Performance Indicators for Seismic Design of Concrete Walls for Housing

Julian Carrillo
Research professor, Universidad Militar Nueva Granada, UMNG, Bogota, Colombia

Sergio M. Alcocer
Research professor, Universidad Nacional Autónoma de México, UNAM, Mexico, D.F.

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SUMMARY:
Performance indicators of the structural response of walls for low-rise concrete housing were developed. Proposed values are applicable within a performance-based seismic design framework. Performance indicators are based on acceptance limits of allowable story drift ratios. The performance indicators proposed herein were derived from test observations and measured response of 36 reinforced concrete walls specimens during shaking table and quasi-static testing. The experimental program included walls with different height-to-length ratios (0.5, 1.0 and 2.0) and walls with openings. Variables studied were the type of concrete (normalweight, lightweight and self-consolidating), web shear steel ratio (0.125% and 0.25%) and the type of web shear reinforcement (deformed bars and welded-wire meshes).

Keywords: performance indicators, acceptance limits, concrete walls, low-rise housing.

1. INTRODUCTION

Construction of low-rise (one-to-two stories high) housing units using reinforced concrete walls has considerably increased in Latin American countries, such as Mexico and Peru. Due to the lateral stiffness and strength of concrete wall structures, seismic force and displacement demands are, most of the times, relatively low. This phenomenon has prompted housing developers to use walls with low concrete compressive strengths (between 15 and 20 MPa) and small thickness (100 mm). Regarding the minimum wall web reinforcement ratio (0.25% in both directions according to ACI Building Code, 2011), when seismic demands do not control design, as it is very often the case, design professionals consider such minimum value to be excessive for controlling diagonal tension cracking. Thus, web steel ratios smaller than that prescribed by the codes are commonly used in practice. Moreover, because of its ease during placement, welded-wire meshes are widely employed as and wall web reinforcement. However, robust technical evidence on the adequacy of using smaller ratios of steel reinforcement and welded-wire meshes when walls are subjected to seismic demands is still scarce.

The first process is to assess the quality of correlation between available models and test results of walls with the characteristics described above. From this process, it was concluded that such correlation was poor (Carrillo and Alcocer, 2012). The peculiar wall characteristics used in low-rise housing are most likely the main reason of this finding. In effect, code design requirements have been derived so that they are applicable to walls with distinctly different characteristics, so that predicted capacity is meant to be a lower bound value. This fact leads to an excessive conservatism, and thus points to an unjustifiable excessive cost of the housing unit, especially if such houses are meant for low-income population.

The aim of this paper is to discuss and present performance indicators of structural response as they relate to the expected wall damage. Performance indicators were derived for different damage levels (performance levels), within a performance-based seismic design (PBSD) framework. Selected performance indicator was allowable story drift ratio because it is a common performance indicator in structural design. Allowable story drift ratios and expected damage were determined from shaking table and quasi-static tests.

2. EXPERIMENTAL PROGRAM

The experimental program comprised testing of 36 isolated cantilever walls. Variables studied were those obtained from current design and construction practice for low-rise housing in Latin America. Quasi-static (monotonic and reversed-cyclic) and dynamic (shaking table) testing series were included. Owing to limitations in the payload capacity of the shaking table equipment used for testing, lightly-reduced scaled models were designed and built (i.e. geometry scale factor, S_L = 1.25) for shaking table testing. A detailed description of the experimental program may be found elsewhere (Carrillo and Alcocer, 2011, 2012). The following variables were studied:

2.1. Height-to-length ratio

Walls with height-to-length ratio (h_w/l_w) equal to 0.5, 1.0 and 2.0, and walls with openings (door and window openings) were tested. Full-scale wall thickness, t_w, and clear height, h_w, were 100 mm and 2.4 m, respectively. Then, to achieve the height-to-length ratio, length of walls was varied.

2.2. Concrete type

Normalweight (N), lightweight (L) and self-consolidating (S) concrete were included in the test series. Nominal concrete compressive strength, f_c', was 15 MPa for all types of concrete. Ranges of measured mechanical properties of concrete for the 36 specimens are presented in Table 1. These properties were obtained at the time of testing walls.

Table 1. Measured mechanical properties of concrete.

Property	Normalweight, N	Lightweight, L	Self-consolidating, S
Compressive strength, f_c, MPa	16.0 – 24.7	10.8 – 26.0	22.0 – 27.1
Elastic modulus, E_c, MPa	8430 – 14750	6700 – 10790	8900 – 11780
Tensile splitting strength, f_t, MPa	1.55 – 2.20	1.14 – 1.76	1.58 – 1.98
Flexural strength, f_r, MPa	2.32 – 3.75	1.43 – 3.29	2.27 – 2.48
Specific dry weight, γ, kN/m^3)	18.8 – 20.3	15.2 – 18.3	18.9

2.3. Web steel ratio

Two web steel ratios, 100% of ρ_{min} (0.25%) and 50% of ρ_{min} (0.125%), were used. Minimum web steel ratio (ρ_{min}) was that prescribed by ACI-318 (2011). Web reinforcement was placed in a single layer at wall mid-thickness and same ratios of horizontal and vertical reinforcement (ρ_h = ρ_v) were used.

2.4. Type of web reinforcement

Deformed bars (D) and welded-wire meshes made of small-gage wires (W) were used. Nominal yield strength of bars and wire reinforcement, f_y, was 412 MPa (for mild steel) and 491 MPa (for cold-drawn wires). Ranges of measured mechanical properties of steel reinforcement for the 36 specimens are presented in Table 2. The behavior of the cold-drawn wire reinforcement was characterized by fracture of material with a slight increment of strain (see the Elongation row).

2.5. Boundary elements

Thickness of boundary elements was equal to web thickness. To assess wall lateral shear strength, longitudinal boundary reinforcement was purposely designed to prevent flexural failure prior to achieving a shear failure. Typical geometry and reinforcement layout of some of the full-scale wall specimens are shown in Fig. 1.

Table 2. Measured mechanical properties of steel reinforcement.

Location in the wall	Boundary: deformed bar	Web: deformed bar, D	Web: welded-wire, W
Type	Mild	Mild	Cold-drawn
Yield strength, f_y, MPa	411 – 456	435 – 447	605 – 630
Ultimate strength, f_{su}, MPa	656 – 721	659 – 672	687 – 700
Elongation, %	9.1 – 16.0	10.1 – 11.0	1.4 – 1.9

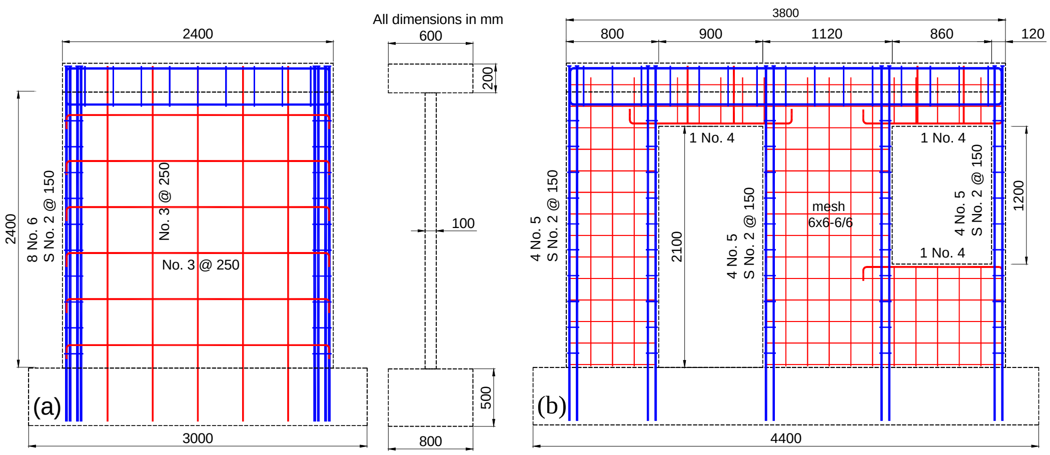

Figure 1. Geometry and reinforcement layout of some wall specimens: (a) h_w/l_w = 1.0, 100% of ρ_{min} and using deformed bars; (b) wall with openings, 50% of ρ_{min} and welded-wire mesh.

2.6. Tests setups

Walls were tested under quasi-static reversed-cyclic loading history; selected wall characteristics were used for tests in shaking table. In quasi-static testing, loading protocol consisted of a series of increasing amplitude cycles. For each increment, two cycles at same amplitude were applied. An axial compressive stress of 0.25 MPa was applied on top of the walls and kept constant during testing. This value corresponded to an average axial stress in the first floor walls of a two-story prototype house. During shaking table testing, models were subjected to a series of base excitations represented by earthquake records associated to three limit states. A mass-carrying load system for supporting the mass and transmitting the inertia forces was purposely developed (Carrillo and Alcocer, 2011).

2.7. Measured response

Main characteristics and failure modes of the 36 wall specimens are presented in Table 3. Measured drift capacities at four limit states (R_{cr}, R_{max}, R_u and R_{uu}) and measured drifts associated to three performance levels (R_{IO}, R_{LS} and R_{CP}) are included in Table 3. Performance levels (IO, LS and CP) are later defined in this paper. For evaluating the observed wall behavior, three failure modes were defined: a) when yielding of the majority of the web shear reinforcement and no web crushing of concrete was observed, a diagonal tension failure (DT) was defined; b) when yielding of some steel bars or wires and noticeable web crushing and spalling of concrete was observed, a diagonal compression failure (DC) was defined, and, c) when yielding of the majority of the web steel reinforcement and noticeable web crushing of concrete was observed, a mixed failure mode (DT-DC) was defined. Test results indicated that the contribution of wall sliding to the whole deformation was negligible for all tests. Therefore, wall sliding at the base (SL) was not purposely included.

Walls reinforced with 50% of the minimum code prescribed web steel reinforcement ratio and using deformed bars or welded-wire mesh, exhibited DT failure. Failure mode was governed by web inclined cracking of concrete at approximately 45-degree angle and yielding of most of web shear reinforcement prior to severe strength and stiffness decay. In walls reinforced with welded-wire meshes, fracture of wires after plastic yielding of web shear reinforcement was observed. Failure was brittle because of the limited elongation capacity of the wire mesh itself (see Table 2). In contrast, walls reinforced using deformed bars and minimum web steel ratio exhibited DT-DC failure. Typical final crack patterns of walls during shaking table testing are shown in Fig. 2. Walls made of normalweight and self-compacting concretes showed comparable behaviors. As expected, walls made of lightweight concrete exhibited lower strengths and larger flexibility.

Table 3. Main characteristics and measured displacement capacities of wall specimens.

Type of testing*	Wall	Type of concrete	Type of web reinforcement	h_w / l_w	$\rho_h = \rho_v$, %	Failure mode	R_{cr} , %	R_{max} , %	R_u , %	R_{uu} , %	Immediate occupancy: R_{IO} , %	Immediate occupancy: R_{cr}/R_{IO}	Life safety: R_{LS} , %	Life safety: R_{max}/R_{LS}	Collapse prevention: R_{CP} , %	Collapse prevention: R_{uu}/R_{CP}
SC	MRN50mC	N	W	0.44	0.12	DT	0.03	0.39	0.45	0.45	0.02	1.50	0.11	3.37	0.39	1.17
SC	MRL50mC	L	W	0.45	0.12	DT	0.05	0.44	0.45	0.45	0.03	1.63	0.26	1.71	0.44	1.01
SC	MRNB50mC	N	W	0.44	0.13	DT	0.04	0.40	0.67	0.78	0.01	2.78	0.15	2.73	0.40	1.94
SC	MCN50mC	N	W	1.00	0.12	DT	0.11	0.47	0.52	0.54	0.02	2.85	0.21	2.23	0.47	1.16
SC	MCL50mC	L	W	1.01	0.12	DT	0.12	0.60	0.63	0.63	0.05	2.91	0.32	1.89	0.60	1.06
SC	MCNB50mC	N	W	1.00	0.12	DT	0.05	0.34	0.40	0.40	0.01	2.02	0.14	2.36	0.34	1.19
DY	MCN50mD	N	W	1.00	0.11	DT	0.09	0.44	0.54	0.54	0.05	1.68	0.28	1.58	0.44	1.23
DY	MCL50mD	L	W	1.00	0.11	DT	0.14	0.62	0.65	0.65	0.05	3.11	0.35	1.76	0.62	1.05
SC	MEN50mC	N	W	1.94	0.12	DT	0.16	0.66	0.68	0.68	0.07	2.62	0.39	1.69	0.66	1.03
SC	MEL50mC	L	W	1.99	0.12	DT	0.21	0.70	0.71	0.71	0.10	2.57	0.43	1.64	0.70	1.00
SC	MVN50mC	N	W	*	0.11	DT	0.06	0.40	0.40	0.60	0.03	2.18	0.18	2.25	0.40	1.52
DY	MVN50mD	N	W	*	0.11	DT	0.05	0.40	0.44	0.72	0.05	1.19	0.22	1.83	0.40	1.79
SC	MRN50C	N	D	0.45	0.14	DT	0.10	0.69	1.01	1.69	0.06	1.93	0.39	1.75	0.69	2.45
SM	MCN50M	N	D	1.01	0.14	DT	0.10	1.01	1.98	2.01	0.08	1.33	0.51	1.98	1.01	2.00
SM	MCL50M	L	D	1.01	0.14	DT	0.14	0.68	1.20	1.86	0.10	1.40	0.43	1.57	0.68	2.74
SC	MCN50C	N	D	1.01	0.14	DT	0.07	0.66	1.02	1.03	0.03	1.95	0.27	2.44	0.66	1.57
SC	MCS50C	S	D	1.01	0.14	DT	0.13	1.01	1.03	1.06	0.02	1.13	0.53	1.92	1.01	1.05
SC	MCL50C	L	D	1.01	0.14	DT	0.07	0.57	0.69	0.70	0.05	1.86	0.32	1.78	0.57	1.23
SC	MCN50C-2	N	D	1.00	0.14	DT	0.11	0.44	0.72	1.40	0.04	2.20	0.21	2.08	0.44	3.21
SC	MCS50C-2	S	D	1.00	0.14	DT	0.06	0.39	0.59	1.61	0.04	2.06	0.20	1.94	0.39	4.11
SC	MCL50C-2	L	D	0.99	0.14	DT	0.11	0.57	1.18	2.23	0.07	1.63	0.32	1.80	0.57	3.91
SC	MEN50C	N	D	1.95	0.14	DT	0.24	1.16	2.07	3.15	0.13	1.83	0.67	1.72	1.16	2.71
SC	MRL100C	L	D	0.45	0.28	SL	0.10	0.57	1.20	1.68	0.06	1.75	0.37	1.52	0.57	2.96
SC	MRN100C	N	D	0.45	0.28	DC-SL	0.10	0.60	0.79	1.05	0.06	1.75	0.38	1.59	0.60	1.73
SM	MCN100M	N	D	1.01	0.28	DC-DT	0.10	0.72	1.71	1.71	0.07	1.45	0.42	1.72	0.72	2.38
SM	MCL100M	L	D	1.01	0.28	DC-DT	0.14	0.98	1.67	1.67	0.10	1.39	0.43	2.26	0.98	1.70
SM	MCS100M	S	D	1.01	0.28	DT-DC	0.15	0.97	2.25	2.25	0.10	1.59	0.58	1.66	0.97	2.33
SC	MCN100C	N	D	1.01	0.28	DC-DT	0.07	0.81	1.34	1.72	0.05	1.92	0.28	2.92	0.81	2.13
SC	MCS100C	S	D	1.01	0.28	DT-DC	0.23	1.01	1.49	1.81	0.15	1.59	0.60	1.68	1.01	1.79
SC	MCL100C	L	D	1.01	0.28	DC	0.12	0.81	0.99	1.30	0.05	2.17	0.63	2.24	0.81	1.62
SC	MCL100C-2	L	D	1.01	0.29	DC	0.18	0.80	1.51	1.70	0.09	1.68	0.45	1.76	0.80	2.13
DY	MCN100D	N	D	1.00	0.26	DT-DC	0.09	0.53	0.58	1.51	0.05	1.76	0.30	1.78	0.53	2.88
DY	MCL100D	L	D	1.00	0.27	DT-DC	0.14	0.50	0.73	1.46	0.06	2.41	0.34	1.48	0.50	2.93
SC	MEN100C	N	D	1.96	0.28	DC-DT	0.24	1.40	1.80	2.50	0.15	1.56	0.75	1.88	1.40	1.78
SC	MVN100C	N	D	**	0.26	DT-DC	0.11	0.67	1.09	2.17	0.07	1.84	0.36	1.86	0.67	3.23
DY	MVN100D	N	D	**	0.26	DT-DC	0.05	0.49	0.82	1.40	0.06	1.74	0.34	1.43	0.49	2.87
Safety levels	Web shear reinforcement made of welded wire mesh					Mean						2.3		2.1		1.3
						Coefficient of variation, CV (%)						27.2		24.6		24.0
	Web shear reinforcement made of deformed bars					Mean						1.8		1.9		2.4
						Coefficient of variation, CV (%)						17.0		17.8		32.9

Notes: * SM and SC= quasi-static (monotonic and reversed-cyclic), DY = dynamic (shaking table), * Wall with openings.

3. PERFORMANCE LEVELS

A performance objective is the description of an acceptable damage level (performance level) of a structure when it is subjected to an earthquake motion associated to a specific intensity (hazard level).

In a PBSD context, performance levels are introduced as limiting values of performance indicators that can be measured in the structural response (Guljas and Sigmund, 2006). When performance levels are established, associated limiting values (performance indicators) became the acceptance criteria whose compliance ought to be verified during subsequent design stages.

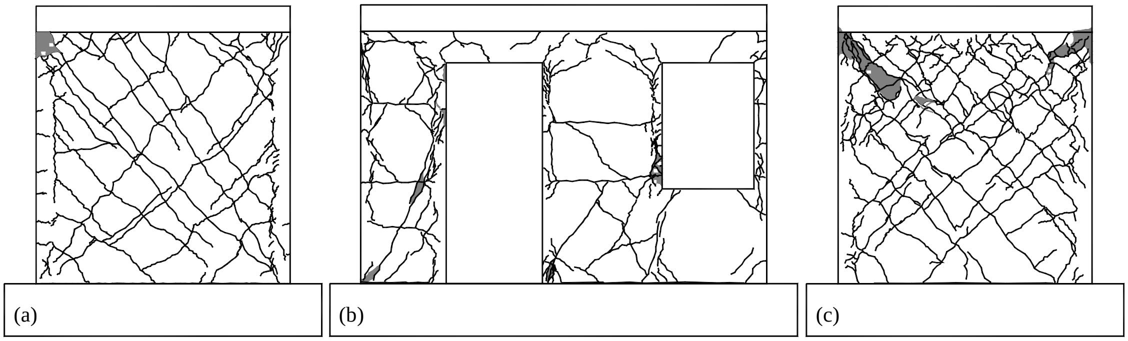

Figure 2. Typical final crack patterns: (a) wall with h_w/l_w = 1.0, 50% of ρ_{min} and using welded-wire mesh (DT failure), (b) wall with openings, 50% of ρ_{min} and using welded-wire mesh (DT failure), (c) wall with h_w/l_w = 1.0, 100% of ρ_{min} and using deformed bars (DT-DC failure).

Up to date, PBSD has been applied basically to systems with a prevailing failure mode under flexure; thus, its implementation for systems failing by shear is still limited. This is the case of reinforced concrete walls failing under shear. One of the main obstacles hindering the implementation of PBSD is the absence of suitable models for predicting the load-displacement curve, as well as of the lack of appropriate performance indicators. From the shape of the envelope of measured hysteresis of specimens described in Table 3, Carrillo and Alcocer (2012) proposed a tri-linear backbone model for describing the seismic performance of walls for low-rise concrete housing. Performance levels and corresponding damage stages are shown in Fig. 3. The tri-linear model refers to three limit states: diagonal cracking (V_{cr}, R_{cr}), peak strength ($V_{\max}$, $R_{\max}$) and ultimate deformation capacity (V_u, R_u). Diagonal cracking limit state is attained when inclined web cracking is observed. Strength limit state corresponds to peak shear strength. Ultimate deformation capacity limit state is associated to any of the two following scenarios: when a 20% drop to the peak shear strength is reached or when web shear reinforcement made of welded-wire meshes fractures. Corresponding drifts at the three limit states for the 36 wall specimens are included in Table 3. R_{uu} is the drift capacity at failure or when the test was finished. A mathematical model of the backbone proposed was developed so that drifts at displacement levels may be calculated (Carrillo and Alcocer, 2012). Following the procedure recommended in Vision 2000 (SEAOC, 1995), three performance levels were selected: immediate occupancy (IO), life safety (LS) and collapse prevention (CP).

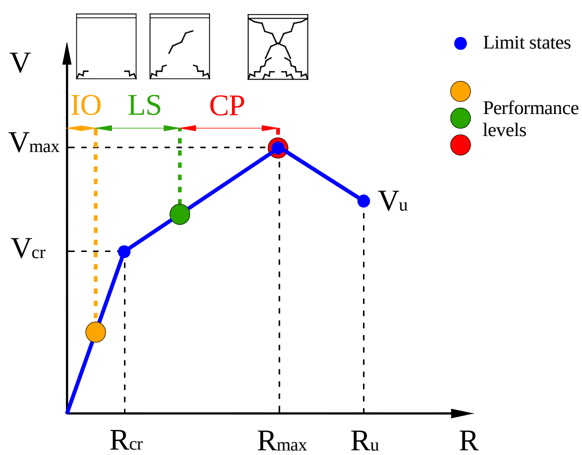

Figure 3. Performance levels and damage stages.

4. PERFORMANCE INDICATORS

For PBSD, a set of quantitative performance indicators should be specified to represent various damage levels for each structural system. As it was mentioned earlier, the behavior of each system analyzed will be judged by comparing relevant measures of performance to acceptance limits. Definition of comprehensive and realistic quantitative performance indicators that are associated to well-known damage stages has been the subject of research and discussion. Considering that PBSD has been mainly applied to systems failing under flexure, most studies include performance indicators for walls exhibiting a ductile failure mode. According to Ghobarah (2004), available story drift limits are conservative for ductile structures, but may be unsafe for non-ductile structures.

To evaluate seismic damage of concrete walls in low-rise housing, whose response is governed by shear deformations, performance indicators related to story drift ratio (R_{allow}) were used. For establishing the limiting values of selected performance levels, the following was considered: (a) for low-income population, housing unit is the main patrimony; therefore, housing rehabilitation should be economical and easily attainable, especially for IO and LS performance levels; (b) consistent with the latter, safety levels of walls failing in shear, associated to threshold values of performance indicators should be higher than those used for medium- and high-rise concrete buildings dominated by flexural deformations; and, (c) response measured during testing indicated that strength and stiffness degradation of concrete walls was rapid as soon as peak shear strength was reached; therefore, drift values close to those at peak shear strength should not be permitted for PBSD of low-cost housing.

Based on technical and economical facts, the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels for low-rise concrete housing are related to initial inclined web cracking, to extension of web inclined cracks to wall edges without penetration into boundary elements, and to wall peak shear strength, respectively. Expected damage for the three performance levels is described in more detail in Table 4. The prescribed damage stages roughly corresponded to design strengths of 25%, 75% and 100% of the peak shear strength, for IO, LS and CP performance levels, respectively. Note that CP performance level was defined to that at peak shear strength, that is, strength degradation was purposely excluded for the reasons explained above.

Typical residual crack patterns associated to performance levels of walls reinforced with 100% of the minimum web steel ratio prescribed by ACI-318 (ρ_{min}) and web shear reinforcement made of deformed bars, as well as of walls reinforced with 50% of ρ_{min} and web shear reinforcement made of welded-wire meshes are shown in Figs. 4 and 5, respectively. Performance indicators in terms of allowable story drift ratios (R_{allow}) are indicated in Figs. 4 and 5. Considering that some crack propagation was observed while maintaining the peak load during intervals of quasi-static testing protocol, the expected damage level and limiting values of residual cracking were described and defined exclusively from damage observed during shaking table testing of concrete walls.

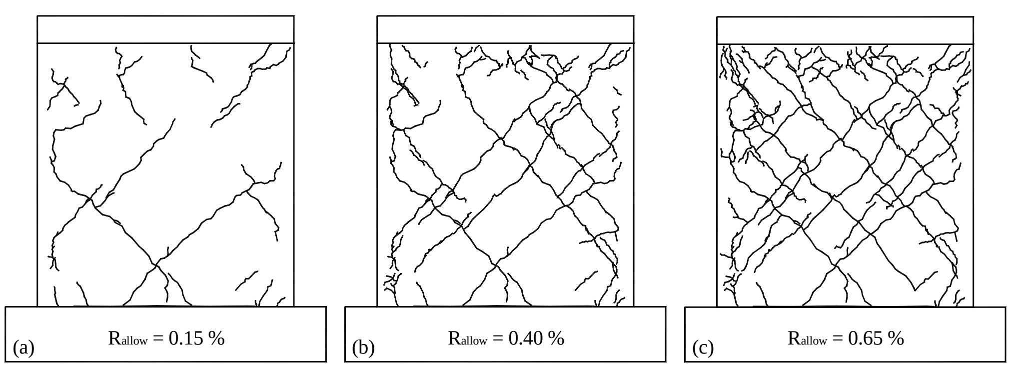

Figure 4. Residual cracking stage for walls reinforced with 100% of ρ_{min} and using deformed bars: (a) IO, (b) LS and (c) CP.

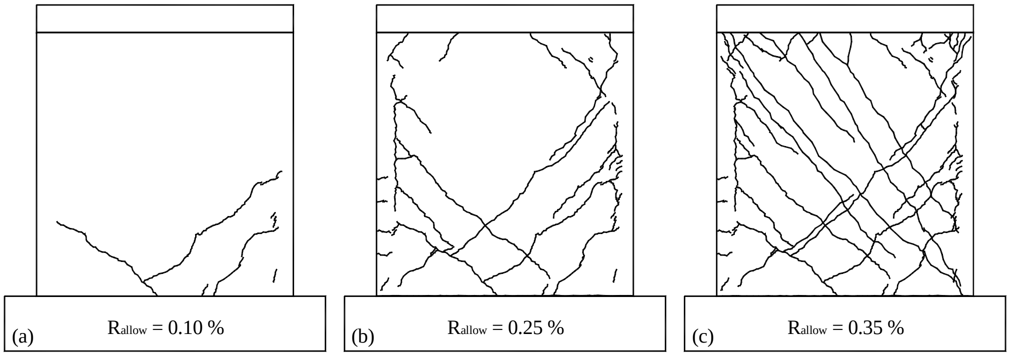

Figure 5. Residual cracking stage for walls reinforced with 50% of ρ_{min} and using welded-wire meshes: (a) IO, (b) LS and (c) CP.

Although the final damage level of walls (Fig. 2) was significantly larger than that associated to the CP performance level proposed in this study (Figs. 4 and 5), walls reinforced with 50% of ρ_{min} and web shear reinforcement made of welded-wire meshes were suitably rehabilitated using jacketing made of steel fiber reinforced concrete. In general terms, the observed and measured performance of rehabilitated walls was satisfactory because strength and displacement capacities measured in the original walls were adequately restored. Further information may be found elsewhere (Ávila *et al*., 2011).

4.1. Allowable story drift ratios

The damage sustained by a structure while it dissipates energy during an earthquake is a direct consequence of displacements in the inelastic range exhibited by the structure. Thus, performance indicators defined in terms of story drift ratios can be directly related to damage. Moreover, drift limits must be controlled to prevent damage to nonstructural components, to avoid structural instabilities, and to avoid human discomfort during frequently occurring low-level excitation (Bertero *et al.*, 1991). Similarly to findings reported by Ghobarah (2004), Duffey *et al*. (1994) stated that allowable story drift limits specified by most seismic codes are generally unconservative for concrete walls with low aspect ratio with characteristics similar to those investigated herein, because such code-based drift limits are likely more directed toward medium and high-rise structures than for low-rise shear walls.

It is desirable to define allowable story drift ratios based on experimental results. In this study, allowable drift ratios were determined from hysteresis curves measured during shaking table and quasi-static tests of reinforced concrete walls. Drift ratio is defined as the relative lateral displacement measured within an inter-story normalized by the story high. Typical measured hysteresis curves for walls with web shear reinforcement made of deformed bars and with web steel ratio equivalent to 50% and 100% of the minimum web steel ratio prescribed by ACI-318 (2011) are shown in Figs. 6 and 7, respectively. Similar results for walls with web shear reinforcement made of welded-wire meshes are presented in Fig. 8. In the graphs, limits proposed for IO, LS and CP performance levels are also indicated. Results for walls with h_w/l_w ratios equal to 0.5, 1.0 and 2.0 are included in these figures. The hysteresis curves were expressed in terms of the normalized shear strength (V/V_{normal}), shear stress (in the right-hand ordinate axis), and lateral drift ratio expressed in percentage. Shear strength predicted using equations proposed as a result of this program (Carrillo and Alcocer, 2012), V_{normal}, was utilized to normalize the measured lateral force, V. Calculated shear strength was computed using as-built wall dimensions and measured mechanical properties of materials (see Tables 1 and 2). The mode of failure is also indicated in the graph.

Analyzing trends for determining R_{allow}, it was evident that the type of web shear reinforcement and the h_w/l_w ratio (or M/Vl_w ratio) were the main factors affecting R_{allow}. For example, for walls with h_w/l_w = 2.0 it would be feasible to define story drift limiting values higher than those for walls with h_w/l_w = 0.5 or 1.0 (Figs. 6 to 8). However, for code purposes, it is unwise to propose R_{allow} values that depend on h_w/l_w or M/Vl_w ratios. This statement is based on the fact that, at the same story, all walls are subjected to practically the same value of story drift demand. With regards to the type of concrete used, significant differences among walls made of normalweight, lightweight and self-consolidating concrete were not observed.

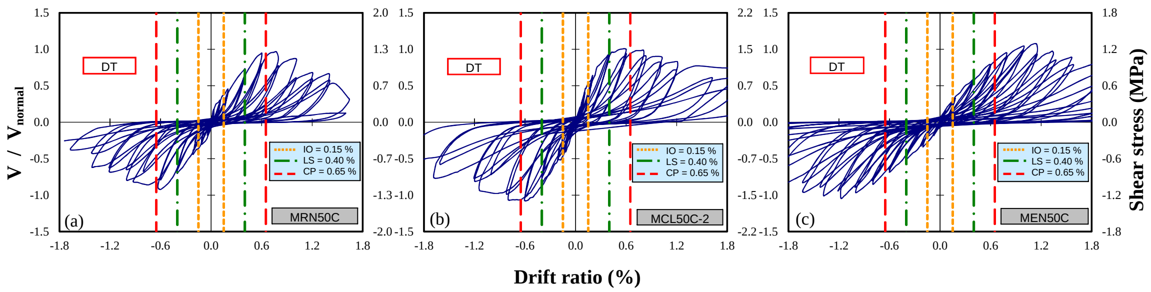

Figure 6. Hysteresis curves and allowable story drift ratios for walls reinforced with 50% of ρ_{min} and using deformed bars: (a) h_w/l_w = 0.5 (normalweight concrete), (b) h_w/l_w = 1.0 (lightweight concrete) and (c) h_w/l_w = 2.0 (normalweight concrete).

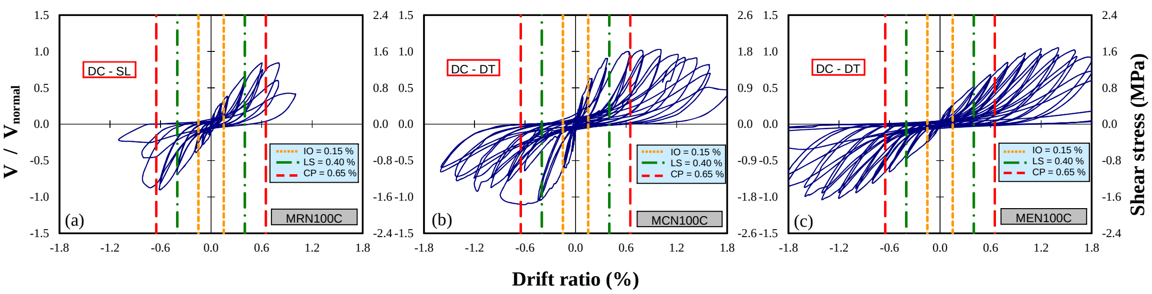

Figure 7. Hysteresis curves and allowable story drift ratios for walls reinforced with 100% of ρ_{min} and using deformed bars: (a) h_w/l_w = 0.5 (normalweight concrete), (b) h_w/l_w = 1.0 (normalweight concrete) and (c) h_w/l_w = 2.0 (normalweight concrete).

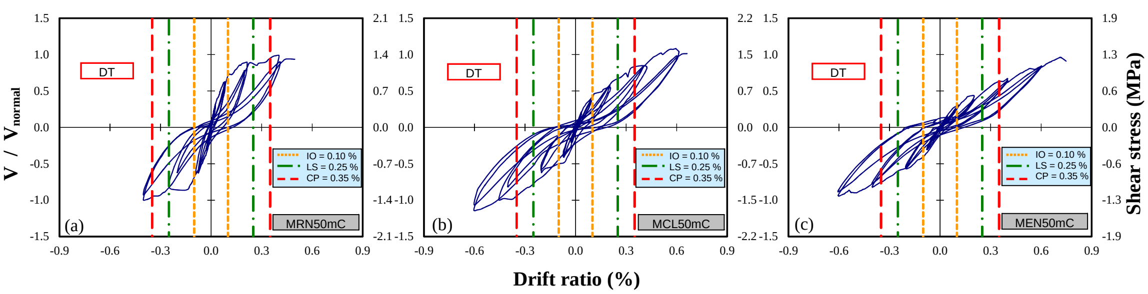

Figure 8. Hysteresis curves and allowable story drift ratios for walls reinforced with 50% of ρ_{min} and using welded-wire meshes: (a) h_w/l_w = 0.5 (normalweight concrete), (b) h_w/l_w = 1.0 (lightweight concrete) and (c) h_w/l_w = 2.0 (normalweight concrete).

Based on measured hysteresis curves, proposed allowable story drift limits and description of the expected damage for the three performance levels are presented in Table 4. As it was mentioned earlier, proposed values only depended on the type of web shear reinforcement. A single value for walls with web shear reinforcement made of deformed bars was established because differences in behavior between walls with 100% and 50% of ρ_{min} were not significant (Figs. 7 and 8). The corresponding damage stages of the walls are shown in Figs. 4 and 5.

Limiting values of allowable story drift ratio (R_{allow}) specified by seismic codes are not necessarily associated to the maximum capacity of structural elements or systems. These limiting values are typically associated to prescribed safety levels so that they can be used in practical structural engineering design. In this study, the safety level of an allowable drift ratio value for a particular performance level was calculated as the ratio between the measured drift capacity at a defined limit state and the measured drift ratio of the performance level. For evaluating the safety levels of allowable drift ratios for IO, LS and CP performance levels (R_{IO}, R_{LS} and R_{CP}), the drift ratios of selected limit states were distributed diagonal cracking, peak shear strength and failure of walls, respectively (R_{cr}, R_{max} and R_{uu}). Therefore, the safety levels of IO, LS and CP performance levels for low-rise concrete walls were defined as R_{cr}/R_{IO}, R_{max}/R_{LS} and R_{uu}/R_{CP}, respectively. Measured drift capacities at limit states (R_{cr}, R_{max} and R_{uu}) and measured drifts associated to different performance levels (R_{IO}, R_{LS} and R_{CP}) were determined from hysteresis curves measured during shaking table and quasi-static testing of wall specimens (Table 3). Mean values and coefficients of variation (mean–*CV*) of the safety levels of allowable drift ratios for IO, LS and CP performance levels of walls with web shear reinforcement made of deformed bars, welded-wire mesh and walls without web shear reinforcement are presented in Table 4.

Table 4. Proposed allowable story drift ratios of concrete walls for housing.

Performance level	Expected damage	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 Safety L. = 1.8 *CV* = 17.0 %	R_{allow} = 0.10 Safety L. = 2.3 *CV* = 27.2 %
LS	*Moderate damage:* Extension of web inclined cracks to the wall edges without penetration into the boundary elements.	R_{allow} = 0.40 Safety L. = 1.9 *CV* = 17.8 %	R_{allow} = 0.25 Safety L. = 2.1 *CV* = 24.6 %
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 Safety L. = 2.4 *CV* = 32.9 %	R_{allow} = 0.35 Safety L. = 1.3 *CV* = 24.8 %

It was considered that a safety level of 2 was appropriate for seismic design of reinforced walls. For unreinforced walls, a safety level of 6 was selected as a target value. Safety levels are dependent on the steel reinforcement ratio and characteristics of web shear reinforcement. For instance, safety levels were higher for walls with web shear reinforcement made of welded wire mesh than those for walls with deformed bars. An exception was observed for the CP performance level of walls with web shear reinforcement made of welded-wire mesh because displacement capacity at failure of these walls was almost equal to displacement capacity at peak shear strength. In all cases, variation of safety levels was high (around 20%). This can be explained by the fact that results from tests of walls having different h_w/l_w ratios were used.

4.2. Limitations of the proposed performance indicators

Performance indicators presented herein are applicable for the performance-based seismic design of walls with: a) M/Vl_w ratios less than or equal to 2.0 and walls with openings (door and window openings), b) thickness of boundary elements equal to web thickness, c) response governed by shear deformations, d) a specified concrete compressive strength, f_c', that varies between 15 and 25 MPa, e) axial stress less than 0.03 f_c', f) web steel ratio lower than or equal to 0.25%, g) web reinforcement made of deformed bars or welded-wire meshes, and h) same amounts of horizontal and vertical web reinforcement. Such limits are also applicable to walls made of normalweight ($19 \le \gamma \le 21$ kN/m^3), lightweight ($15 \le \gamma \le 19$ kN/m^3) and self-consolidating ($19 \le \gamma \le 21$ kN/m^3) concretes.

5. CONCLUSIONS

Based on data analysis of the experimental program reported herein, the following conclusions can be withdrawn:

- Within a performance-based seismic design framework, and due to the lack of acceptance limits for shear-dominated walls, a series of performance indicators were developed.
- Development was based on an extensive experimental and analytical research program. Prototype housing is prevalent in several countries in Latin America. Experimental variables comprised different concrete types (normalweight, lightweight, self-compacted), distinct web reinforcement ratios and type of steel reinforcement (deformed bars and welded-wire meshes), wall geometry, and the presence of wall openings.
- Selected performance indicators were story drift ratios (R_{allow}). These limits were developed for immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels.
- Proposed values for performance indicators are summarized in Table 4, and are readily applicable in structural design of low-rise housing made of walls with the characteristics discussed in the paper.
- Because of the inherent brittle nature of the mode of failure of walls tested, performance indicators were purposely developed to be conservative, i.e. limits are smaller than those measured at peak strength. These values provide an adequate level of safety against rapid deterioration of stiffness and strength after reaching peak lateral load capacity.

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