the rate of exceedance of accelerations of the site. The probability density function of the intensity of any event $f_A(a)$ is defined as:

$$f_A(a) = -\frac{1}{v_o} * \frac{dv(a)}{da} \tag{8}$$

Also:

$$\Pr ob(a_m) = f_A(a) * \Delta a \tag{9}$$

Considering "na" number of accelerations in the analysis:

$$E[\phi] = \sum_{m=1}^{na} \phi(a_m) * \Pr ob(a_m) \tag{10}$$

The transition matrix for N events is:

$$\Psi^N : \phi^1 * \phi^2 * \ldots * \phi^N \tag{11}$$

The term $\psi^N_{i,j}$ from the transition matrix ψ is the conditional probability that the Nth event will cause d_j damage, given that the state before the first event was d_i.

Be V_o the initial probability vector of the damage state:

$$V_o = \{\Pr ob(D(t_o) = d_1), \Pr ob(D(t_o) = d_2), \ldots, \Pr ob(D(t_o) = d_n)\} \tag{12}$$

And V_N the probability vector at the end of Nth event:

$$V_N = \{\Pr ob(D(t_N) = d_1), \Pr ob(D(t_N) = d_2), \ldots, \Pr ob(D(t_N) = d_n)\} \tag{13}$$

The probability vector at the end of the Nth event, results:

$$V_N = V_o * \Psi^N \tag{14}$$

3.7 Estimation of the structural reliability taking into account the damage accumulation process during probable future earthquakes assuming a Poisson process

Assume that for an interval [0,t], the number of N events is known and that the occurrence of an earthquake is independent of the previous earthquake, however, the intensities of the seismic events that will occur are unknown. Therefore, the matrix ψ is unknown and the vector V_N is random:

$$E[V_N] = V_o * E[\Psi^N] = V_o * E[\phi^1] * E[\phi^2] * ... * E[\phi^N] \quad (15)$$

Therefore:

$$E[V_N] = V_o * \left(\sum_{m=1}^{na} \phi(a_m) * \operatorname{Prob}(a_m) \right)^N \quad (16)$$

The i^{th} term of the vector V_N is the probability that the damage is equal to d_i at the end of the N^{th} event. The n^{th} term of the vector V_N is the probability that the structure fails at time t at the end of the N^{th} event. Finally, the expected value of V at the end of the interval (0.t) is:

$$E[V] = \sum_{m=0}^{\infty} E[V_N] * P(N = m) \quad (17)$$

Where P(N = m) is the probability that the number of seismic events occurred in the place in the period [0,t] is m, considering that the occurrence of the events follows a Poisson type process, it results:

$$P(N = m) = \frac{(v_o * t)^m * e^{-v_o * t}}{m!} \quad (18)$$

The reliability of the structure R(t) results:

$$R(t) = 1 - V_n \quad (19)$$

Where V_n is the n^{th} term of vector E [V].

4. Description of the structures

4.1 Present building

The present building 780 PRE consist in reinforced concrete frames with infill masonry panels in the longitudinal direction and confined masonry walls with RC frames in the transversal direction. The infill panels have a thickness of 0.13 m and the dimensions of the RC elements are presented in Fig. 6. [2]

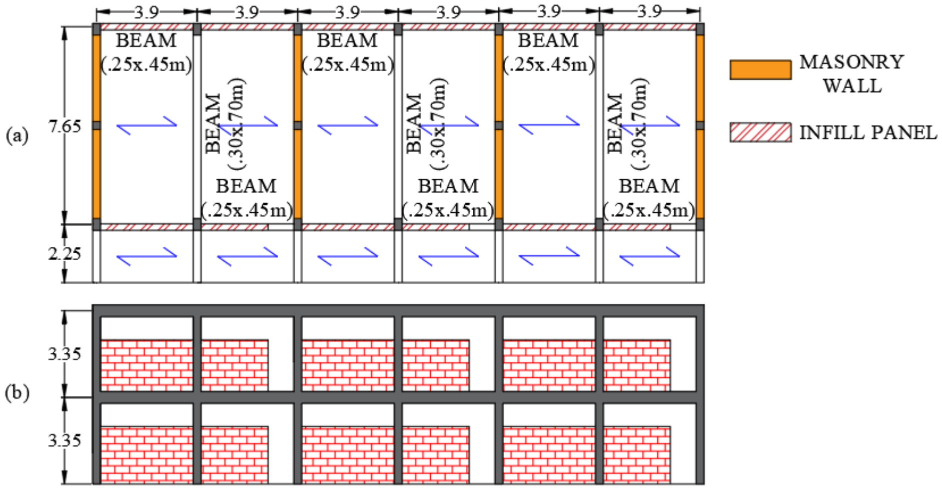


Fig. 6 – Present building 780 PRE: (a) Plant view; and (b) Elevation view [2]

4.2 Incremental retrofitting technique

In this technique, steel frames with concentric bracing are added to the structure. The bracings are welded to the steel frames connected to the RC frame through shear connector. In the first phase, two steel beams will be placed for each axes of the first story. The problem in this stage is that second story will not improve its strength and the retrofitted structure don´t increase its resistance greatly. In the second phase a steel frame is collocated in the second story (Fig. 7), improving its performance as well as the overall performance of the structure. Its failure is related to the flexure. [2]

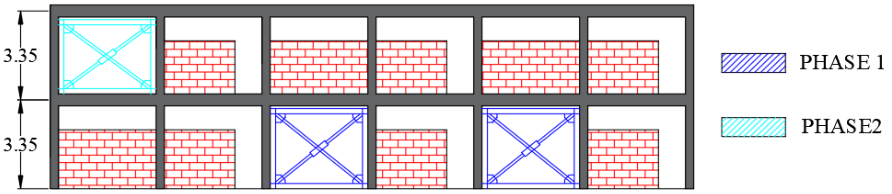


Fig. 7 – Scheme of retrofitting building [2]

5. Application of the methodology

5.1 Definition of the Global Damage States (GDS)

Through nonlinear static analysis and division of the capacity curve, the 5 GDS are determined, as indicated in item 3.1. Fig. 8 shows the capacity curves for the present building (F0), reinforced in phase 1 (F1) and reinforced in phase 2 (F2).

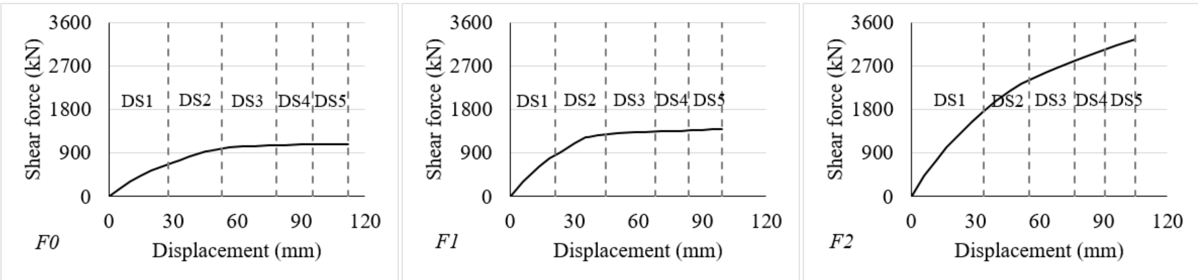


Fig. 8 – Capacity curves of the buildings

5.2 Estimation of the modified moment-rotation curve for each damaged element of the building associated to each defined GDS

For each GDS, the damage of the elements is estimated by obtaining the modified moment - rotation curve, as indicated in item 3.2. As a demonstration, Fig. 9 and Table 1 show the damage estimation of the beam V0.25x0.45m for each GDS.

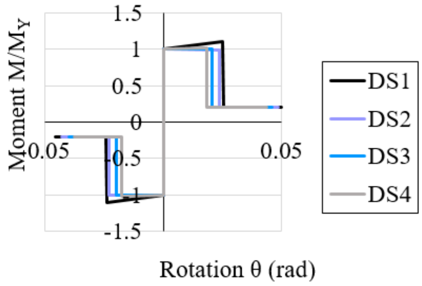


Fig. 9 – Modified moment – rotation curve for each GDS of the beam V0.25x0.45m

Table 1 – Values of the modified moment – rotation curve for each GDS of the beam V0.25x0.45m

POSITIVE							NEGATIVE						
GDS	θ_F (rad)	θ_i (rad)	D_i	ε	M_i/M_Y	$M_{i'}/M_Y$	GDS	θ_F (rad)	θ_i (rad)	D_i	ε	M_i/M_Y	$M_{i'}/M_Y$
DS1	0.025	0	0	0	1.00	1.00	DS1	-0.024	0	0	0	-1.00	-1.00
DS2	0.025	0.0013	0.05	0.0034	1.00	0.99	DS2	-0.024	-0.0011	0.04	0.0030	-0.99	-0.99
DS3	0.025	0.0043	0.17	0.0114	1.01	1.01	DS3	-0.024	-0.0043	0.18	0.0120	-1.01	-1.01
DS4	0.025	0.0065	0.26	0.0173	1.02	1.01	DS4	-0.024	-0.0062	0.26	0.0172	-1.02	-1.01

5.3. Estimation of the building capacity curve for each GDS considering the accumulated damage in elements

The capacity curves of the buildings for each GDS are determined according to item 3.3 as shown in Fig. 10.

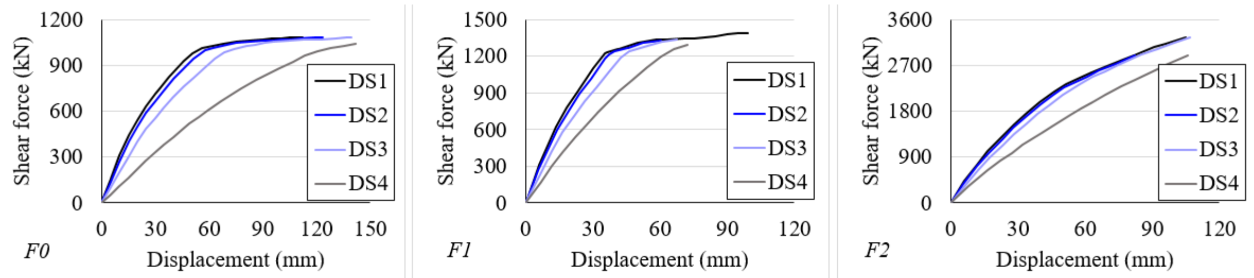


Fig. 10 – Capacity curves associated with each GDS of the buildings

5.4 Elaboration of seismic fragility curves for each GDS with the capacity spectrum assessment method using the software FRACAS

Fragility curves are obtained for each GDS, according to item 3.4. As seismic demand, it is used Peruvian and Chilean seismic records occurred in the last years on the coast, as shown in Table 2. As a demonstration, Fig. 11 shows the fragility curves of the buildings for the GDS - Life Safety.

Table 2 – Peruvian and Chilean seismic records

Seismic Records	Date	PGA (g) EW - NS	Magnitude	Focal Depth (km)	Duration (sec.)
Huacho	17/10/1966	0.18 - 0.27	8.1	24	66
Huaraz	31/05/1970	0.11 - 0.10	6.6	64	45
Lima	03/10/1974	0.20 - 0.18	6.6	13	98
Arequipa	23/06/2001	0.30 - 0.22	6.9	33	199
Pisco	15/08/2007	0.28 - 0.34	7.0	40	218
Tarapacá	13/06/2005	0.73 - 0.54	7.9	111	252
Constitución	27/02/2010	0.54 - 0.35	8.8	30	180
Angol	27/02/2010	0.70 - 0.93	8.8	30	180

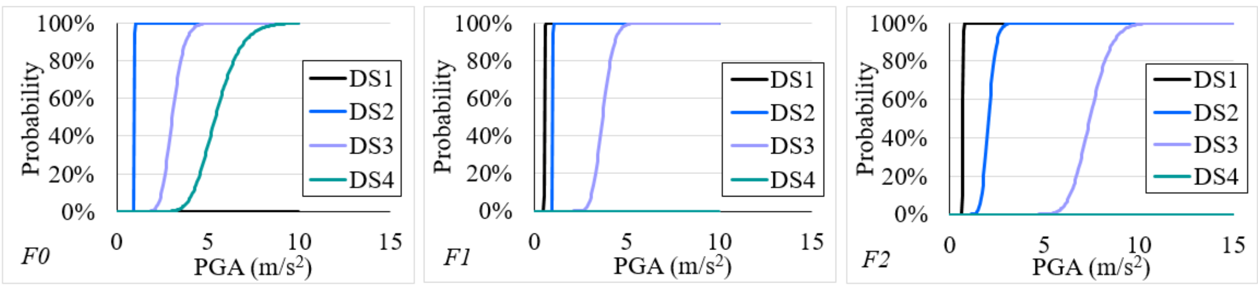


Fig. 11 – Fragility curves of the buildings for the GDS – Life Safety

5.5 Determination of damage transition probability matrices for different seismic intensity levels

Damage transition probability matrices are determined for seismic accelerations from 0.5 m/s^2 (0.05 g) to 10 m/s^2 (1 g), as indicated in item 3.5. In Table 3, it is shown the damage transition probability matrices for an acceleration of 7 m/s^2 (0.7 g) for the buildings.

Table 3 – Damage transition probability matrices for PGA 7 m/s^2

F0 Building	DS1	DS2	DS3	DS4	DS5
DS1	0	0	0.001	0.999	0
DS2	0	0	0.001	0.999	0
DS3	0	0	0	0.103	0.897
DS4	0	0	0	0	1
DS5	0	0	0	0	1

F1 Building	DS1	DS2	DS3	DS4	DS5
DS1	0	0	0.088	0.909	0.003
DS2	0	0	0.040	0.960	0
DS3	0	0	0	1	0
DS4	0	0	0	1	0
DS5	0	0	0	0	1

F2 Building	DS1	DS2	DS3	DS4	DS5
DS1	0	0	0.969	0.031	0
DS2	0	0	0.881	0.119	0
DS3	0	0	0.684	0.316	0
DS4	0	0	0	1	0
DS5	0	0	0	0	1

5.6 Implementation of a Markov Chains Model to obtain the probability distribution function of each GDS after a determinate number of earthquakes

The initial conditions as indicated in item 3.6, to determine the structural reliability are The number of damage states (n) is 5, the number of accelerations (na) is 20, the number of events (N) is 50, the lifetime of the buildings (t) is 50 years and the initial probability vector of the damage states is Vo = {1,0,0,0,0}.

The rate of exceedance of the accelerations (va) of the Peruvian coast is used, which is shown in Fig. 12, because it is in this zone where the greatest occurrence of earthquakes and of great intensities occurs, therefore the seismic danger is greater in this zone.

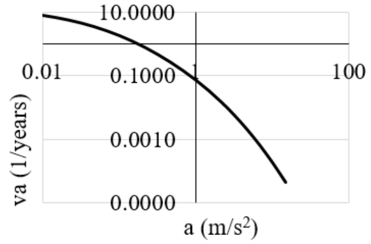


Fig. 12 – Acceleration exceedance rate (va) for the Peruvian coast

The probability vector is determined at the end of the Nth event, for N = 0, 1, 25 and 50, as shown in Fig. 13.

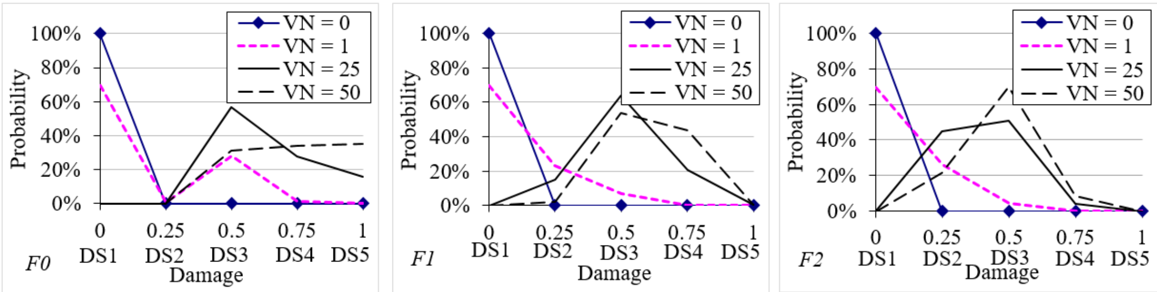


Fig. 13 – Damage density after N events for buildings

For N = 25, the damage stage of Life Safety (DS3) is the most probable for the buildings F0 (57%), F1 (64%). and F2 (51%).

For N = 50 events, the F0 building presents an equal probability of 30% for the damage state of Life Safety (DS3), Near Collapse (DS4) and Collapse (5), F1 building presents an equal probability of 45% for the damage states of Life Safety and Near Collapse and F2 building presents that the most probable damage state is Life Safety with 70% of probability.

5.7 Estimation of the structural reliability taking into account the damage accumulation process during probable future earthquakes assuming a Poisson process

Through a Poisson process, the final probability of damage states is determined for an indeterminate number of events (N), as indicated in item 3.7. Fig. 14 shows the graphics of the variation of the final probability of damage states along the lifetime of the buildings.

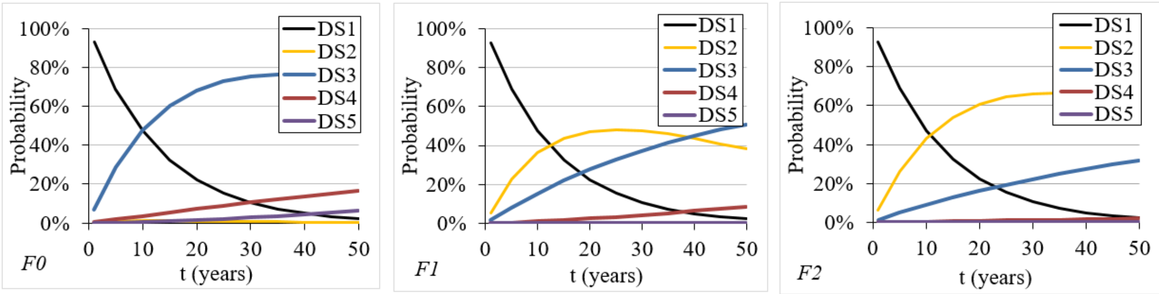


Fig. 14 – Final probability of damage states of the buildings

Table 4, Table 5 and Table 6 show the probability of building damage over 10 years, 25 years and 50 years (lifetime).

Table 4 – Damage probability of the buildings for 10 years

Building	Operational	Functional	Life Safety	Near Collapse	Collapse
	DS1 = 0%	DS2 = 25%	DS3 = 50%	DS4 = 75%	DS5 = 100%
F0	0.47	0.01	**0.48**	0.04	0.00
F1	**0.47**	0.36	0.15	0.01	0.00
F2	**0.47**	0.43	0.09	0.00	0.00

Table 5 – Damage probability of the buildings for 25 years

Building	Operational	Functional	Life Safety	Near Collapse	Collapse
	DS1 = 0%	DS2 = 25%	DS3 = 50%	DS4 = 75%	DS5 = 100%
F0	0.15	0.01	**0.73**	0.09	0.02
F1	0.15	**0.48**	0.33	0.03	0.00
F2	0.15	**0.64**	0.19	0.01	0.00

Table 6 – Damage probability of the buildings for 50 years

Building	Operational	Functional	Life Safety	Near Collapse	Collapse
	DS1 = 0%	DS2 = 25%	DS3 = 50%	DS4 = 75%	DS5 = 100%
F0	0.02	0.00	**0.74**	0.17	0.06
F1	0.02	0.38	**0.51**	0.09	0.00
F2	0.02	**0.64**	0.32	0.02	0.00

It is obtained the structural reliability as indicated in Eq. (19), as shown in Fig. 15.

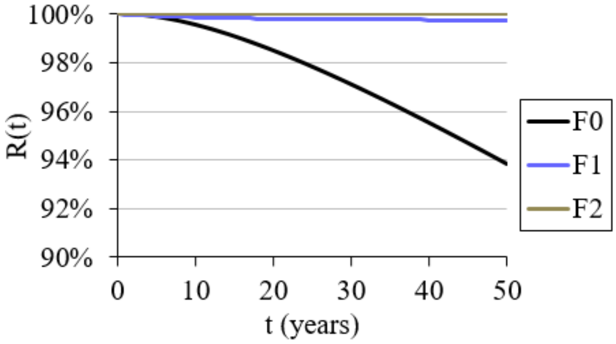


Fig. 15 – Reliability of the buildings

As shown in Fig. 15, the reliability of the three buildings are very high. Since the probability of collapse is very low in the original structure, the effect of retrofitting could be neglected. But, if is consider that failure of the structure correspond to Life Safety damage state exceedance, then the first retrofitting stage could increase the reliability in 40% and second retrofitting stage increase reliability in 66%, as shown in Fig. 16.

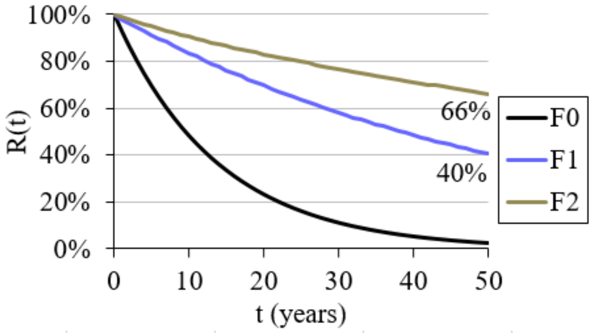


Fig. 16 – Reliability for the damage state (DS3) – Life Safety of the buildings

5.8 Estimation of the structural reliability considering initial damage

5.8.1 Reliability of the reinforced structure in phase 1 (F1)

From the reliability of the F0 building (Fig. 16) it is determined that the structure must be reinforced after 10 years, because the probability for the damage state (DS3) – Life Safety is more than 50%. For the building F1 is considered an initial probability vector of the damage state Vo = {0.93,0,0.07,0,0}. It is obtained that the final probability of damage states does not differ significantly from that obtained previously (Fig. 14).

5.8.2 Reliability of the reinforced structure in phase 2 (F2)

It is considered that the building will be reinforced after 5 years of the first intervention. The final probability of damage states is obtained for the F2 building, considering an initial probability vector of the damage state V_o = {0.86,0.06,0.08,0.0}, as shown in Fig. 17.

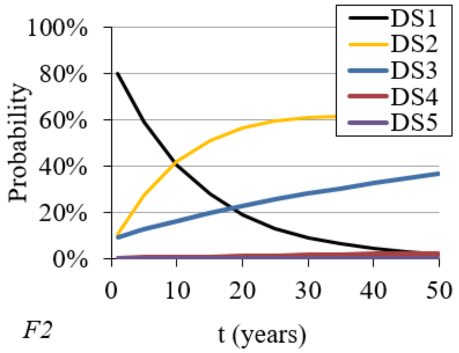


Fig. 17 – Final probability of damage states of the F2 building

5. Conclusions

This paper proposes and develops a simplified method, which aims to determine the structural reliability, considering the damage due to the occurrence of probable earthquakes during the service life of a building. As a case of study, a school building is analyzed with an incremental retrofitting technique, which consists of two phases (with a partial performance objective and with a total performance objective).

From the results obtained, it is concluded that the building requires a first intervention after 10 years, because, although the problems of the joints have been corrected, this does not guarantee a good performance in the long term, due to the fact that the building has deficiencies in resistance and rigidity in the longitudinal direction. Therefore, after 10 years the F0 building presents a 48% probability of damage state of Life Safety (Fig. 14.F0 and Table 4), which increases over the years and this damage state represents an global damage of 50% of the building.

Considering a first intervention, F1 building presents a probability of 47% of Functional damage state (Fig. 14.F1), predominating over the other damage states, until past 40 years, in which the Life Safety damage state starts to increase. The Functional damage state represents a global damage of 25% of the building. Considering a second intervention, F2 building presents a probability of 64% of Functional damage state (Fig. 14.F2), predominating over the other damage states along the building's lifetime. This state of damage represents an overall damage of 25% of the building.

The reliability obtained considering the final damage state of Collapse (Fig, 15), indicates that after 50 years (lifetime), the probability of collapse is null for the present building, and the incremental retrofitting building. This is consistent with reality, since buildings do not usually collapse after 50 years (lifetime), they present damage on the elements, which directly affects the functionality of the building. For this reason, reliability is determined by considering the final damage state of Life Safety (Fig. 16). It is obtained that after 50 years (lifetime), for the present building (F0) the probability of damage state of Life Safety is approximately 100%, affecting the total functionality of the building. Therefore, it is concluded that the building needs to be reinforced. With a first intervention, the building presents a decrease in the damage state of Life Safety to 60% and with the second intervention, there is a decrease to 34%. It is concluded that considering an incremental retrofitting in two phases, after 50 years (lifetime), the reliability is 66% for the Functional damage state.

The procedure developed in item 5.7 does not consider initial damage, it is assumed that the buildings are relatively new, which is not entirely true, since although the present building has been repaired, there are some elements with minor damage which have not been intervened. Comparing the results obtained considering initial damage (Fig. 17), for the first intervention, the building does not present a significant variation in the probabilities of damage states throughout the lifetime of the building. As well, for the second intervention, there is a minor variation in the probability of the Operational damage state, but this does not influence the rest of the damage states, being the Functional damage state predominant.

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7. Copyrights

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