Shear Displacements as Function of Ductility Levels for Squat Reinforced Concrete Walls

A.L. Vidot-Vega
University of Puerto Rico at Mayagüez

SUMMARY:
Squat RC walls (aspect ratios less than 2.0) are frequently used in building structures to provide lateral stiffness. Different types of failures have been observed in past experimental tests of squat RC walls depending on the levels of reinforcement ratios, the level of inelastic action and design philosophy. According to previous studies shear displacements on these structural systems can contribute as much as 80% of the total displacements. This paper presents a study of the shear displacements of squat RC walls as function of the displacement ductilities. Existing data from experimental tests have been reviewed together with existing simplified methods to determine the shear displacements in RC elements. It was found that some of the existing methods underestimate or overestimate the shear deformations, but the agreement is better for walls with aspect ratios higher than 0.50. The paper provides recommendations for future research in this topic.

Keywords: shear displacements, squat walls, reinforced concrete, ductility

1. INTRODUCTION

Reinforced concrete (RC) walls with aspect ratios (height to length) are important structural elements used to support buildings against earthquake actions and winds. In regions of moderate and high seismicity these elements are typically designed and detailed according to capacity-design principles. Structural walls play an important role in a building's resistance to seismic lateral forces. Some RC walls have experienced significant damage while others have performed well in past and recent earthquakes depending on factors such as the magnitude of the earthquake, location and age of the building, soil type, geometry and reinforcement detail of the wall, and others. For example, the performance of RC wall buildings during the 2010 Chilean earthquake varied from operational through collapse (Boroschek et al 2010). These observations open some questions about the adequacy of seismic design approaches and modern building codes and about the adequacy of the performance levels currently used for design and assessment. For RC walls the shear displacements become important as the aspect ratio is reduced and cannot be neglected as usually is done for flexural dominated members. Accuracy in the calculations of all displacements components is necessary to guarantee an accurate estimation of the displacement capacity of these structural systems.

Many researchers have tried to better understand the behaviour of RC walls and have developed models to predict their load-displacement behaviour, failure modes and damage levels (e.g. Lefas et al. 1990, Pilakoutas and Elnashai, 1995a and 1995b, Salonikios et al. 2010, Palermo and Vecchio, 2002). While all of these researches have provided us with better understanding of the response and failure modes of low to medium-rise RC walls, there is still work that needs to be done in the prediction of the displacement capacity of these systems. Simplified models are available that allow engineers to calculate the different displacement components of RC walls that can be used for preliminary calculations and analyses. This paper examines two of these simplified models to evaluate the accuracy of the results compared against data available from experimental tests and to evaluate future research needs in this area. The program Response2000 was also selected to perform sectional and member analyses of several RC walls to have additional comparisons.

2. DATA REVIEW OF EXISTING EXPERIMENTAL TESTS OF SQUAT RC WALLS

A summary of some of the most significant tests found in the literature about RC walls with aspect ratios less and equal than 2.0 is accomplished in this section. These tests included RC walls subjected to static cyclic and monotonic loading with and without axial load. Findings relating flexural and shear deformations, strength and stiffness degradation, material strains and types of failure are highlighted.

Lefas et al (1990) tested 13 RC rectangular walls with aspect ratios of 1 and 2 subjected to monotonic loading. They studied the effects of axial load, horizontal reinforcement, aspect ratio and concrete strength on the wall behaviour and failure. A compression zone failure was observed in all the walls. The presence of axial load delayed the occurrence of such failure. As the aspect ratio decreases the failure zone became more extensive. They measured strains at the longitudinal bars with strain gages and found significant post –yield deformations prior to wall failure. They determined that the stiffness is enhanced by axial load and as the axial load increases the increase in stiffness is less pronounced.

Salonikios et al (1999 and 2000) tested 11 RC walls with aspect ratios of 1.0 and 1.5 and different reinforcement orientations (traditional and diagonal) subject to static cyclic loading and constant axial load in some cases. The walls experienced sliding deformations. Later Salonikios (2002, 2004 and 2007) studied in detail the displacement components (flexural, web shear, sliding shear), displacement and curvature ductilities, and plastic hinge lengths of these walls. The shear deformation components (sliding + web shear) contributed between 20 to 80 % to the total wall deformation and usually increased with the ductility level. Sliding shear deformations increases substantially for displacement ductilities above 2.5. He formulated predictive equations to determine the deformations due to web and sliding shear based on theory of truss analogy, mechanics of materials and empirical calibrations.

Hidalgo et al (1996, 2002) tested a total of 26 RC walls in double bending subject to static cyclic loading. The walls failed in shear by diagonal tension failure and have aspect ratios from 0.35 to 1.0. They found that deformation capacity of the walls decreases as the aspect ratio decreases. Important findings from these tests were that the normalized dissipated energy remains fairly constant and that the strength deterioration of wall specimens increased with decreasing values of the aspect ratio and of both horizontal and vertical reinforcement.

Hun et al (2002) tested four RC walls with and without boundary elements subjected to cyclic loading. The specimens did not fail in premature shear; they behaved more in a flexural manner. Shear deformation components account for 18% to 34% of the total deformations measured during testing.

Greifenhagen and Lestuzzi (2005) tested four lightly reinforced RC shear walls with aspect ratios of 0.69 for which the horizontal reinforcement, axial force ratio, and concrete compressive strength were varied. All specimens developed the nominal flexural strength and the maximum base shear was controlled by flexure, not by premature shear failure. Shear deformation components contribute between 20 to 47% (average 30%) to the total deformation depending on the ductility level. The sliding shear component increases as the ductility level increases. They did not compute the shear deformation components from diagonal LVDTs as is usually done; they instead used horizontal displacement transducers and flexural deformations to obtain the shear components.

Massone (2006) performed an experimental program that involved testing of 14 wall pier (WP) and spandrel specimens (WS) with shear span-depth ratio of 0.5 and 0.44, respectively. The RC walls were tested in double bending and subject to static cyclic loading and different axial load ratios. The failure was controlled by shear. They proposed a model that accounted for the flexural-shear interaction to predict the inelastic response of reinforced concrete squat walls. The shear deformation components contribute from 27 to 85% to the total deformations depending on the level of ductility. In general shear deformation components increased as ductility level also increased.

Carrillo and Alcocer (2008) performed an experimental program in which they tested a variety of squat RC walls subject to static cyclic loading representing typical housing construction in Mexico. Diagonal tension and compression shear failures were observed in the walls with 50% and 100% of the reinforcement established by the code, respectively. Shear displacement components represented 52 to 82% of the total displacement.

Kuang and Ho (2008) tested 8 large-scale non-seismically detailed, squat reinforced concrete shear walls with aspect ratios of 1.0 and 1.5 and with and without boundary confinement. A displacement ductility factor of 2.5 to 3 was achieved for the walls without boundary confinement and of 4.5 to 5 for walls with boundary elements. Shear displacements accounted for 30 to 60% of the total displacements.

In summary, different types of failures (flexural, shear-diagonal tension, shear-diagonal compression) were observed in these tests depending on the level of horizontal and vertical reinforcement ratio, the level of inelastic action and design philosophy. Table 2.1 shows a summary of important variables for all tests (compressive strength (f_c'), steel strength (f_y), transverse steel reinforcing spacing (s), horizontal and vertical reinforcement ratio (ρ_h, ρ_v), axial load ratio, web thickness (t_w), aspect ratio (h_w/l_w), ultimate displacement (δ_u), ultimate shear strength (V_u), and displacement ductility (μ). The displacement ductility on Table 2.1 was reported by the authors of the different tests or in some cases was calculated using the displacement when the strength decays by 20% over the wall yield displacement. The contribution of shear displacements varied between tests, in general increasing with the ductility level (Fig. 2.1-2.2). Shear deformations from Greifenhagen and Lestuzzi (2005) deviates a little from the pattern of other tests. Little data regarding material (steel and concrete) strains and stiffness as a function of ductility level were found. Figure 2.2 shows the data from all the RC wall tests studied in this paper (Table 2.1 excluding the Hidalgo et al (2002) tests) as function of ductility levels and aspect ratios (height to length). The shear displacement ratios varied for 18 to 85%. In general, larger shear displacements were obtained for walls with smaller aspect ratios and exhibiting failure controlled by shear in diagonal tension and compression.

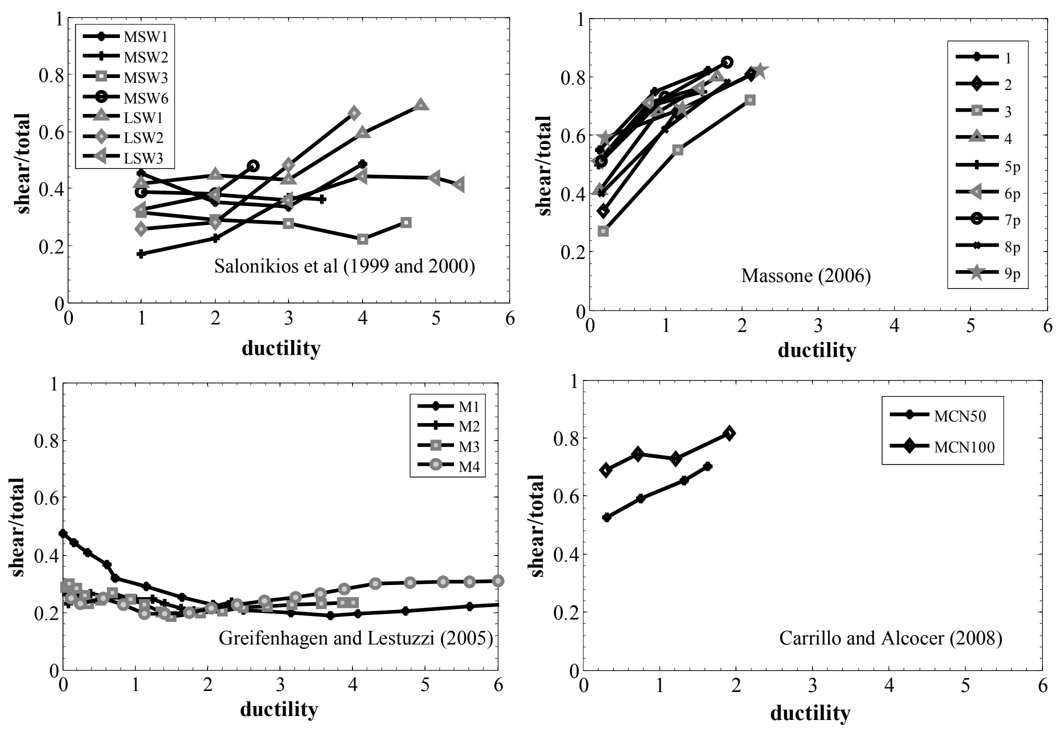

Figure 2.1. Shear to total displacement ratio as function of displacement ductilities

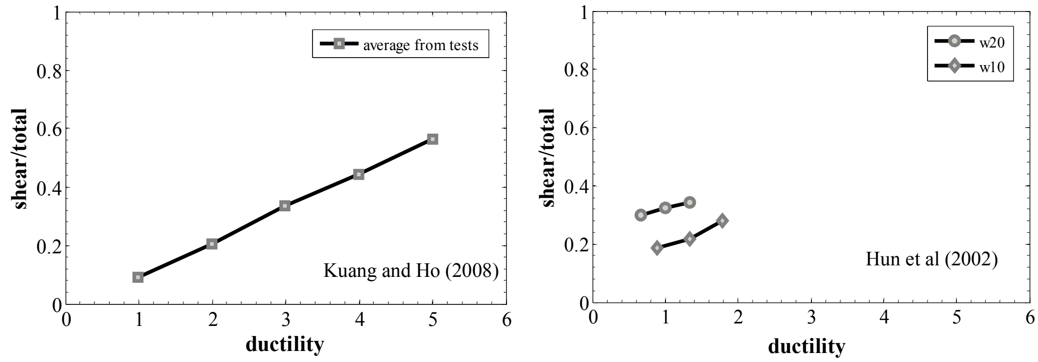

Figure 2.1. (cont.) Shear to total displacement ratio as function of displacement ductilities

Table 2.1. Static Cyclic Tests Summary

Author (s)	Name/ No.	fc' MPa	f_{yh} MPa	s (mm)	ρ_h (%)	ρ_v (%)	axial load ratio	t_w (mm)	h_w/l_w	d_b (mm)	V_u (kN)	μ
Salonikios et al, 1999	MSW1	26.10	610	49	0.57	0.56	0.00	100	1.50	20.50	195	4.00
	MSW2	26.20	610	100	0.28	0.28	0.00	100	1.50	26.00	124	3.40
	MSW3	24.10	610	100	0.28	0.28	0.07	100	1.50	20.10	173	4.65
	MSW6	27.50	610	49	0.57	0.56	0.00	100	1.50	19.98	188	4.00
	LSW1	22.20	610	49	0.57	0.56	0.00	100	1.00	10.80	262	4.64
	LSW2	21.60	610	100	0.28	0.28	0.00	100	1.00	10.40	186	3.90
	LSW3	23.90	610	100	0.28	0.28	0.07	100	1.00	16.30	250	5.34
Hidalgo et al, 2002	1	19.40	392	180	0.13	0.25	0.00	120	1.00	13.12	68	4.78
	2	19.60	402	170	0.25	0.25	0.00	120	1.00	15.08	124	5.36
	4	19.50	402	110	0.38	0.25	0.00	120	1.00	15.09	185	5.36
	6	17.70	314	180	0.13	0.26	0.00	120	0.69	7.95	63	5.81
	8	15.70	471	170	0.25	0.26	0.00	120	0.69	9.95	169	4.85
	9	17.60	367	110	0.26	0.26	0.00	100	0.69	9.76	116	6.11
	10	16.40	367	140	0.25	0.25	0.00	80	0.69	8.35	95	5.22
	11	16.30	362	220	0.13	0.25	0.00	100	0.50	4.85	63	5.47
	12	17.00	367	110	0.26	0.13	0.00	100	0.50	6.95	127	7.74
	13	18.10	370	110	0.26	0.25	0.00	100	0.50	4.96	128	5.47
	16	18.80	367	140	0.25	0.25	0.00	80	0.35	4.48	95	8.25
	7	17.80	471	170	0.25	0.26	0.00	120	0.69	11.27	169	5.49
	14	17.10	367	280	0.13	0.16	0.00	80	0.35	3.05	53	5.61
Lefas et al, 1990	SW11	52.30	520	80	1.10	2.40	0.00	70	1.00	8.25	260	2.30
	SW12	53.60	520	80	1.10	2.40	0.10	70	1.00	8.86	340	3.06
	SW13	40.60	520	80	1.10	2.40	0.20	70	1.00	8.88	330	2.32
	SW14	42.10	520	80	1.10	2.40	0.00	70	1.00	11.21	265	2.87
	SW15	43.30	520	80	1.10	2.40	0.10	70	1.00	8.05	320	2.79
	SW16	51.70	520	80	1.10	2.40	0.20	70	1.00	5.78	355	2.31
	SW17	48.30	520	80	0.37	2.40	0.00	70	1.00	10.75	247	2.76
Greifenhagen & Lestuzzi, 2005	M1	50.70	504	175	0.30	0.30	0.03	100	0.69	5.00	204	5.60
	M2	51.00	504	175	0.00	0.30	0.03	100	0.69	12.13	203	5.90
	M3	20.10	745	122	0.30	0.30	0.10	80	0.77	7.07	176	5.80
	M4	24.40	745	122	0.30	0.30	0.05	80	0.77	9.00	135	8.00
Massone, 2006	WS-T1-S1	25.50	424	330	0.28	0.43	0.00	152	0.50	8.51	634	2.94
	WS-T2-S1	31.40	424	330	0.28	0.40	0.00	152	0.50	13.50	454	6.26
	WS-T2-S2	31.00	424	330	0.28	0.40	0.00	152	0.50	6.26	492	2.25
	WS-T1-S2	43.70	424	330	0.28	0.40	0.00	152	0.50	14.10	752	6.53
	WS-T5-N10-S1	28.30	424	305	0.28	0.23	0.10	152	0.44	6.00	754	2.76
	WS-T5-N10-S2	31.40	424	305	0.28	0.23	0.10	152	0.44	7.00	821	4.33
	WS-T5-N5-S1	31.90	424	305	0.28	0.23	0.05	152	0.44	7.80	650	2.74
	WS-T5-N5-S2	32.00	424	305	0.28	0.23	0.05	152	0.44	6.50	683	3.00
	WS-T5-N0-S1	29.90	424	305	0.28	0.23	0.00	152	0.44	8.00	405	2.94
Carrillo & Alcocer, 2003	MCN50mD-36	24.78	632	150	0.11	0.11	0.02	80	1.00	11.14	329	1.64
	MCN100D-37	24.78	436	320	0.28	0.28	0.02	80	1.00	30.34	471	4.88
Hun et al, 2002	w20	34.20	342	225	0.28	0.32	0.10	200	2.00	80.90	386	6.52
	W10	36.20	342	175	0.28	0.32	0.10	200	2.00	85.90	443	9.24
Kuang and Ho, 2008	U1.0	30.40	520	150	1.05	0.92	0.10	100	1.00	11.78	737	3.10
	U1.5	34.90	520	150	1.05	0.92	0.07	100	1.50	14.00	689	2.80
	C1.0	35.20	520	150	1.05	1.05	0.11	100	1.00	14.88	718	3.10
	C1.5	34.20	520	150	1.05	1.05	0.07	100	1.50	15.86	681	2.60
AVERAGE		29.59	481	169	0.44	0.66	0.04	109.20	0.89	14.09	328	4.52

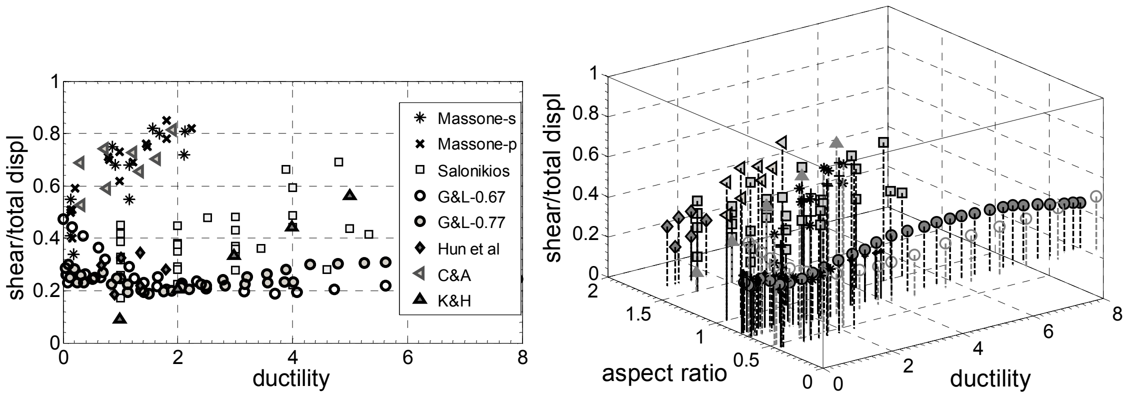

Figure 2.2. Shear to total displacements ratio for all tests given on Table 1 as function of ductility and aspect ratio

3. SIMPLIFIED METHODS REVIEWED

Two methods are reviewed in this paper to obtain shear displacement components and also shear forces which are used without the need to perform a detailed finite element model of the wall.

3.1. First methodology:

The first method to obtain shear deformations is the one outlined by Priestley et al. (2007) or by Miranda et al. (2005) with some modifications performed by Krolicki et al (2011) to account for RC walls failing in pre-emptive shear. The calculations of the shear strength are primarily based on the UCSD shear model by Kowalsky and Priestley (2000) with also the modifications presented in Krolicki et al (2011). The total force-deformation response curve is a combination of an idealized flexure response curve and an idealized shear response curve at different member limit states. The basic methodology and equations are described briefly next. Prior to flexural cracking the shear displacements (Eqn. 3.0) are found by dividing the force when flexural cracking occurs to the elastic stiffness (K_{se}) given by Eqn. 3.1. At the onset of shear cracking and after flexural cracking occurs the shear displacements can be obtained by using a reduced stiffness (K_{eff}, Eqn. *3.2*) and subtracting the contribution of the concrete strength (V_{cr}) at cracking (Eqn. 3.3). After shear cracking and up to flexural yield strength the displacements are found in function of a cracking stiffness ($K_{s,cr}$) which is derived from the truss analogy theory (Paulay 1975 and discussed later elsewhere). Eqn (3.4) it is a simplified version of the equation that is obtained from truss analogy assuming that the cracking angle is 45 degrees and the section have vertical stirrups. In these equations, G is the shear modulus, A_s is the shear area, H is the length of the wall, E is the concrete modulus of elasticity, E_s the modulus of elasticity of the steel, ρ_a is the reinforcement ratio, b_w is the width of wall, t_w is the thickness, I_{eff} and I_{gross} are the effective and gross moment of inertia, respectively. In addition, F_y is the yield steel strength, $V_{c,sc}$ is the concrete contribution to shear strength after shear cracking $\Delta_{f,y}$ is the flexural yield displacement, Δ_f is flexural displacement after yielding. The different phases are summarized in Fig.2. If the wall is expected to be dominated by flexural failure, the yield strength and yield displacement can be then substituted with their nominal counterparts.

Prior to flexural cracking:

$$K_{se} = \frac{GA_s}{H} \tag{3.0}$$

$$\Delta_{se} = \frac{F_{cr}}{K_{se}} \tag{3.1}$$

At the onset of shear cracking (after flexural cracking):

$$K_{eff} = \frac{GA_s}{H}\frac{EI_{eff}}{EI_{gross}} \tag{3.2}$$

$$\Delta_{sl} = \frac{V_{cr} - F_{cr}}{K_{eff}} \tag{3.3}$$

After shear cracking to flexural yield:

$$K_{s,cr} = \frac{0.25\rho_a}{0.25+10\rho_a} E_s b_w t_w \tag{3.4}$$

$$\Delta_{s,sc} = \frac{F_y - V_{c,sc}}{K_{s,cr}} \tag{3.5}$$

After flexural yield:

$$\Delta_s = \Delta_f \left(\frac{\Delta_{s,sc}}{\Delta_{f,y}}\right) \tag{3.6}$$

3.1. Second methodology:

The second method reviewed in this paper is the one developed by Salonikios (2007) based on experimental data from tests of RC walls with aspect ratios of 1.0 and 1.5 by Salonikios et al (1999 and 2000). He proposed several equations to calculate shear displacements that include the calculation of the shear cracking and sliding components that depends if the wall has conventional or diagonal reinforcement and if the effects of axial loads are included. The shear displacements due to diagonal shear cracking are calculated using Eqn 3.7 which is also developed from truss analogy theory including only the contribution of conventional web reinforcement. The displacements due to sliding are obtained by Eqn. 3.8 using a reduced modulus of elasticity that was calibrated against the experimental data proposing empirical equations to obtain the constant *Y*. An example of one of these equations it is shown in Eqn. 3.9 which depends on the total displacement (Δ) of the wall. This requires an interactive procedure to obtain Y which is described in the aforementioned references. The shear displacements can then added to the calculated flexural displacements to obtain the total displacement capacity of the wall.

$$\Delta_{cr,sh} = \frac{d}{2E_s}\left(\frac{V_{sh}}{A_h}\right)\frac{a_s^2}{2.25} \tag{3.7}$$

$$\Delta_{sl} = \frac{VH}{YEb_w t_w} \tag{3.8}$$

$$Y = -0.0004916\Delta^3 + 0.0153725\Delta^2 - 0.1635911\Delta + 0.6 \tag{3.9}$$

4. ANALYSIS OF RESULTS FROM SIMPLIED MODELS

Two groups of experimental tests of squat RC walls were selected as study cases from the Table 2.1. The tests performed by Massone (2006) and Salonikios et al (1999) were chosen since they have detailed data from different displacement components. The RC walls tested by Massone (2006) have aspect ratios from 0.44 to 0.55 and were tested as double bending. The tests from Salonikios et al (1999) were single bending walls with aspect ratios from 1.0 to 1.5. Test 5 and LSW3 have axial loads of 10% and 7%, respectively. The other two tests have no axial load. The analytical responses were obtained by two methods: (1) Krolicki et al (2011)-USCD-K and (2) Salonikios (2007)–S as described in section 3. The USCD-K results were obtained only for four points of response according to Fig. 4.1 (F_{cr}, V_{cr}, F_y, F_u). For S method the experimental forces were used in Eqn. 3.8. Fig. 4.2 shows the analytical and experimental (solid) force vs. displacement responses of these tests. It is also included the analytical shear strength enveloped predicted by the first method. From these figures it can be noted that for the walls with axial loads the shear envelope and analytical responses predicted by USCD-K are better that for the walls with no axial load. In general the S method predicts the response with good agreement until that the wall strength begins to deteriorate. The analytical (dashed lines) and experimental (solid line) shear to total displacement ratios are presented in Fig. 4.3 for the same tests. The analytical ratios with the two methods were obtained for the same four points around the force-displacement curve (Fig.4.1). Depending of the type of failure experienced by the walls, all of the points along the curve may not be reached. The experimental results show a tendency for the shear to displacement ratio to increase as ductility increases. However, the results obtained with the analytical models are quite variable. Generally, the ratios obtained from the models increases until the wall reached yielding and then they become almost constant. In terms of the ratio predicted the S method give much closer results with the experimental data.

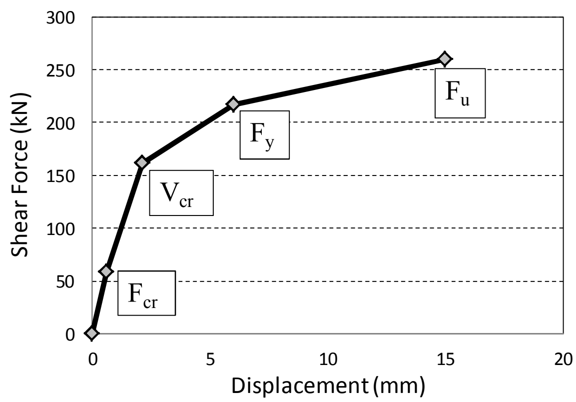

Figure 4.1 Example of force vs. displacement response

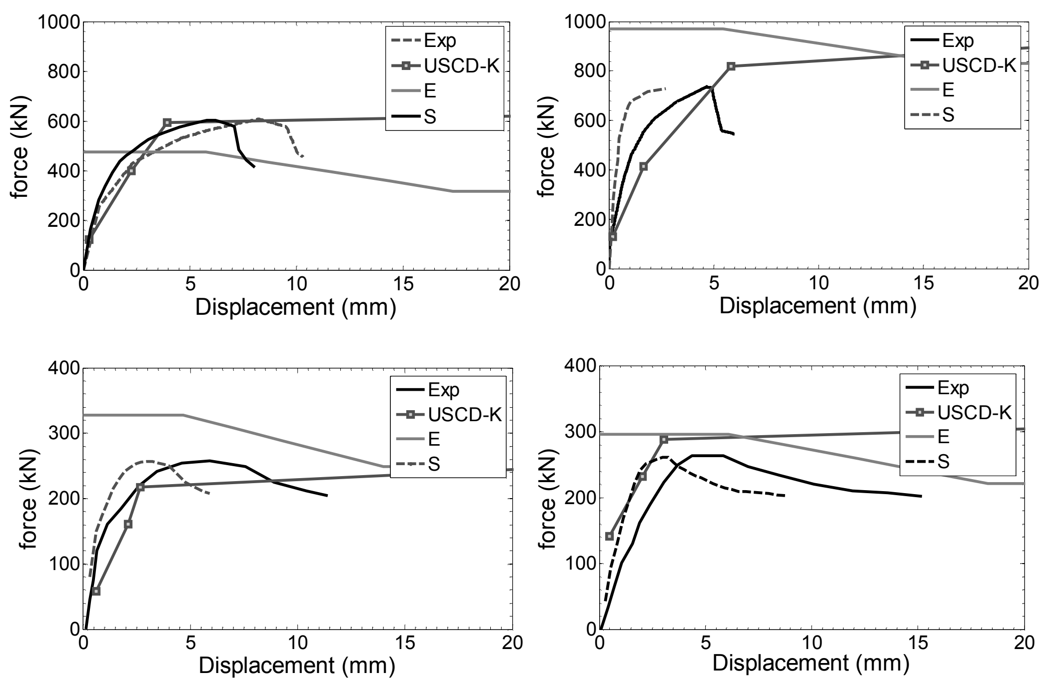

Figure 4.2 Force vs. displacement responses (a)-(b) Test 1 & Test 5 from Massone (2006), (c)-(d) LSW1 & LSW3 tests from Salonikios et al (1999)

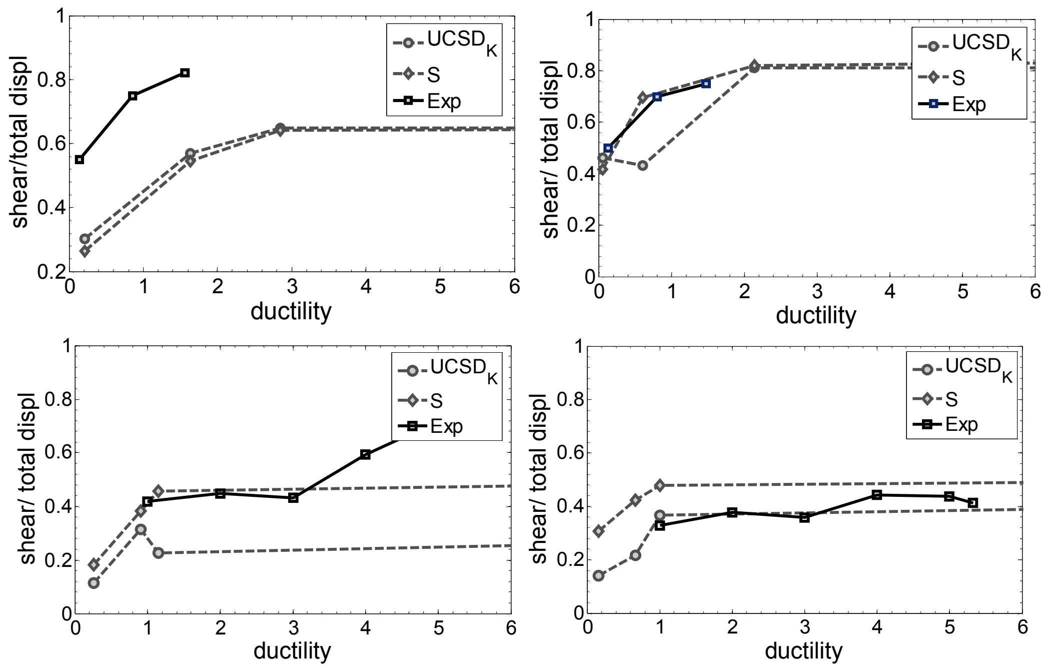

Figure 4.3. Shear to total displacement ratio vs. ductility (a)-(b) Test 1 & Test 5 from Massone (2006), and (c)-(d) LSW1 & LSW3 tests from Salonikios et al (1999)

5. USE OF RESPONSE 2000 AS TOOL FOR PRELIMINARY ANALYSIS OF RC WALLS

In this section it is presented some of the results obtained with the program Response 2000 developed at the University of Toronto by Prof. Evan Bentz in a project supervised by Prof. Michael P. Collins. Response 2000 is a sectional analysis program that can calculate the strength and ductility of a reinforced concrete cross-section subjected to shear, moment, and axial load. The program finds the full load-deformation response of a RC section using the modified compression field theory (Collins 1978, Bentz 2000). The program can also perform member analysis in which shear strains along the member, shear forces, and total displacements can also be obtained. A flexural analysis was performed in order to obtain the displacement due to flexure for several walls. These displacements were subtracted from the total displacements obtained with Response 2000 to determine the shear components.

The analytical and experimental force vs. displacement responses are shown for walls LSW1 and LSW3 (Table 2.1) in Fig. 5.1 and for walls MSW1 and MSW3 in Fig. 5.2. These walls were tested by Salonikios et al (1999). Walls LSW1-MSW1 and LSW3-MSW3 have 0% and 7% of axial load, respectively. The responses obtained with Response 2000 are denoted with the letter R. It can be observed that Response2000 which is based on the modified compression field theory can predict reasonably well the responses for walls with no axial load; for walls with axial load the program predicts a more rigid behavior that can perhaps be improved using other constitutive models for the concrete and steel. In terms of shear displacement ratios there are some variability in the results which are shown in Fig. 5.3 only for walls LSW1 and LSW3. For the wall LSW3 which have axial load, Response 2000 predicts higher shear to total displacement ratios and they are almost constant as displacement ductility increases. For wall LSW1 with no axial loads this program underestimates the shear to total displacement ratio as displacement ductility increases and have an opposed trend with the experimental results.

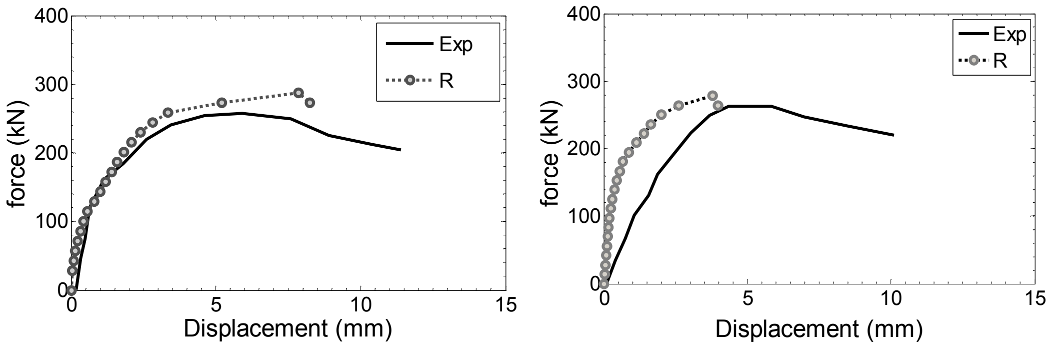

Figure 5.1. Force vs. displacement responses (a)-(b) LSW1 & LSW3 tests from Salonikios et al (1999) with results from Response2000

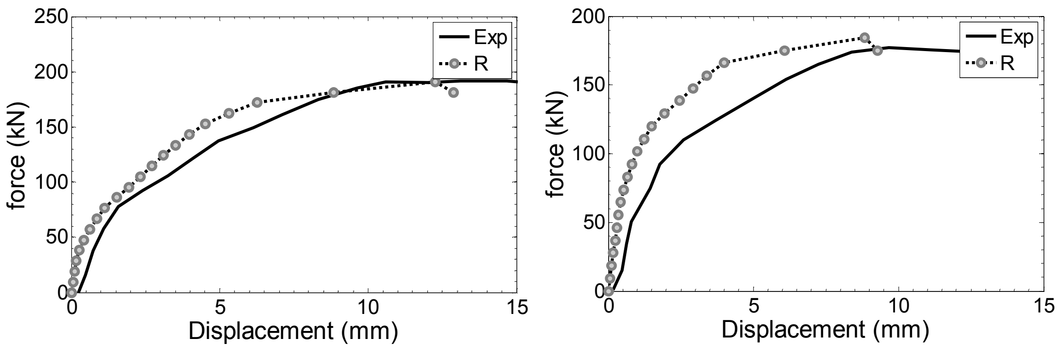

Figure 5.2. Force vs. displacement responses (a)-(b) MSW1 & MSW3 tests from Salonikios et al (1999) with results from Response2000

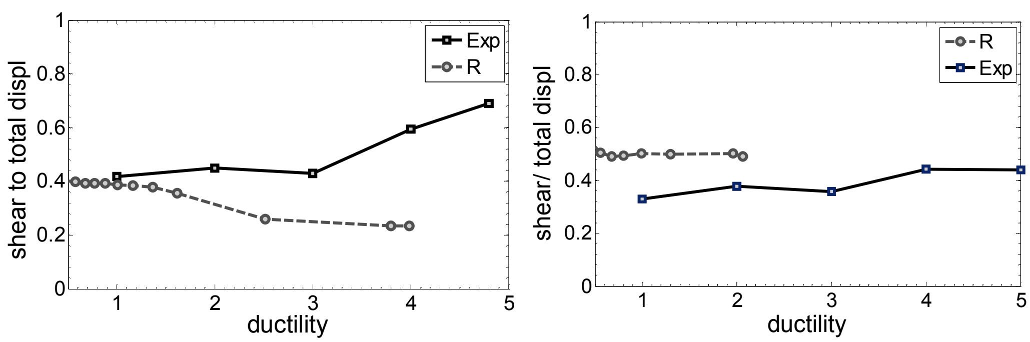

Figure 5.3. Shear to total displacement ratio vs. ductility (a)-(b) LSW1 & LSW3 tests from Salonikios et al (1999)

5. CONCLUSIONS

Two simplified methods were reviewed that allow the rapid calculation of shear displacement components and shear strength capacities. The shear displacements obtained with the equations proposed by Salonikios (2007) are in better agreement with the experimental data of the walls studied in this paper when compared to the modified procedure by Krolicki et al (2011).The drawback from the Salonikios equations is that they required an interactive procedure to obtain the reduced modulus of elasticity constant (Y) since the equation depend of the total displacement capacity. When experimental tests are not available this becomes a great disadvantage. The results obtained with the

USCD-K method for the shear strength and displacement response of the walls studied in this paper are quite variable, but the agreement is better for walls with aspect ratios higher than 0.50. For walls not failing in pre-emptive shear as the ones tested by Salonikios is better to use nominal yield instead of first yield in the USCD-K model to obtain the total displacement capacity. The program Response 2000 which is based on the compression field theory was also evaluated in this study to determine its capability to predict the shear displacements and force vs. displacement responses of squat RC walls. It was found that in some cases this program underestimated the shear displacements while in other cases they were overestimated. For future work, it is recommended to perform more experimental tests on squat RC walls to develop models that allow the calculation of shear displacements when the strengths of the walls begin to deteriorate. The results from these tests can also be used to have better damage limits states and displacement capacity prediction equations to be used in the engineering practice for preliminary assessment.

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