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EFFECTS OF SUCCESSIVE EARTHQUAKES ON SEISMIC RESPONSE ON REINFORCED CONCRETE BUILDINGS IN EL SALVADOR

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Abstract

The effects of successive earthquakes in the response of RC moment-resisting frame systems for low ductility (intermediate moment frames) and high ductility (special moment frames) levels commonly designed in El Salvador is assessed in the present document. The outcome of the research may have great impact on the Response Modification Factor (R), Displacement Amplification Factor (Cd) and Allowable Drift (Δa), currently defined on the current national building code for seismic design in El Salvador, which still considers a single event as basis for the design of new structures. Three strong ground motion records from El Salvador are used, one near-fault source and two far-field source; with such input 15 sequences of one, two and three successive earthquakes are generated. Two sets of building are studied: a low ductility (1, 2, 3 and 4 stories) group and a high ductility (3, 6, 9 and 12 stories) group, 1353 analysis have been performed in total. The methodology comprised both nonlinear static analysis (static pushover) and incremental nonlinear dynamic analysis (dynamic pushover) for each model and earthquake sequence. Performance criteria at elements (local failure) and global failures are evaluated in all models. No damage evaluation in elements and torsion effects are considered. The results indicate that the Response Modification Factor "R" decreases as the number of records in the sequences of events increases, highlighting the importance of considering more than one event in its definition; additionally, for short periods (<0.43 sec.) the Response Modification Factor "R" is closely related to the elastic period of the buildings. The Displacement Amplification Factor "Cd" does not show significant variation by increasing the number of records in the sequence; however, it is observed this factor is linked to the elastic period of the building, which is opposed to the recommendations established on the current national building code of El Salvador. In addition, it has been found that low ductility buildings are being designed for greater forces than required, while high ductility buildings are being designed for lower forces than the ones recommended on the current national building code. Finally, results suggest the Displacement Amplification Factor found on the current Code is not conservative for buildings with periods longer than 1.0 sec whilst the Allowable Drift is considered conservative in all cases.

Keywords: successive earthquakes; response modification factor; incremental nonlinear dynamic analysis, allowable drift

1. Introduction

The seismic design of buildings is based on proportioning members of the seismic framing system for actions determined from a linear analysis using prescribed lateral forces. The concept of a Response Modification Factor has been proposed based on the premise that well-detailed seismic framing systems could sustain large inelastic deformations without collapse (ductile behavior) and develop lateral strengths in excess of their design strength (reserve strength) [1].

The Static Lateral Force Method analysis procedure remains the most widely used in the practice of seismic design. Although this procedure does not produce estimates of nonlinear response quantities, it is a valuable analysis and design tool for the seismic design [2]. The Static Lateral Force Method is based on representing the nonlinear response in seismic framing systems by using a Response Modification Factor (R). The conventional approach to reducing elastic spectral forces using a Response Modification Factor (R) to reach the level of strength design is widely used by seismic codes and design methods based in forces, including a final check of deformations by a Displacement Amplification Factor (Cd) without exceeding an Allowable Drift (Δa).

The current seismic design code of El Salvador, developed in 1996, recommends the Static Lateral Force Method and the use of those three parameters (R, Cd and Δa) for the seismic design; such parameters have been developed considering a single event, as seismic input, in the design of new structures. However, El Salvador is located in an active seismic region produced by the subduction of the Cocos plate under the Caribbean plate; as a consequence of this interaction, there is local seismicity, within the country, as well as active volcanoes. As a result, the capital city, San Salvador is affected by two main sources of seismicity. The first one, strong events, with magnitudes up to Mw 7.7, with foci between 30 km and 200 km along the Pacific coast and, the second one, low to medium size earthquakes, with maximum magnitudes around Mw 6.8, with depths less than 20 km produced by the system of local faults, which coincides with the volcanic chain [3].

This seismic activity generates events which are close in time and space, giving little chance to rehabilitate or strengthen damage structures. This fact should be considered in the evaluation of seismic risk for El Salvador, since buildings located at volcanic chain need to be designed to withstand two or three consecutive events, whether far-field source or near-fault source, or a combination of both. Therefore, the seismic performance of structures is strongly conditioned by the cumulative effects of two or more successive seismic events [4].

2. Definitions and procedure

2.1 Key components of Response Modification Factor

The Response Modification Factor is expressed by the product of four factors [1] [5], as shown in Eq. (1).

$$R=R_S R_\mu R_R R_\zeta \tag{1}$$

Where R_S is the overstrength factor, R_μ is the ductility factor, R_R is the redundancy factor, and R_ζ is the damping factor. The redundancy factor has had little research and development reaching excluded from the formulation of R; on the other hand, the damping factor applies to systems with damping devices, and their use must not be applicable in seismic codes involving methods by forces [1] [2] [5]. Other studies [6] consider that the damping is a property included in the ductility factor, and that the overstrength and redundancy factors should be considered as one alone. That is why the evaluation of redundancy and damping factors has been left out of the determination of R, as shown in Eq. (2).

$$R=R_S R_\mu \tag{2}$$

2.2 Evaluation of Response Modification Factor considering the ground motion

This section contains most of the concepts and criteria of the methodology proposed by Mwafy & Elnashai [6] [7], as shown in Fig. 1; which is the procedure most suitable to assess the effects of consecutive earthquakes on the factor R, in the opinion of the authors. The overstrength factor denoted as R_S in Eq. (2), is denoted here as Ω_d, reconsidering the Response Modification Factor on Eq. (2) as shown in Eq. (3).

$$R=\Omega_d R_\mu \tag{3}$$

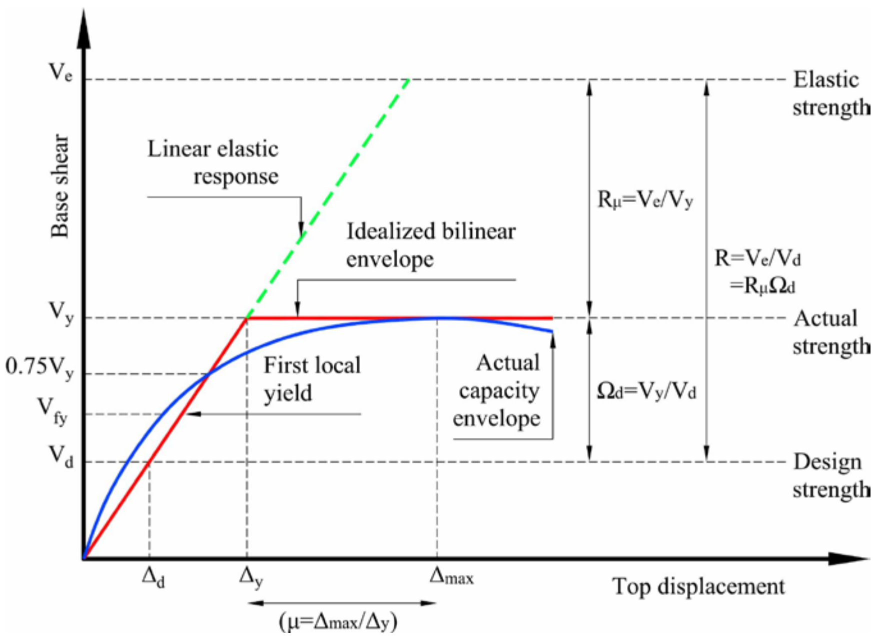

Fig. 1 – Relationships between response modification factor (R), overstrength factor (Ωd), ductility factor (Rμ) and displacement ductility factor (μ), after [6].

In seismic codes, the Response Modification Factor reduces the elastic base shear strength (V_e) at the level of shear design strength (V_d). The elastic and design strengths are obtained from the elastic acceleration spectrum site $(S_a)^{el}$ and from the design spectrum used $(S_a)^{in}$, respectively, as shown in Eq. (4), Where $(S_a)^{el}$ and $(S_a)^{in}$ are the elastic and design spectral acceleration, respectively, for the predominant period of the structure.

$$R_{code}=(S_a)^{el}/(S_a)^{in} \tag{4}$$

The collapse is attained using an earthquake with a bigger spectrum than the elastic spectrum for the period under consideration. Therefore, the Eq. (5) can be used to evaluate the Response Modification Factor for a particular structure subjected to a specific strong ground motion record.

$$R_{c,dy}=(S_a)^{el}{}_c/(S_a)^{in} \tag{5}$$

The subscript "c" and "dy" refers to the collapse and yield level (the level of yield that has been assumed in the design), respectively. This expression relates the spectral ground acceleration that produces the collapse with the spectral acceleration for which it was designed. Moreover, it is know that the level of effective yield of a structure is higher than the yield level assumed in the design (overstrength), therefore to obtain a more accurate value of the Response Modification Factor have to be considered the effective yield. Therefore, a modification to the Eq. (5) in order to relate the spectral ground acceleration, produced by the collapse of the structure, with the spectral acceleration that produces the level of effective yield is introduced; this relationship is shown in Eq. (6).

$$R_{c,ay}=(S_a)^{el}{}_c/(S_a)^{el}{}_y \tag{6}$$

Where the subscript “ay” refers to the effective yield. Assuming the response spectrum design, effective yield and collapse have constant dynamic amplification (ratio of acceleration response against peak acceleration), at least for the range of periods considered; Eq. (5) and (6) can be rewritten in terms of their absolute acceleration as follows:

$$R_{c,dy}=a_{g(collapse)}/(a_{g(design)}/R_{code})=a_{g(collapse)}/a_{g(yield\text{-}design)} \tag{7}$$

$$R_{c,ay}=a_{g(collapse)}/a_{g(yield\text{-}effective)} \tag{8}$$

Where $a_{g(collapse)}$ is the peak acceleration that produced collapse, $a_{g(design)}$ is the design acceleration, $a_{g(yield\text{-}design)}$ is the PGA design divided by the force reduction factor used in the design (R_{code}), and $a_{g(yield\text{-}effective)}$ is the effective yield during an earthquake. The difference between $a_{g(yield\text{-}design)}$ and $a_{g(yield\text{-}effective)}$ is that the first one concerns the PGA to produce the level of yield considered in the design, while the second one, is the peak acceleration when the first yield actually occurs. In short, it is a relationship between the seismic intensity believed to produce yield and the seismic intensity which actually produces it.

Eq. (7) and (8) relate the intensity of the collapse load to the elastic seismic forces, design and effective yield, respectively. Eq. (7) adopts the consideration that yield occurs in the design acceleration (PGA design) divided by R_{code}. However, the definition of $R_{c,dy}$ has the disadvantage of not considering the difference between the design spectrum and the spectrum ground motion which actually produces yield, that is why it has not been used in the present research.

Then, there is the need to modify the Eq. (8) by adding the overstrength factor (Ω_d=actual to design strength) to $R_{c,ay}$, which will be evaluated through Nonlinear Static Analysis (static pushover) and Incremental Nonlinear Dynamic Analysis (Dynamic pushover), the proposed amendment is shown in Eq. (9).

$$R'_{c,ay}=R_{c,ay}\Omega_d=(a_{g(collapse)}/a_{g(yield\text{-}effective)})\Omega_d \tag{9}$$

This last adjustment preserves features of the original definition of $R_{c,ay}$ in terms of the dependence of the ground motion with acceleration that produces collapse $a_{g(collapse)}$ and the acceleration that actually produces yield $a_{g(yield\text{-}effective)}$, this feature provides certain advantages $R'_{c,ay}$ over $R_{c,dy}$, which ignores this dependence in the denominator. The main deficiency of the Eq. (9) is the onerous computational procedure and is based on the assumption of a constant dynamic amplification. However, it is an effective way to evaluate the Response Modification Factor for a particular structure submitted to a specific earthquake [6].

2.3 Evaluation of Displacement Amplification Factor

To estimate the (inelastic) maximum deformation that can develop into severe earthquakes, seismic design codes specify a Displacement Amplification Factor (Cd) to amplify the calculated elastic deformations [8]. Based on the definitions of Response Modification Factor and Fig. 1, the Displacement Amplification Factor can be derived as follows [8]:

$$Cd=\Delta_{max}/\Delta_d= (\Delta_{max}/\Delta_y)(\Delta_y/\Delta_d)=\mu_s(\Delta_y/\Delta_d) \tag{10}$$

Applying properties of proportionality on Fig. 1, have to Δ_y/Δ_d is equal to V_y/V_d, therefore:

$$Cd=\mu_s(V_y/V_d)=\mu_s\Omega_d \tag{11}$$

2.4 Evaluation of Allowable Drift

In this research, the maximum drifts obtained from Incremental Nonlinear Dynamic Analysis before reaching the first collapse criterion are evaluated to be compared with the Allowable Drifts for low ductility (Δa=0.015) and high ductility (Δa=0.020) buildings recommended by the current national building code of El Salvador.

3. Components of the analytical study

3.1 Buildings

The purpose of the investigation is evaluate the effects of successive earthquakes in the response of RC moment-resisting frame systems for low ductility (intermediate moment frames) and high ductility (special moment frames) levels commonly designed in El Salvador. Two groups of building are studied: low ductility (1, 2, 3 and 4 stories) and high ductility (3, 6, 9 and 12 stories). According to the current seismic code of El Salvador, the parameters used during the seismic design of the buildings are R=5 and 12; Cd=5 and 8; Δa=0.020 and 0.015, each parameter value for low ductility and high ductility, respectively. The geometric characteristics of the buildings are shown in Fig. 2.

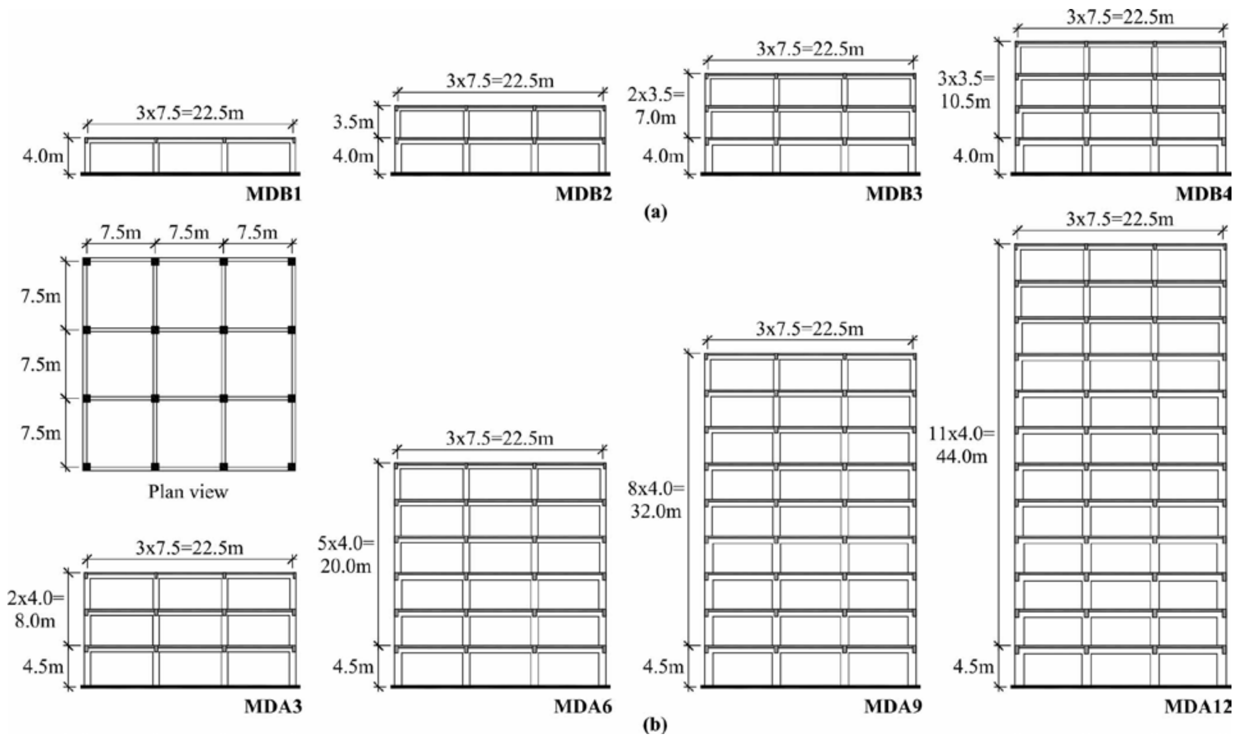

Fig. 2 – Sectional elevation of the buildings: (a) low ductility group; (b) high ductility group.

3.2 Input ground motion and earthquake sequences

Three strong ground motion records from El Salvador are used (Fig. 3), one near-fault source (October 10, 1986; Mw=5.7) and two far-field source (January 13, 2001; Mw=7.6) to generate 15 sequences of one, two and three successive earthquakes (Fig. 4). To consider the damping effects at free vibration phase of the models at the end of each consecutive record, an interval of 100 seconds with acceleration 0 is used.

3.3 Performance parameters

Five and four performance criteria associated with local and global faults, respectively, are defined. Local failures are related to the individual behavior of each of the structural elements that are part of the structural system. Furthermore, global faults are related to the behavior of the entire structural system as a whole.

Performance criteria for local faults used [9]: i) yield reinforcement steel; ii) rupture of reinforcing steel; iii) ultimate concrete strain; iv) ultimate curvature; v) shear strength. Performance criteria for global faults used [6]: i) upper limit of the interstory drift (ID) ratio equal to 3%; ii) a drop in the overall lateral resistance by more than 10%; iii) a upper limit of the stability index (ID x story gravity load/story shear); iv) formation of a column hinge mechanism.

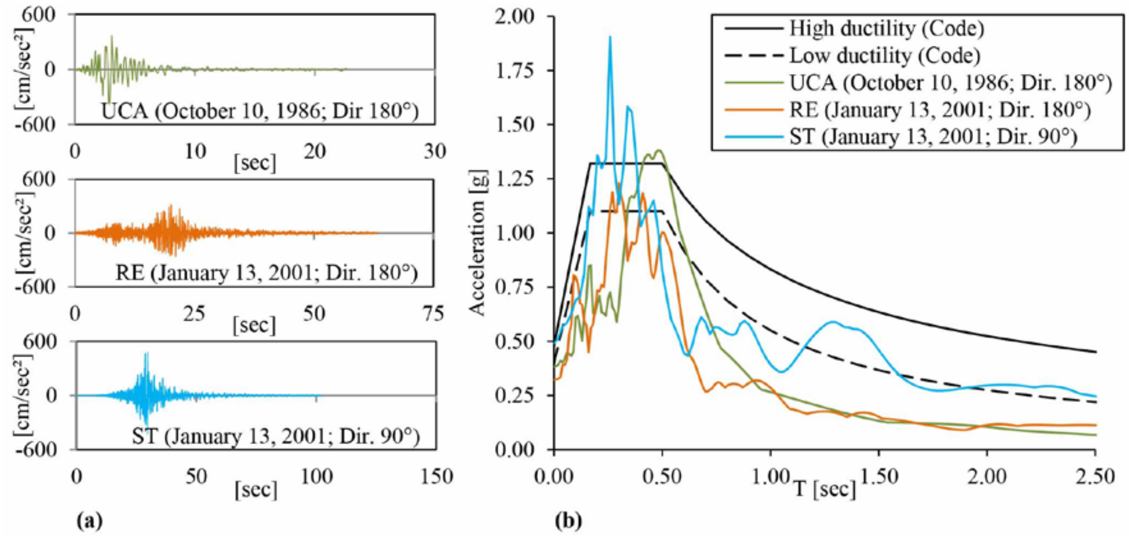

Fig. 3 – Input ground motion: (a) records; (b) elastic response spectra.

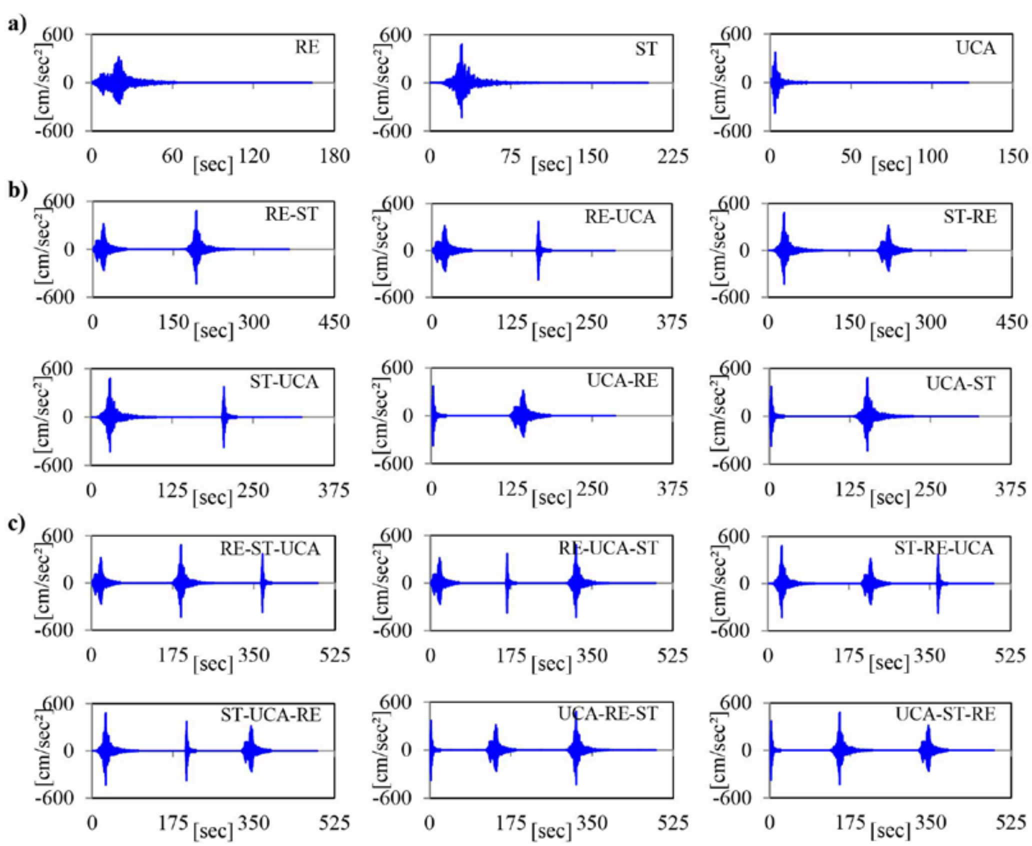

Fig. 4 – Successive earthquake sequences: (a) one record; (b) two records; (c) three records.

4. Analytical results: Static and dynamic curves

For each model, the base shear (V) vs. displacement (Δ) relationships are plotted for Nonlinear Static Analysis (static pushover) and Incremental Nonlinear Dynamic Analysis (dynamic pushover), as shown in Fig. 5. The strength at the design level is compared with the level of maximum shear obtained from analysis.

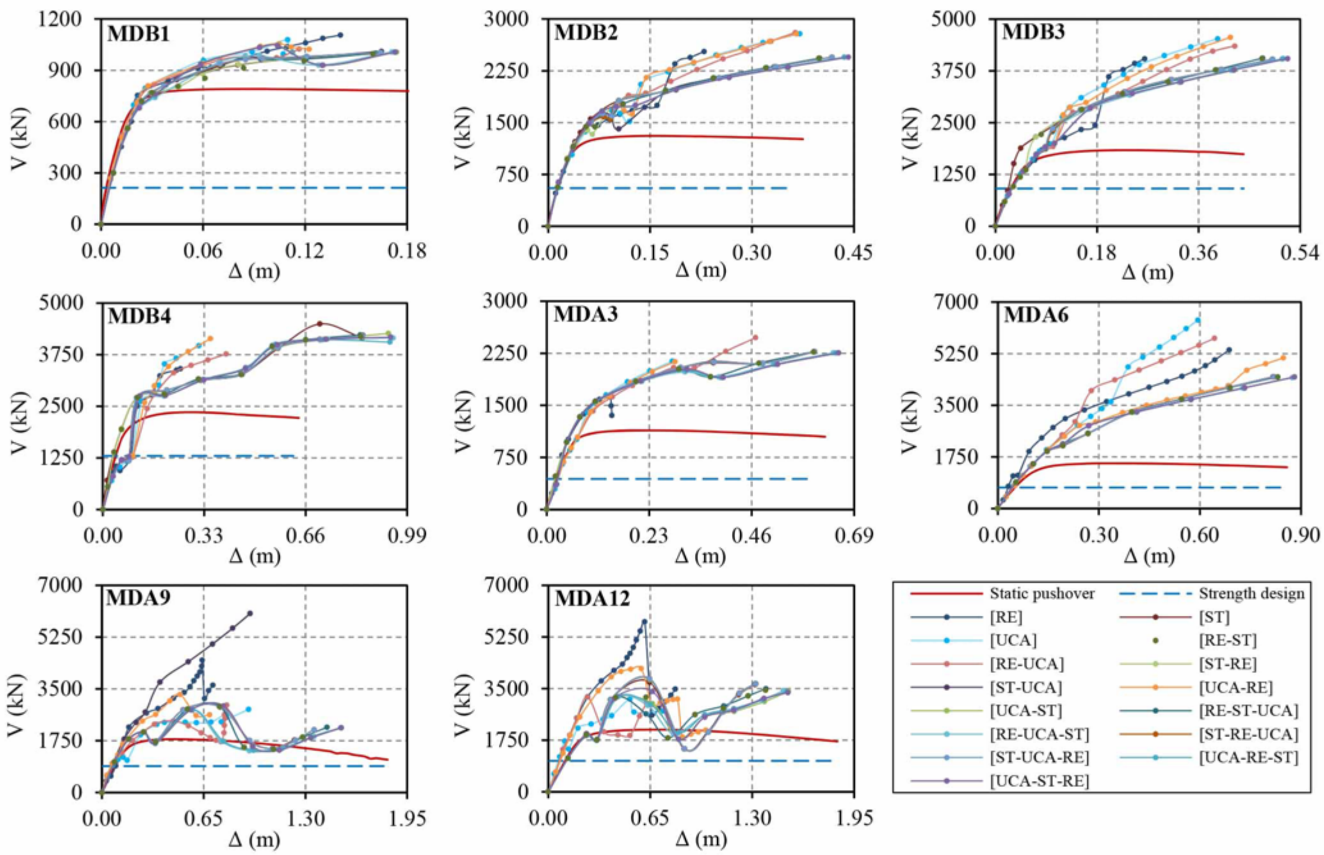

Fig. 5 – Base shear (V) vs. displacement (Δ) relationships for Nonlinear Static Analysis (static pushover) and Incremental Nonlinear Dynamic Analysis (dynamic pushover).

Analyzing the Strength Design firstly is observed that the Static Pushover curves of low ductility models have more capacity than those of the high ductility models. Second, the Dynamic Pushover curves show higher strength than the pushover plots. These arguments demonstrate the overstrength exhibit by the models which are beyond its design strength. Fast decreasing resistance observed in MDA9 and MDA12 possibly are due to their height, reaching conditions of geometric instability (inter-story drifts greater than 3% and stability index ID greater than 0.3) sooner than the other models analyzed which resulting in strength drops around 10%.

For Dynamic Pushover, the first global failure of the MDB1 model ($T_{elastic}$=0.204 sec) is produced by formation of a plastic hinge mechanism in columns. For MDB2 ($T_{elastic}$=0.309 sec), MDB3 ($T_{elastic}$=0.364 sec) and MDB4 ($T_{elastic}$=0.426 sec) models as well as for MDA3 ($T_{elastic}$=0.525 sec) and MDA6 ($T_{elastic}$=0.833 sec) models, the first global failure occurs because the maximum drift set is exceeded. The first global failure of the MDA9 model ($T_{elastic}$=1.164 sec) is a product of numerous factors which are related to the inter-story drift and stability index. In the case of the MDA12 model ($T_{elastic}$=1.415 sec), the first global failure is related to strength drops higher than 10% and index stability values exceeding the values defined as limit.

It is observed that models with very short elastic periods (≤0.20 sec), the collapse is produced by a weak story due to generation of a plastic hinge mechanism in columns. In the range of elastic intermediate periods (>0.20 sec and <0.90 sec), the collapse is produced by large lateral displacements which yields global instability in the models. For elastic high periods (≥0.90 sec), excessive lateral displacements produce significant increments beyond the stability index considered as secure, resulting in strength drops by more than 10%.

5. Evaluation of seismic response factors

5.1 Response modification factors

The Response Modification Factor ($R'_{c,ay}$) is obtained for each model in the Incremental Nonlinear Dynamic Analysis, the average of each sequence for 1, 2 and 3 successive earthquakes (1 se, 2 se & 3 se) is shown in Table 1, which also shows the Factor Modification Factor ($R_{code)}$ recommended by the current Code. For purposes of comparison, is calculated the Response Modification Factor (R_{sp}) based on the capacity curve (static pushover). These values are plots as shown in Fig. 6 depending on the number of stories of each model.

Table 1 – Response modification factors.

Model	**R_{code}**	**R_{sp}**	**$R'_{c,ay}$**		
			1 se	**2 se**	**3 se**
MDB1	5	6.4	18.1	15.7	12.9
MDB2		6.1	17.8	15.9	12.6
MDB3		5.0	9.9	8.6	8.4
MDB4		4.3	8.9	7.6	7.2
MDA3	12	10.7	9.3	8.1	7.5
MDA6		10.0	8.6	7.2	6.8
MDA9		9.9	10.1	8.3	7.5
MDA12		9.8	9.6	7.2	6.9

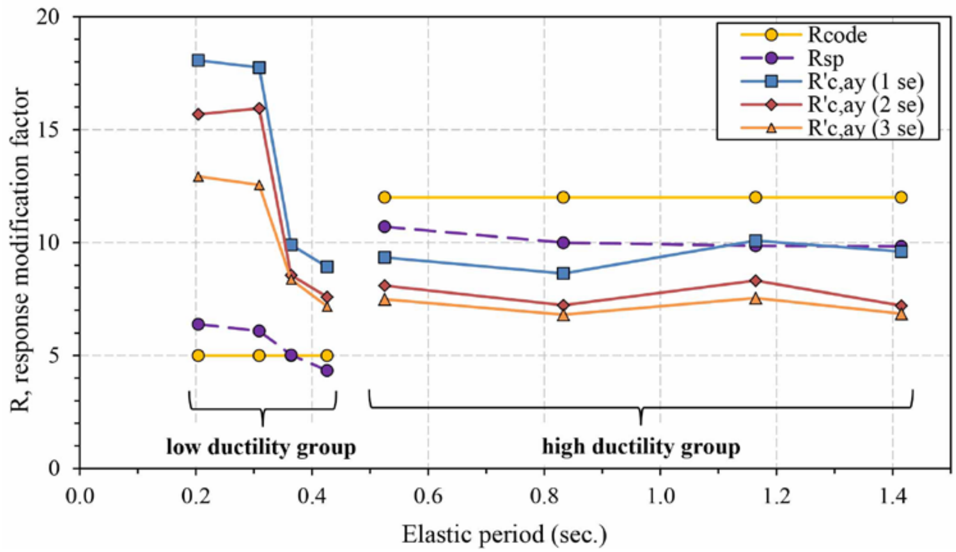

Fig. 6 – Response modification factors: R_{code}, R_{sp} & $R'_{c,ay}$ (1, 2 & 3 successive earthquakes).

The values of R_{sp} are 6.4 to 4.3 for low ductility models (R_{code}=5); and 10.7 to 9.8 for high ductility models (R_{code}=12). The results confirms largely those recommended values by the current Code, which is consistent with the age it was published (1996), in which the procedures used to find out the Response Modification Factor, likely corresponded to a Static Pushover analysis without considering the dynamic response of seismic event records.

The values of $R'_{c,ay}$ are 18.1 to 8.9 for one successive earthquake in low ductility models. The big difference in results for one successive earthquake, compared with those obtained from Static Pushover, show there is a great variability in the dynamic response of the models considering ground motion records (for the range of short elastic periods of 0.20 sec. to 0.43 sec). It is noted that as elastic model period increases, the $R'_{c,ay}$ decreases, the above shows a strong dependence between both parameters for short periods less than 0.43 sec. In the case of models for high ductility, the values obtained for $R'_{c,ay}$ are 10.1 to 8.6 for one successive earthquake. Although there are differences with the results from Static Pushover, these are not as marked as in low ductility models. The above is because the range of elastic periods (0.53 sec. to 1.42 sec.) is greater, which show a tendency to uniformity on results on $R'_{c,ay}$ by increase the elastic period of the models.

The reductions on values obtained of $R'_{c,ay}$ for low ductility models are 10.2% to 14.9% for two successive earthquakes, and 15.5% to 29.3% for three successive earthquakes. While for high ductility models, the reductions are 13.4% to 24.9% for two successive earthquakes, and 19.8% to 28.6% for three successive earthquakes. This allows the observing of a significant reduction in the values obtained for two successive earthquakes; however, considering three successive earthquakes, not significant reduction is obtained. The above highlights the importance of considering more than one record in the definition of Response Modification Factor, but no more than two records; it looks better in Fig. 6. Finally, the results of $R'_{c,ay}$ show a strong dependence on the elastic period of the models for short periods (<0.43 sec.), as other authors have mentioned [10], contrary as specified by the current Code in which constant values are recommended.

5.2 Displacement amplification factors

As in previous section, the results obtained for the Displacement Amplification Factors are shown in Table 2 and Fig. 7. Where $Cd_{analysis}$ is obtained before reaching the first collapse criterion evaluated for each successive earthquake in the Incremental Nonlinear Dynamic Analysis; Cd_{sp} is obtained from Static Pushover analysis; and Cd_{code} are the values recommended by the current Code.

Table 2 – Displacement amplification factors.

Model	Cd_{code}	Cd_{sp}	$Cd_{analysis}$		
			1 se	2 se	3 se
MDB1	5	15.6	24.2	18.2	15.2
MDB2		8.6	15.7	13.8	15.8
MDB3		6.4	11.5	13.3	12.1
MDB4		5.1	5.6	6.0	6.1
MDA3	12	8.2	11.8	13.9	13.8
MDA6		6.0	9.3	9.8	11.7
MDA9		4.9	10.1	9.5	8.6
MDA12		5.4	6.9	5.0	6.2

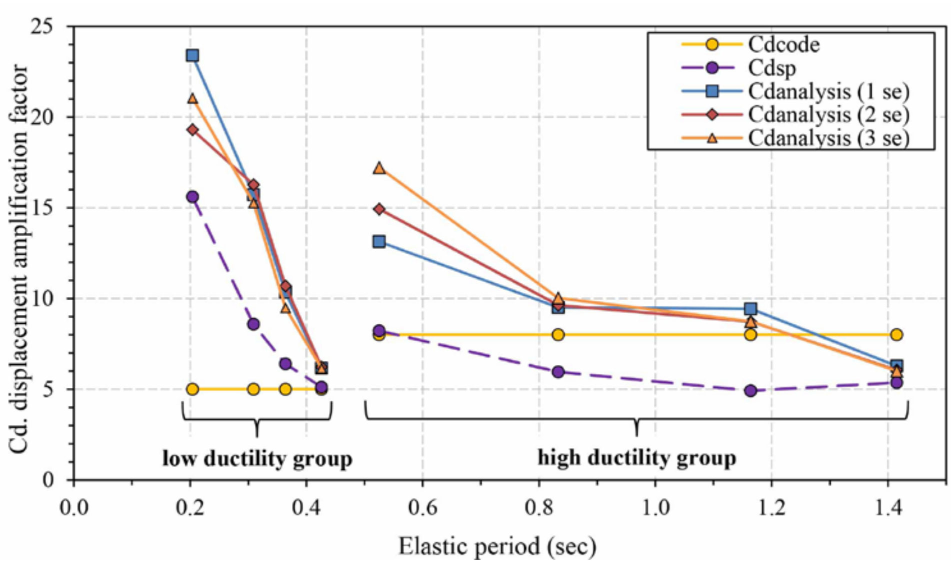

Fig. 7 – Displacement amplification factors: Cd_{code}, Cd_{sp} & $Cd_{analysis}$ (1, 2 & 3 successive earthquakes).

The values obtained for Cd_{sp} are 15.6 to 5.1 for low ductility models (Cd_{code}=5); and 8.2 to 5.4 for high ductility models (Cd_{code}=8). These results are disagreeing with the current Code, since for low and high ductility are above and below the recommended values, respectively. However, the values obtained show a tendency to decrease as elastic period increases, marking a strong dependence between these parameters.

The values for $Cd_{analysis}$ are 24.2 to 5.6 for low ductility models; and 11.8 to 6.9 for high ductility models for one successive earthquake. In assessing the seismic response of two and three successive earthquakes, the values obtained show that this factor is indifferent to the cumulative effect of successive earthquakes, since no major variation is obtained by increasing the number of records in seismic sequences.

5.3 Drifts

The maximum inter-story drift envelope obtained from the Dynamic Pushover analysis, before reaching the first criteria of global collapse, for each model, is shown in Fig. 8; in addition, the Allowable Drift (Δa) recommended by the current Code (0.020 and 0.015 for low and high ductility models, respectively), is included.

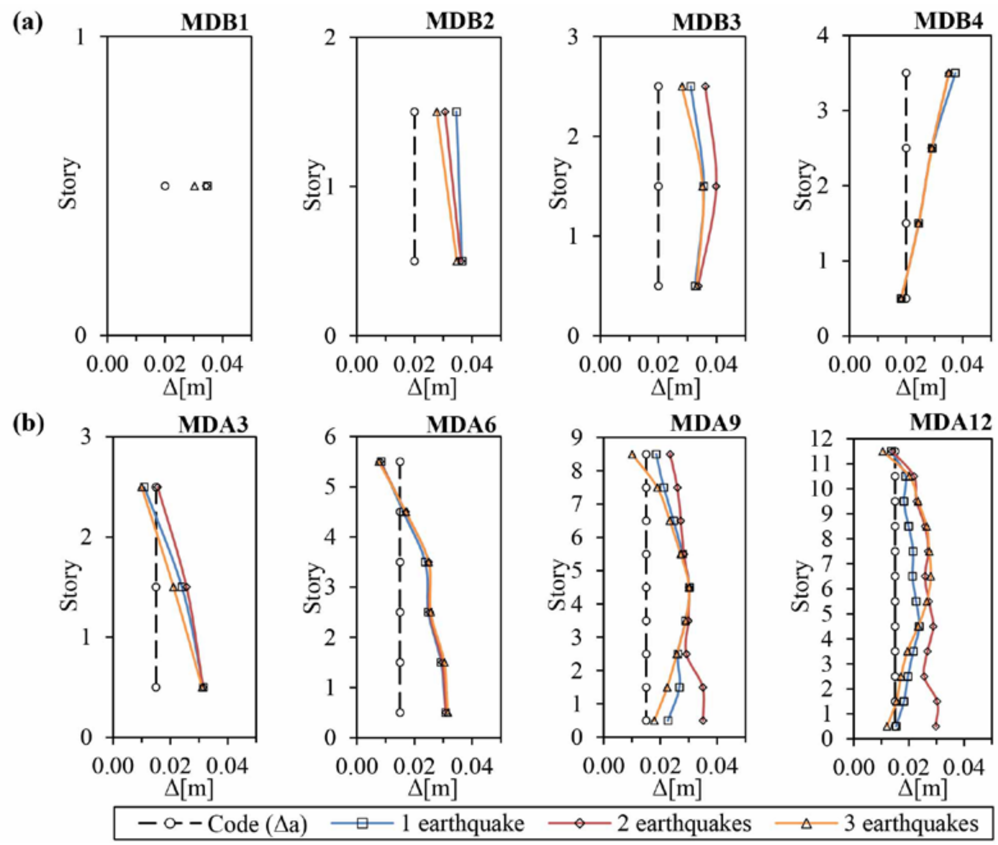

Fig. 8 – Maximum inter-story drifts envelope: (a) low ductility; (b) high ductility.

Low ductility models present average maximum drifts around 0.031, 0.032 and 0.030 for sequences of 1, 2 and 3 successive earthquakes, respectively, which are compared with the Allowable Drift Δa=0.020. While for high ductility models, the average maximum drifts around 0.022, 0.026 and 0.022 for sequences of 1, 2 and 3 successive earthquakes, respectively; which are compared with the Allowable Drift Δa=0.015. The results are within the range of values obtained by other authors [6].

Firstly, the result of the average maximum drifts are very close to each other for low and high ductility models, there is no significant difference by increasing the number of successive earthquakes. Second, a clear trend is that the point where the first global collapse is reached, the maximum drifts can be limited to 0.030 and 0.022 for low and high ductility models, respectively. Finally, whether the values defined on the current Code

(0.020 and 0.015, for low and high ductility models, respectively) are accepted as the allowable drifts which control nonstructural damage, and those are compared to the average maximum values obtained from analysis (0.030 and 0.022) which are drifts just before reaching the first global collapse criterion; then the latter represent an increase of 50.0% and 46.7% over the allowable values, found on the Code, for low and high ductility models, respectively.

6. Conclusions

When the effect of successive earthquakes are evaluated in the response of RC moment-resisting frames designed in El Salvador, the results shows that the Response Modification Factor decreases as the number of records increase. The results shows that new buildings for low ductility are being designed, according to current Code, for conservative seismic loads; and buildings with very short elastic periods (<0.30 sec) could be designed even for seismic forces lower than currently used, even considering the effect of successive earthquakes. In contrast, the results shows that new buildings for high ductility are being designed for seismic forces smaller than those occurring during seismic events, a situation that worsens when considering the effect of successive earthquake by the significant reduction in the value of R, which it is obtained by considering two or more records in seismic sequences. The values of Displacement Amplification Factor recommended by the current Code are conservative for buildings with periods lower than 1.2 sec, but not for buildings with periods longer than 1.2 sec, which tend to be lower respect to the current Code. The recommended values of Allowable Drift in the current Code are considered conservative as they are about half of the maximum values obtained. However, it should be special attention to the fact that the allowable values are related to damage control from an economic point of view in the current Code, while the values obtained are associated with the first criterion of collapse achieved.

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