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ASSESSMENT OF DAMAGE INDEXES FOR PBSD OF LOW-RISE RC WALLS

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Abstract

Damage progression indexes are widely used to evaluate the performance of structural elements in buildings and bridges subjected to seismic actions. Evidence from earthquakes suggests that shear failure or combined shear-flexure behavior is responsible for a large proportion of failures. Previous studies have reported that most of current damage indexes are unsuited for squat, thin and lightly-reinforced concrete walls, which are characterized by a failure mode dominated by shear instead of flexural deformations. Experimental studies have been conducted to evaluate the seismic performance of these particular squat walls using damage indexes. These new damages indexes are based on the relation between the damaged area and the area of the façade of the RC wall, on the fractal dimension of the cracking recorded after an earthquake, on the stiffness degradation of walls, and one approach based on the widely used Park & Ang index but using a novel formulation for parameters included in this index. These damage indexes include limiting values and expected damage associated to performance levels for code-based seismic design and rehabilitation of low-rise RC walls. These damage indexes were calibrated using the experimental results obtained from quasi-static and shake-table tests performed on 39 thin RC walls constructed with typical characteristics of low-rise housing. Variables were the wall geometry (solid walls with different aspect ratios, and walls with door and window openings), type of concrete (normal-weight, light-weight and self-consolidating), web shear steel ratio (0.125% and 0.25%), type of web shear reinforcement (deformed bars and welded-wire mesh), and testing method. This paper is aimed at summarizing, comparing and discussing parameters, procedures, advantages and drawbacks of these damage indexes, including the limiting values for performance-based seismic design of low-rise housing.

Keywords: damage index, squat concrete wall, crack pattern, fractal dimension, stiffness degradation.

1. Introduction

Estimate of seismic damage on structures and the related seismic requirements are based mainly on the qualitative judgment of engineering. Most of current earthquake-resistant building codes exclude explicit recommendations of tolerable damage and thus they do not evaluate the performance of the building after the onset of damage. Damage index (*DI*) is a concept introduced for assessing damage in a quantitative manner. It consists on mathematical functions based on several structural parameters that quantify the structural damage in a scale varying between 0 to 1; zero for the no damage condition and one for the structural failure state [1].

Several damage indexes have been proposed in the literature on seismic damage assessment across a wide range of structures and loading types. There are various methods for damage detection and for computing of those indexes. Low-rise concrete housing in Latin America embraces particular characteristics such as thin concrete walls, low concrete strength, low axial loads, low steel reinforcement ratios, and web shear reinforcement made of deformed bars and welded-wire mesh. Current guidelines for estimating seismic damage are more directed to medium- or high-rise buildings. In addition, most of available indexes have been proposed to quantify the damage in structural elements that generally fail by flexion. All these conditions establish a limitation for using these indexes in low-rise reinforced concrete (RC) walls because they are generally dominated by shear deformations.

A large inventory of recent low- and medium-rise (up to four-story height) residential buildings in some countries in Latin America consists of RC walls with thickness smaller than 150 mm and only one center layer of web reinforcement. Previous experimental programs have provided information about the seismic behavior of low aspect ratio RC thin walls [2-6], and there are some studies aimed at evaluating its seismic performance in terms of damage. For instance, Carrillo and Alcocer [7] utilized a damage index based on the relation between the damaged area (area of cracks) and the area of the façade of the RC wall. Carrillo *et al*. [8] proposed a damage index to estimate the damage level and the residual performance based on the fractal dimension of the cracking recorded after an earthquake. Carrillo [9] proposed a damage index based on the stiffness degradation of walls. Such index depends on the story-drift ratio and the number of cycles experienced by the wall during a particular seismic event. Carrillo *et al*. [12] proposed a novel formulation for parameters included in the Park & Ang damage index [10, 11]. These four damage indexes were calibrated using the experimental results obtained from quasi-static monotonic and reversed-cyclic test performed on 39 thin RC walls constructed with typical characteristics of low-rise housing. This paper is aimed at summarizing, comparing and discussing parameters, procedures, advantages and drawbacks of these four damage index, including the limiting values for limit states and performance levels of low-rise housing.

2. Experimental program

Variables studied in the experimental program are summarized in Table 1. Variables were the wall geometry (solid walls with different aspect ratios, and walls with door and window openings), type of concrete, web shear steel ratio, type of web shear reinforcement, and testing method. Details of the experimental program can be found elsewhere [2-6].

3. Damage index based on residual cracking

3.1 Damage index

Carrillo and Alcocer [7] developed a damage index based on the maximum width of residual cracks for assessing the damage stage of concrete walls. Such index depends mainly on the area of all residual cracks recorded on the façade at the end of an earthquake record. The damage index DI_{cracks} proposed by Carrillo and Alcocer [7] is calculated using Eq. (1).

$$DI_{cracks} = \frac{\sum(l_{crack} \times w_{crack})}{A_{facade}} \times 100 \quad (1)$$

where l_{crack} and w_{crack} are the length and maximum width of a residual crack measured at the end of an earthquake record, and A_{facade} is the wall surface area on one side. The maximum value of w_{crack} was also calculated and was labeled as w_{res}. For each wall tested, the relationship between w_{res}, or I_{cracks}, and maximum story drift measured during testing stages was plotted and evaluated (Fig. 1). It was noted that the rate of increase of residual cracking was not significantly influenced by the type and amount of web shear reinforcement.

Table 1 – Variables of the experimental program

Variable	Description
Height-to-length ratio (h_w/l_w)	$h_w/l_w \approx$ 0.5, 1.0, 2.0 and also, wall with openings (door and window). Full-scale wall thickness (t_w) and clear height (h_w) were 100 mm (4 in.) and 2.4 m (94.5 in.), respectively. Then, to achieve the height-to-length ratio, wall length was varied.
Concrete type	Normal-weight (N), lightweight (L) and self-consolidating (S). Nominal concrete compressive strength, f_c', was 15 MPa (2175 psi).
Web steel ratio (vertical, ρ_v , and horizontal, ρ_h)	100% of ρ_{min} (0.25%), 50% of ρ_{min} (0.125%), 0% of ρ_{min} = without reinforcement (for reference). Minimum web steel ratio (ρ_{min}), is that prescribed by ACI-318 (2011). Wall reinforcement was placed in a single layer at wall mid-thickness.
Type of web reinforcement	Deformed bars (D) and welded-wire mesh made of small-gage wires (W). Nominal yield strength of bars and wire reinforcement, f_y, was 412 MPa (60 ksi) (for mild-steel) and 491 MPa (71 ksi) (for cold-drawn wires).
Boundary elements	Thickness of boundary elements was equal to thickness of wall web (prismatic cross section). Longitudinal boundary reinforcement was designed and detailed to prevent flexural and anchorage failures prior to achieving the typical shear failure observed in RC walls for low-rise housing.
Axial compressive stress, σ_v	σ_v = 0.25 MPa (36.3 psi) was applied on top of the walls and kept constant during testing. This value corresponded to an average axial stress at service loads of first story walls of a two-story prototype house.
Type of testing	Quasi-static (monotonic and reversed-cyclic) and dynamic (shake table).

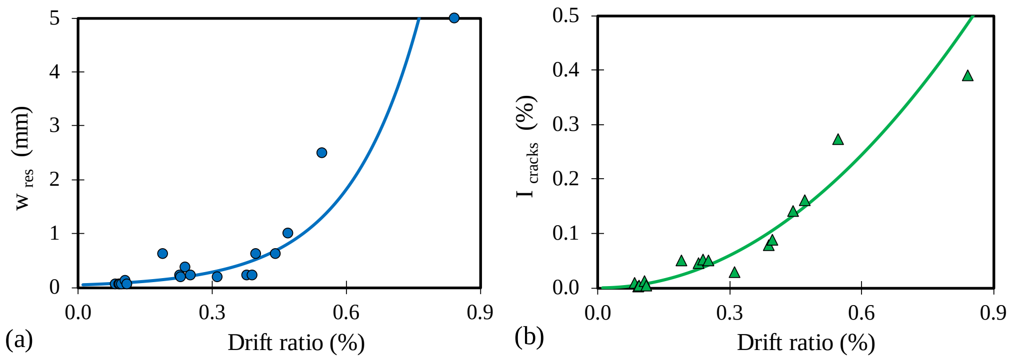

Fig. 1 – Residual cracking: (a) measured residual crack width, (b) residual damage index.

According to Fig. 1, for a story drift ratio equal to 0.5%, w_{res} and I_{cracks} are equal to 0.98 mm and 0.17%, respectively. For a RC wall with h_w/l_w = 1.0 and h_w = 2.4 m, the total area of residual cracks is roughly equal to 9800 mm^2 (2400 mm × 2400 mm × 0.17%), and thus, the total length of cracks with a crack width equal to 0.98 mm is roughly equal to 10 m (9800 mm^2 / 0.98 mm). In this way, the length of cracks to be repaired can be estimated.

3.2 Limiting values

Damage indexes also require the definition of the limiting values of cracking after which failure occurs. Limiting values of w_{res} and I_{cracks} are included in Table 2. Limiting values of residual cracking (w_{res}, I_{cracks}) associated to the defined limiting values of story drift ratios were established using the damage observed during shaking table tests of RC walls.

Table 2 – Proposed performance indicators

Performance level	Expected damage	Indicator	Type of web shear reinforcement	
			Deformed bars	Welded-wire mesh
IO	*Minor damage:* - Flexural cracking at the boundary elements and minor web inclined cracks.	R_{allow}	0.15 %	0.10 %
		w_{res}	0.10 mm	0.08 mm
		I_{cracks}, %	---	---
LS	*Moderate damage:* - Extension of web inclined cracks to the wall edges without penetration into the boundary elements.	R_{allow}	0.40 %	0.25 %
		w_{res}, mm	0.50 mm	0.20
		I_{cracks}, %	0.10 %	0.04 %
CP	*Significant damage:* - Noticeable web diagonal cracking and/or yielding of some web steel bars/wires. - Moderate web crushing of concrete and damage around openings.	R_{allow}	0.65 %	0.35 %
		w_{res}	2.5 mm	0.4 mm
		I_{cracks}	0.30 %	0.08 %

4. Damage index based on fractal dimension of cracking

Crack width is one of the main indicators of damage severity experienced by RC structural components during an earthquake. However, damage quantification based on visual inspection of cracking pattern can be a subjective estimate because the damage criterion depends on the expertise of the inspector engineer. Although characteristics of cracks (length, maximum width, residual width) are key indicators of structural damage, pattern and distribution of cracks of the damaged structural component should be also considered. The structural engineering community has applied the approach of fractal theory for proposing alternative and innovative methodologies of damage and performance evaluation. Fractals are a tool describing the self-similarity in a complex and irregular geometric object or a physical system. The Fractal Dimension (*FD*) is a mathematical parameter that measures the geometric complexity level of a pattern rather than evaluates the filling property of a particular geometric plane or space.

4.1 Damage index

To improve the quantitative analysis of structural damage, Carrillo *et al*. [8] developed an empirical damage index (*DI*) for rapid estimation of the damage level and the residual performance of thin and lightly-reinforced concrete walls subjected to seismic demands. Taking into account the relationship between the damage level and the cracking pattern of walls, the modified damage index proposed by Carrillo *et al.* [8] is based on the *FD* of the pattern and propagation of cracking recorded in thin and lightly-reinforced concrete walls for low-rise housing. Such damage index is computed using Eq. (2) and the parameters included in the index are shown in Fig. 2a.

$$DI = \frac{FD_i - FD_{ini}}{FD_u - FD_{ini}} \tag{2}$$

where FD_i is the fractal dimension of the current condition of visible cracks (e.g., in the i^{th} inspection), and FD_{ini} is the fractal dimension computed once the cracks (including shrinkage cracks) become visible for the first time ("initial stage"). According to Carrillo *et al.* [8], for code-based seismic design and rehabilitation, FD_i in Eq. (2) may be the fractal dimension of the current stage of visible cracks related to a particular limit state or performance level. The inspector engineer who carried out the post-earthquake inspections should draw or take a picture of the residual cracking pattern of the wall and convert it to digital format for computing the value of the fractal dimension. According to Carrillo *et al.* [8], FD_u is the value of the fractal dimension for surface cracks related to loss of lateral resistance limit state of thin RC walls ("final stage"). Although Carrillo *et al.* [8] define FD_{ini} similarly to the index proposed by Farhidzadeh *et al.* [13], values of FD_{ini} and FD_u were those obtained from the measured response on walls with the particular characteristics of low-rise housing. The damage index proposed by Carrillo *et al.* [8] describes the difference between the current stage of crack pattern and the baseline FD_{min}.

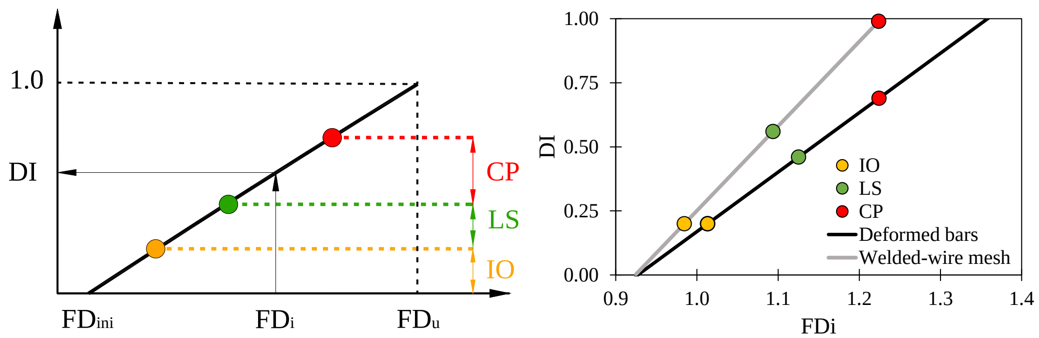

Fig. 2 – Proposed damage index: (a) parameters included, (b) DI in terms of web shear reinforcement.

4.2 Limiting values

Damage index proposed by Carrillo *et al.* [8] includes limiting values associated with four limit states: diagonal cracking, maximum shear strength, loss of lateral resistance, and failure of the wall or end of test. The damage index proposed by Carrillo *et al.* [8] also includes limiting values related to three performance levels of RC walls for low-rise housing: immediate occupancy (IO), life safety (LS) and collapse prevention (CP). Table 3 shows the expected damage index at defined limit states (DI_{cr}, DI_{max} and DI_u) and performance levels (DI_{IO}, DI_{LS} and DI_{CP}). Damage indexes were arranged in terms of aspect ratio of walls, type of concrete, web steel ratio, type of web shear reinforcement, and type of testing. As shown in the Table 3, walls reinforced with deformed bars and welded-wire mesh have attained 20% of the performance capacity at the IO performance level (DI_{IO} = 0.20). At the LS performance level, walls reinforced with deformed bars and walls with welded-wire mesh have reached 46% (DI_{LS} = 0.46) and 56% (DI_{LS} = 0.56) of the performance capacity, respectively. At the CP performance level, walls reinforced with deformed bars and walls with welded-wire mesh have attained 69% (DI_{CP} = 0.69) and 99% (DI_{CP} = 0.99) of the performance capacity, respectively. Although such two types of walls with different web shear reinforcement have comparable shear strength capacities [2], residual capacity at CP performance level of walls with welded-wire mesh is scarcely 1% (1-0.99) while such capacity of walls with deformed bars is 31% (1-0.69). The damage index proposed by Carrillo *et al.* [8] can be also used for cost estimation of seismic rehabilitation. For instance, these significant differences of residual capacity are directly related to lower costs of seismic rehabilitation of walls with deformed bars when compared with wall with welded-wire mesh.

Table 3 – Damage index associated to limit states and performance levels in terms of different variables

Variable		Limit states			Performance levels		
		DI_{cr}	DI_{max}	DI_u	DI_{IO}	DI_{LS}	DI_{CP}
Aspect ratio (h_w/l_w)	h_w/l_w = 2.0	0.13	0.79		0.09	0.46	0.87
	h_w/l_w = 1.0	0.26	0.75	1.00	0.21	0.47	0.71
	h_w/l_w = 0.5	0.21	0.75		0.19	0.48	0.79
Type of concrete	Normalweight	0.24	0.77		0.21	0.47	0.78
	Lightweight	0.24	0.76	1.00	0.18	0.49	0.73
	Self-consolidating	0.21	0.72		0.14	0.40	0.63
Web steel ratio	0% ρ_{min}	0.20	0.82		0.05	0.20	0.25
	50% ρ_{min}	0.22	0.81	1.00	0.18	0.49	0.85
	100% ρ_{min}	0.26	0.69		0.22	0.47	0.69
Type of web reinforcement	Deformed bars	0.24	0.69		0.20	0.46	0.69
	Welded-wire mesh	0.23	0.91	1.00	0.20	0.56	0.99
	No reinforcement (0% ρ_{min})	0.20	0.82		0.05	0.20	0.25
Type of testing	Quasi-static monot.	0.18	0.67		0.11	0.29	0.43
	Quasi-static cyclic	0.25	0.76	1.00	0.20	0.47	0.76
	Shake table	0.23	0.83		0.27	0.63	0.98

5. Damage index based on stiffness degradation

Widely used global damage indexes, such as ductility and drift ratios, may fail to take into account the fact that repeated loading cycles at a given amplitude generally cause more damage than a single cycle. To improve the quantitative analysis of structural damage of low-rise RC walls under a particular seismic excitation, an empirical energy-based low-cycle fatigue damage index, *DI*, was proposed by Carrillo [9]. This index is computed using Eq. (3) and the functional form is shown in Fig. 3a. Since significant reduction in stiffness is expected to occur with the reversed loading, this index correlates the stiffness degradation and the destructiveness of the earthquake in terms of the duration and intensity of the ground motions. Initially, Carrillo [9] developed a stiffness degradation model of low-rise RC walls that considers simultaneously the increment of damage associated to low-cycle fatigue, energy dissipation and the cumulative cyclic parameters, such as displacement demand and hysteretic energy dissipated. Then, Carrillo [9] examined the relationship between earthquake characteristics and number of cycles on the seismic response. Results measured during shake table tests were used to develop the equations of the damage index, and therefore, Carrillo [9] argued that the actual strain rate induced by earthquake excitation was suitably included.

$$DI = 1 - \left(\frac{K}{K_o} \right) \tag{3}$$

where (K/K_0) is defined as the ratio between cycle stiffness and initial stiffness, associated with the number of equivalent hysteretic cycles for a given value of drift ratio, *R*. Carrillo [9] found that the two-branch model shown in Fig. 3b was suitable for describing stiffness degradation (K/K_0) of low-rise RC walls subjected to dynamic loads. The stiffness degradation model is divided into two branches that join at drift ratio R'. According to test data reported by Carrillo [9], R' represents the drift ratio associated to peak shear strength for a given value of *N*. Stiffness degradation rate of the first branch was higher than that of the second branch. The shape of the proposed model is slightly different to the typical function reported of stiffness degradation in quasi-static tests that follows a continuous decaying curve (see dotted line in Fig. 3b).

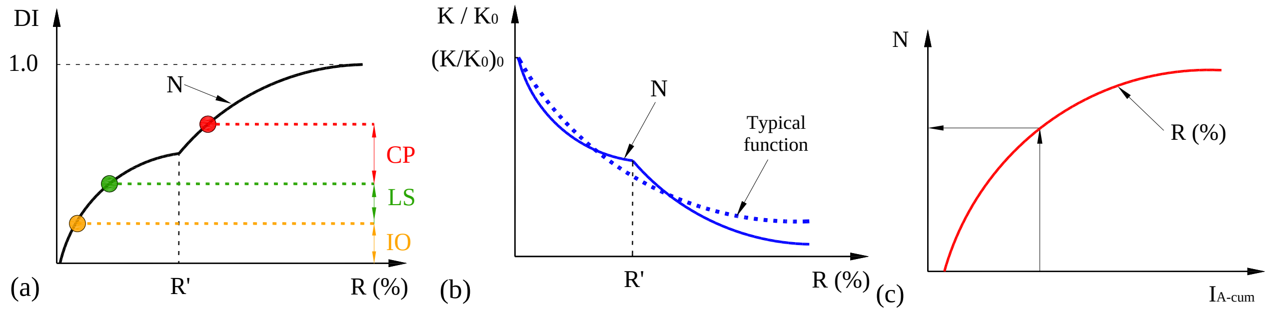

Fig. 3 – Proposed damage index: (a) functional form, (b) stiffness degradation model, (c) seismic demand model.

Carrillo [9] proposed a nonlinear K/K_0 model that depends mainly on the number of cycles associated to a constant value of drift ratio N and on drift ratio R. The functional form of the K/K_0 model is computed using Eq. (4).

$$\frac{K}{K_0} = a_1 \, R^{b_1} \qquad R < R' \tag{4}$$
$$\frac{K}{K_0} = a_2 \, R^{b_2} \qquad R \geq R'$$

where R' should be calculated using Eq. (5). The number of equivalent hysteretic cycles at a given range of drift ratio, N, should be estimated using Eq. (6) [4].

$$R' = c_1 \ln N + c_2 \geq 0 \tag{5}$$

$$N = \frac{E_{cum-i}}{E_j} \tag{6}$$

Parameter N can be considered as an energy-based low-cycle fatigue parameter and refers to equivalent hysteretic cycles as it represents the maximum value of the ratios between the cumulative energy dissipated in cycle i, E_{cum-i}, and the energy dissipated that is associated to a range of drift ratio j, E_j, which is calculated using Eq. (7).

$$E_j = \frac{1}{n_1} \sum_{i=1}^{n_1} E_i \tag{7}$$

where E_i is the energy dissipated in cycle i and n_1 is the number of cycles that are associated to a given range of drift ratio. Carrillo [9] found that wall geometry was the main parameter affecting stiffness behavior [4]. Therefore, two wall categories were identified for sorting test data of stiffness degradation: category A for solid walls and category B for walls with openings. Hence, the proposed values for constants c_1 and c_2 are presented in Table 4. In Eq. (4), variables a_1 and a_2 depend on $(K/K_0)_0$ and R', and they should be calculated using Eqs. (8) and (9), respectively. Carrillo [9] observed that the effect of openings increases the damage index in walls when compared with solid walls.

$$a_1 = \frac{(K / K_0)_0}{R_0^{\,b_1}} \tag{8}$$

$$a_2 = a_1 \, R'^{\,(b_1 - b_2)} \tag{9}$$

where drift ratio R_0 represents the minimum value of drift ratio to be used in the model for assuring mathematical stability and was purposely established as 0.005%, and $(K/K_0)_0$ is the initial stiffness degradation associated to R_0 and should be calculated using Eq. (10).

$$(K / K_0)_0 = f_1 N^{g_1} \tag{10}$$

where constant f_1 also depends on wall geometry and values are presented in Table 4. Variable g_1 depends on f_1 and N_{di} and should be calculated using Eq. (11).

$$g_1 = \frac{\log (1/ f_1)}{\log N_{di}} \tag{11}$$

where N_{di} is the minimum number of equivalent hysteretic cycles observed during dynamic testing (N_{di} = 21). The range of the number of cycles, N, should be N_{di} and the maximum number of steady cycles recorded during dynamic testing ($\sim 300 \geq N \geq N_{di}$).

Table 4 – Constants for the stiffness degradation model proposed

Category	Wall geometry	c_1	c_2	d_1	d_2	e_1	e_2	f_1	FK
A	Solid	-0.123	0.846	0.077	-0.427	-0.215	-0.623	4.66	0.183
B	Openings	-0.123	0.826	0.061	-0.459	-0.188	-0.610	5.49	0.189

In Eqs. (4), (8) and (9), variables b_1 and b_2 depend mainly on N and should be calculated using Eqs. (12) and (13), respectively.

$$b_1 = d_1 \ln N + d_2 \leq -0.001 \tag{12}$$

$$b_2 = e_1 \ln N + e_2 \tag{13}$$

Upper and lower limits of dynamic stiffness degradation can be computed using Eq. (14), which is a modification of Eq. (10).

$$(K / K_o)_{o(env)} = (K / K_0)_0 \, (1 \pm FK) \tag{14}$$

where FK represents the mean value of coefficients of variation of ratios between values predicted using Eqs. (3) to (10), and test data. Values of FK proposed by Carrillo [9] for the two wall geometries are presented in Table 4. Considering that design and assessment processes involve prediction of the maximum and the minimum probable capacities, respectively, upper limit of Eq. (14) can be used, for example, for design purposes, and the mean value or the lower limit for assessment of seismic performance [9].

5.1 Seismic demand model

Carrillo [9] argued that the first step to calculate K/K_0 ratio of a low-rise RC wall subjected to a particular earthquake record, is to estimate, for a given value of drift R, the number of equivalent hysteretic cycles N (at the given range of drift ratio) induced by an earthquake. The model shown in Fig. 3c was proposed by Carrillo and Alcocer [14] for representing the relation between earthquake demand and parameters that define the degradation model. Eq. (15) was proposed by Carrillo [9] to estimate the N value from the cumulative Arias intensity of the earthquake records, $I_{A\text{-}cum}$, which is expressed in m/s. A cumulative Arias intensity, $I_{A\text{-}cum}$, was used by by Carrillo and Alcocer [14] for considering the effect of the entire series of earthquake records applied during shake table testing. This cumulative parameter represents the Arias intensity of the earthquake record being analyzed plus the Arias intensity of the preceding records.

$$N = \left(\frac{\mathrm{I}_{A-cum}}{\mathrm{I}_{A-1}} \right)^{x} \tag{15}$$

where $\mathrm{I}_{A\text{-}1}$ is the Arias intensity for N = 1 and for a given story drift, and x is a fitted parameter. Parameters $\mathrm{I}_{A\text{-}1}$ and x depend on the story drift of the equivalent hysteretic cycles, R; a value of R may be the story drift at a given performance level. From trends of experimental results, Carrillo and Alcocer [14] proposed Eqs. (16) and (17) for calculating these two parameters. Upper and lower limits of number of equivalent hysteretic cycles are computed with Eq. (18).

$$\mathrm{I}_{A-1} = 1.25\, R^{\,1.75} \tag{16}$$

$$x = 1.05\, R^{\,0.01} \tag{17}$$

$$\mathrm{I}_{A-1(env)} = \mathrm{I}_{A-1}\,(1 \pm FN) \tag{18}$$

where FN represents the mean value of the coefficients of variation of the ratios between values predicted using Eqs. (15) to (17), and test data (FN = 0.267). In the same way to stiffness degradation model, upper and lower limits of Eq. (18) can be used for seismic design or for performance assessment of low-rise RC walls, respectively.

5.2 Limiting values

Table 5 shows the limiting values of damage index proposed by Carrillo [9] and the associated expected damage levels [7]. The expected damage level was described and defined exclusively from damage observed during shake table testing of RC walls. In Table 5, limiting values of proposed damage index are also associated to the three performance levels of RC walls for low-rise housing [7].

Table 5 – Performance indicators in terms of the damage index

Performance level	*DI*	Damage level
Immediate occupancy (OI)	$DI < 0.10$	No damage
	$0.10 \leq DI < 0.20$	Minor damage
Life safety (LS)	$0.20 \leq DI < 0.40$	Moderate damage
Collapse prevention (CP)	$0.40 \leq DI < 0.70$	Significant damage
---	$0.70 \leq DI < 0.95$	Severe damage
	$DI \geq 0.95$	Potential for collapse

6. Damage index based on Park and Ang approach

One of the most widely used DI is the formulation proposed by Park & Ang [10, 11]. Although damage index formulated by Park & Ang is currently implemented in several computational tools, the index has not been calibrated for squat and thin elements controlled by shear deformations. The original formulation proposed by Park & Ang [10, 11] consists of a linear combination of the damage caused by excessive post-elastic deformations and the energy dissipated by the hysteresis or repeated cyclic loading effect of the element, as shown in Eq. (19).

$$DI_{PA} = \frac{u_{\max}}{u_{mon}} + \beta \frac{E_H}{F_y\, u_{mon}} \tag{19}$$

where u_{max} is the ultimate deformation recorded on the element due to reversed-cyclic loading, u_{mon} is the ultimate deformation recorded on the element for monotonic loading, E_H is the total energy dissipated by hysteresis cycles, F_y is the yield strength of the element, and β is a non-negative parameter. The ultimate deformation registered on the specimen during monotonic loading (u_{mon}) was not available for all the specimens included in the experimental program reported by Carrillo *et al*. [12], because not all cyclic tests were accompanied by monotonic tests. Hence, Carrillo *et al*. [12] estimated this variable from the envelope of the hysteresis curves recorded for each of the walls. The ultimate deformation related to monotonic test was assimilated to the ultimate deformation taken from the envelope curve of the measured hysteresis cycles (δ_{um}), applying the aforementioned 20% strength degradation criterion. Based on those results, Carrillo *et al*. [12] estimated that δ_{um} was 1.3 times larger than the ultimate deformation recorded on the envelope curve of the hysteresis cycles (δ_{uce}). Hence, δ_{um} was used in Eq. (19) instead of u_{mon} to calculate DI_{PA}.

Since yielding of longitudinal bars in squat reinforced concrete walls is not observed in the same way than in columns, Carrillo *et al*. [12] found that the yield strength of the squat element (F_y) is equivalent to 80% of the maximum strength recorded in the wall [2, 7]. Finally, the total energy dissipated by hysteresis cycles (E_H) corresponded to the sum of all areas encircled by the hysteresis cycles during the entire reversed-cyclic test until the ultimate state condition related to the 20% strength degradation criterion was observed.

Carrillo *et al*. [12] proposed a novel formulation for the parameter β included in the Park & Ang damage index. Initially, regression analyses were performed considering some of the variables originally considered by Park & Ang [10, 11], i.e. aspect ratio (h_w/l_w) and web transversal reinforcement ratio (ρ_w); while others variables were excluded, such as type of concrete, longitudinal reinforcement (ρ_l) and normalized axial load (n_0). The type of concrete was excluded because it is not commonly used as a design parameter (type of concrete) for practical design. In addition, Carrillo *et al*. [6] reported that initial stiffness, hysteresis curves and energy dissipation of walls made of light-weight and normal-weight concrete was readily comparable. The two latter variables (ρ_l and n_0) were also excluded because they became less relevant in the dominant shear failure mode of these walls and less significant in the case of low-rise buildings; i.e. values of ρ_l and n_0 are low and almost constant for the prototype of low-rise house.

Cumulative ductility (μ_{cum}) was considered as a relevant variable to calculate β based on previous studies [15]. This parameter is defined [15] as the sum of the ductility demands that have exceeded the elastic limit as presented in Eq. (20).

$$\mu_{cum} = \frac{\sum_{k=1}^{n} \delta_k}{\delta_y} \tag{20}$$

where δ_y is the deformation at the top of the wall at flexural yield condition, and δ_k is the maximum plastic deformation at the top of the wall for cycle k that is computed using Eq. (21). As explained previously, since the flexural yield condition was not observed in the squat reinforced concrete walls, δ_y was associated to the yield strength F_y that was computed as 80% of the maximum strength recorded in the wall.

$$\delta_k = \begin{cases} 0 \; when \; \delta_k < \delta_y \\ \delta_k \; when \; \delta_k > \delta_y \end{cases} \tag{21}$$

Carrillo *et al*. [6] analyzed the correlation between these four key parameters (ρ_w, h_w/l_w and μ_{cum}) and β_{exp}. The highest correlation coefficient was obtained when the influence of μ_{cum} was analyzed. Hence, Carrillo *et al*. [6] selected parameters μ_{cum} and ρ_w as key parameters to develop a novel mathematical function to calculate β in the case of squat, thin and lightly-reinforced concrete walls. The model represented in Eq. (22) was found by Carrillo *et al*. [6] as the model with the highest correlation for combining both parameters ρ_w and μ_{cum}. Graphical representations of these models are presented in Fig. 4a. As shown in Fig. 4b, the model β_3 underestimates slightly parameter β mainly for walls with low ductility capacity that resulted in large values for β_{exp} (for example; walls with web shear reinforcement made of welded-wire meshes).

$$\beta_3 = 1.39 \times \left(\frac{\mu_{cum}}{0.001^{\rho_w}} \right)^{-0.42} \quad (22)$$

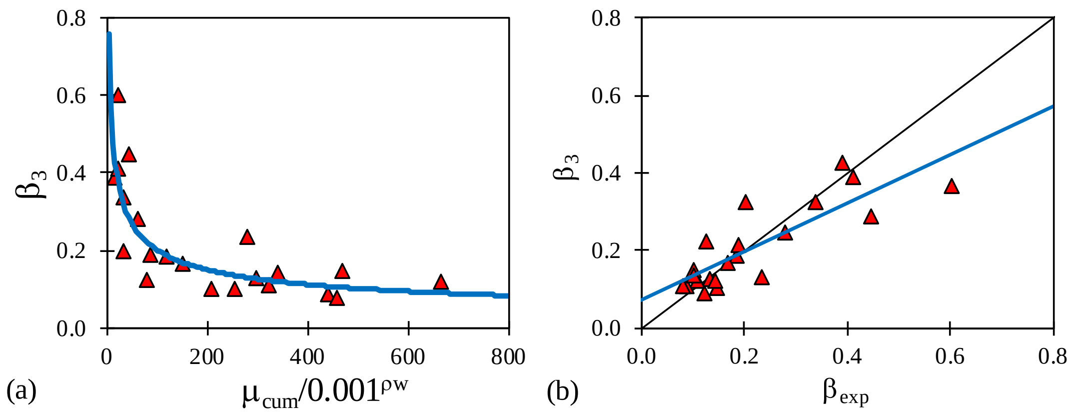

Fig. 4 – Model β_3: (a) graphical representation, (b) comparison between β_{exp} and model β_3.

7. Conclusions

Parameters, procedures, advantages, drawbacks and limiting values of four damage indexes for damage assessment of thin and low-rise RC walls were summarized, compared and discussed in the paper. These four damage indexes were calibrated using the experimental results obtained from quasi-static monotonic and reversed-cyclic test performed on 39 thin RC wall specimens. The four damage index includes limiting values and expected damage associated to limit states and performance levels for code-based seismic design and rehabilitation of low-rise RC walls. Those values allow for estimating the damage stage of thin and lightly-reinforced concrete walls after an earthquake and thus, allow for computing the residual capacity of walls. The first one was an index based on the relation between the damaged area (area of cracks) and the area of the façade of the RC wall. The second one was an index based on the fractal dimension of the cracking recorded after an earthquake. This approach considers implicitly parameters such as pattern and distribution of cracking which removes the subjectivity and the variability associated with damage assessment based on visual inspection.

Low-cycle fatigue is representative of structures subjected to earthquakes and is defined as the failure of critical elements at deformations levels approaching or exceeding the yield stress of the structure. Damage indexes that account for low cycle fatigue should explicitly consider the effect of cumulative loading. The third approach was an energy-based low-cycle fatigue index based on the stiffness degradation of walls. This index considers simultaneously the increment of damage associated to the low-cycle fatigue, the distribution of the amplitude of plastic cycles, and the cumulative cyclic parameters such as displacement demand and hysteretic energy dissipated by RC walls dominated by shear deformations. The index is a useful and effective tool for quantitative assessment of low-rise RC walls to specified random ground motions. The four one was the broadly used Park & Ang damage index that includes a novel formulation for squat, thin and lightly-reinforced concrete walls, because its failure mode was dominated by shear instead of flexural deformations. Comparison between the computed damage index and crack pattern evolution observed in the walls at different damage states demonstrated the ability of the model to numerically assess the damage of the wall specimens for different performance levels when applying the Park & Ang damage index that includes the novel formulation. All these indexes are intended to improve the quantitative analysis of structural damage under a particular seismic excitation. These indexes have the potential to be incorporated in future design procedures as a design variable for damage evaluation, structural assessment, retrofitting decision-making and disaster-planning of low-rise housing having RC walls with the characteristics of those walls described in the research program and subjected to earthquake-induced deformations.

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