

STRENGTH AND DEFORMATION CAPACITY OF SHEAR WALLS

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Abstract

A research program was conducted to investigate the relationship between shear strength and deformation capacity of reinforced concrete structural walls. The program included testing of eight large-scale reinforced concrete structural walls subjected to constant axial load and reversed cyclic lateral loading. Primary test variables included wall height-to-length (aspect) ratio (1.0, 1.5 and 2.0), axial load level (0.025Agf'c, 0.10Agf'c, 0.15Agf'c, and 0.20Agf'c), and wall shear stress level (between $4\sqrt{f_c}$ and $8\sqrt{f_c}$). In addition, a comprehensive database for well-detailed walls was developed to assess the influence of wall curvature ductility and plastic rotation on shear strength. Results from the study provide valuable information for modeling structural wall behavior.

Keywords: reinforced concrete; shear wall; strength; deformation; ductility



1. Introduction

Reinforced concrete structural walls are one of the most commonly used structural elements in resisting wind and earthquake loads. Although a significant number of tests and studies have been conducted for shear walls, additional research is needed on certain topics, such as wall shear strength and deformation capacity. Current ACI Code provisions for assessing the strength of walls are relatively unsophisticated. The equation used to compute the nominal shear strength of a reinforced concrete wall has remained effectively unchanged between the 1983 Code and the 2014 Code. Recent studies have shown that wall ductility demands have significant impact on wall shear strength [1]. In addition, in the majority of prior tests, accurate and direct measurements of important response quantities such as the relative contributions of flexure, shear, and anchorage to lateral displacement, and the variation of these response quantities over the length and height of the test specimen, were not collected.

A research program, including both analytical and experimental components, was conducted to fill some of the identified knowledge gaps. The test program consisted of eight large-scale reinforced concrete shear wall specimens designed such that nonlinear shear deformations were expected to contribute substantially to total displacement response. The wall specimens were heavily instrumented to obtain detailed response information and to provide data for development and validation of analytical models. In addition, a comprehensive wall database has been constructed to include nearly sixty wall tests with well-detailed boundaries and constructed with medium or high ductility reinforcing steel.

2. Experimental Program

2.1 Wall details

Eight large-scale reinforced concrete wall specimens, subjected to a constant axial load and a single reversed cyclic lateral load at the top, were tested. The specimens were heavily instrumented to obtain detailed response information. These specimens were 15.2cm thick, 1.22m long, and either 1.22m, 1.83m, or 2.44m tall. Primary test variables included wall height-to-length (aspect) ratio (1.0, 1.5 and 2.0), axial load level ($0.025A_{g}f_{c}$, $0.10A_{g}f_{c}$, $0.15A_{g}f_{c}$, and $0.20A_{g}f_{c}$), and wall shear stress level (between $4\sqrt{f_{c}}$ and $8\sqrt{f_{c}}$). The ratio of longitudinal boundary reinforcement varied between 3.2% and 7.1%. Vertical and horizontal web reinforcement ratios were identical and they ranged from 0.27% to 0.73%, which exceeded the minimum requirement of 0.25% from ACI 318-14 S.18.10.2.1 [2]. Transverse reinforcement at the wall boundaries satisfied ACI 318-14 S18.10.6.4 requirements for special structural walls. Key attributes of all wall specimens are summarized in Table 1. The naming convention used for each wall reflects important test variables. For example, RW-A20-P10-S38 indicates a wall with an aspect ratio of 2.0 (A20), an axial load ratio of 0.10A_gf'_c (P10) and a shear stress level of $3.8\sqrt{f_{c}}$.

The first five specimens were designed to yield in flexure prior to strength loss, with the level of shear stress at flexural yield as a primary variable. The ratio of the lateral load at nominal moment capacity to the nominal shear strength determined from ACI 318-14 S18.10.4, $(V@M_n)/V_n$, computed using specified material properties, varied from approximately 0.8 to 0.9, which is very close to the design limit (for $\phi = 1.0$). This ratio was reduced to 0.6 for the sixth wall and was increased up to nearly 2.0 for the seventh and eighth walls to assure a shear dominant failure for the last two specimens.



Test	Specimen code	h/l	$\rho_t = \rho_l$	ρ_b	$(V@M_n)/V_n$	$P/(A_{a}f'_{a})$	$(V@M)/(A \sqrt{f'})$	$V/(\Delta \sqrt{f'})$
No.	~peeimen code	w/ 1w	(%)	(%)	(***********	1 / (1 -g1 C)	$(\mathbf{v} \otimes \mathbf{w}_n)/(\mathbf{A}_{cv} \mathbf{v}_c)$	$n'(1 \text{ cv } V^{1}c)$
1	RW-A20-P10-S38	2.0	0.27	3.23	0.80	0.10	3.8	NA
2	RW-A20-P10-S63		0.61	7.11	0.88	0.10	6.3	NA
3	RW-A15-P10-S51	. 1.5	0.32	3.23	0.80	0.10	5.1	NA
4	RW-A15-P10-S78		0.73	6.06	0.84	0.10	7.8	NA
5	RW-A15-P2.5-S64		0.61	6.06	0.79	0.025	6.4	NA
6	RW-A15-P10-S55		0.73	3.23	0.60	0.10	5.5	NA
7	RW-A10-P15-S60	1.0	0.29	5.01	1.82	0.15	NA	6.0
8	RW-A10-P20-S60	1.0	0.29	3.67	1.91	0.20	NA	6.0

Table 1 - Wall specimen attributes

Notes: h_w/l_w is the aspect ratio; ρ_t is the horizontal web reinforcement ratio; ρ_l is the vertical web reinforcement ratio; ρ_b is the boundary longitudinal reinforcement ratio; $(V@M_n)/V_n$ is the design ratio of the lateral load corresponding to the nominal moment capacity over the nominal shear strength using the specified compressive strength of concrete and specified yield strength of reinforcement; $P/(A_g f_c)$ is the design axial load ratio using the design axial load and specified compressive strength of concrete; $(V@M_n)/V_n$ is the design ratio of the lateral load corresponding to the nominal moment capacity over the nominal shear strength using the specified material load corresponding to the nominal moment capacity over the nominal shear strength using the specified material strengths; $(V@M_n)/(A_{cv}\sqrt{f_c})$ and $V_n/(A_{cv}\sqrt{f_c})$ are the design ratio of the average shear stress at nominal moment and shear capacity, respectively, over $\sqrt{f_c}$ using the specified material strengths, for f_c in psi units; NA is "Not Applicable".

Figure 1 and Table 2 show the cross-section and reinforcement details of all eight specimens. Diameters of #2, #3, #4, #5, #6 are 1/4 in. (6.4 mm), 3/8 in. (9.5 mm), 1/2 in. (12.7 mm), 5/8 in. (15.9 mm), 3/4 in. (19.1 mm), respectively, whereas nominal diameters of the D6a and D6b reinforcement were 6 mm. Although the center-to-center spacing between adjacent longitudinal boundary reinforcement is shown as 50 mm (2 in.) in Figure 1, this spacing was 70 mm (2.75 in.) for the specimen with highest axial load, RW-A10-P20-S60. Boundary longitudinal reinforcement consisted of eight deformed bars (either #4, #5, or #6), whereas web reinforcement consisted of two curtains of either 6mm-diameter or #3 bars. Transverse reinforcement at the wall boundaries satisfied ACI 318-14 S18.10.6.4 requirements for Special Boundary Elements, which reflect the highest category of wall boundary confinement and restraint against bar buckling that is prescribed by ACI 318-14. Yield strength was approximately 475 MPa for #4, #5, #6 bars, 440 MPa for #2, #3, D6a bars, and 515 MPa for D6b bars. Ultimate strength of #2 was 490 MPa, while the average ultimate strength for all remaining reinforcement was approximately 635 MPa.

Three concrete cylinders were tested for each wall specimen to obtain the average compressive strength. The average concrete compressive strength at test date for the eight wall specimens ranged from 45 MPa to 56 MPa. Concrete clear cover over boundary vertical reinforcement was selected to be greater than or equal to one vertical boundary bar diameter (either US #4, #5, or #6); therefore, a maximum aggregate size of 3/8 in. (9.5 mm) was specified. More details on material properties are provided by Tran and Wallace [3][4][5] and Tran et al. [6].





Figure 1 - Wall reinforcement details

Test No.	Wall specimen	"a"	"b"	"c"	"d"
1	RW-A20-P10-S38	4#4	4#4	6D6a @140 (@5.5in.)	D6b @140 (@5.5in.)
2	RW-A20-P10-S63	4#6	4#6	5#3 @152 (@6in.)	#3 @152 (@6in.)
3	RW-A15-P10-S51	4#4	4#4	7D6a @114 (@4.5in.)	D6b @114 (@4.5in.)
4	RW-A15-P10-S78	4#6	4#5	6#3 @127 (@5in.)	#3 @127 (@5in.)
5	RW-A15-P2.5-S64	4#6	4#5	5#3 @152 (@6in.)	#3 @152 (@6in.)
6	RW-A15-P10-S55	4#4	4#4	6#3 @127 (@5in.)	#3 @127 (@5in.)
7	RW-A10-P15-S60	4#5	4#5	6D6b @127 (@5in.)	D6b @127 (@5in.)
8	RW-A10-P20-S60	4#5	4#5	6D6b @127 (@5in.)	D6b @127 (@5in.)

Table 2 - Wall reinforcement details

2.2 Test procedures and instrumentation

The cantilever wall specimens were tested in an upright position, with a horizontal lateral load applied 8 ft (2440 mm), 6 ft (1830 mm), and 4 ft (1220 mm) from the base of the wall for the aspect ratio 2.0, 1.5, and 1.0 specimens, respectively. Axial load was applied using a loading beam positioned across the top of the wall with two, hollow-core jacks, one on each side of the wall, applying load near the ends of the loading beam. The forces in the jacks were resisted by post-tensioning bars that were attached to the laboratory strong floor (Fig. 2). The lateral load was applied through a friction mechanism using two plates, one on either wall face along with through-wall post-tensioning bars, to spread the lateral load uniformly across the top of the wall. The reversed cyclic lateral load was applied to the wall at a very slow rate. An out-of-plane support frame was used to prevent wall twisting. The testing protocol for the first six specimens consisted of load-controlled cycles, generally three cycles at 1/4, 1/2, and 3/4 of the expected yield force, followed by displacement-controlled cycles, typically three cycles at top drift ratios of 0.375%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, and two cycles at top drift ratios of 3.0%, and 4.0%. For the last two specimens with shear dominant failure, RW-A10-P15-S60 and RW-A10-P20-



S60, the testing protocol included load-controlled cycles at 1/8, 1/4, 1/2, and 3/4 of the nominal shear capacity that was computed in accordance with ACI 318-14, followed by displacement-controlled cycles at drift ratios of 0.125%, 0.25%, 0.375%, 0.5%, 0.75%, 1.0%, 1.5%, and 2.0%.



Figure 2. Test setup for typical wall specimen

Linear variable differential transformers (LVDTs) were used to measure lateral wall displacements, wall foundation sliding and uplift, and to allow determination of wall average concrete strains over specified gauge lengths (e.g., to enable calculation of wall curvature). The LVDT layout for a wall with an aspect ratio of 1.5 is shown in Fig. 3. Reinforcement strains were measured at 30 locations using strain gauges affixed to vertical boundary reinforcement, vertical and horizontal web reinforcement, and transverse reinforcement over the height of about $l_w/2$ above the wall-foundation interface. Load cells were used to measure the applied lateral and axial load.



Figure 3. Sensor configuration for walls with aspect ratio 1.5 (Units: in.; 1 in. = 2.54 cm)



3. Failure Modes

Photographs of the eight wall specimens at failure are presented in Fig. 4. For the walls with moderate shear stress, i.e., RW-A20-P10-S38 and RW-A15-P10-S51, strength loss was a result of a diagonal tension failure, initiated by concrete crushing and buckling of wall boundary longitudinal reinforcement. For the wall with an aspect ratio of 2.0 and high shear stress, i.e. RW-A20-P10-S63, crushing of core concrete at wall boundaries, together with diagonal compression, led to out-of-plane buckling of boundary longitudinal reinforcement and some web vertical reinforcement. For the wall with an aspect ratio of 1.5, high shear stress, and axial load of $0.1A_g f_c$, i.e. RW-A15-P10-S78, strength loss was caused by large diagonal compression leading to shear sliding and significant out-of-plane buckling of wall boundary longitudinal reinforcement. For the wall with an aspect ratio of 1.5 and high shear stress but low axial load $(0.025A_g f_c)$, i.e., RW-A15-P2.5-S64, large diagonal compression and significant shear sliding were observed, followed by in-plane buckling of boundary longitudinal reinforcement. For both of the walls with an aspect ratio of 1.0, i.e., RW-A10-P15-S60 and RW-A10-P20-S60, diagonal tension failure occurred. More details on failure modes of test specimens are provided by Tran and Wallace (2015) [5] and Tran et al. (2015) [6].



Figure 4. Photos of wall specimens at failure

4. Shear Strength and Deformation Capacity

A relationship between shear strength and ductility demand is mentioned in PEER/ATC 72-1 [1] for both columns and walls. The relationship used for columns in ASCE/SEI 41-06 is presented in Fig. 5 for three types of failure modes: shear, flexure-shear, and flexure. Shear failure types consist of diagonal tension failure, web



crushing, and sliding shear. Flexure failures include concrete crushing followed by buckling of longitudinal boundary reinforcement and eventual fracture of longitudinal reinforcement. Flexure-shear failures are typically characterized by concrete crushing followed by buckling of longitudinal boundary reinforcement and/or diagonal tension failures [3]. For both flexure and flexure-shear failures in walls, the boundary element reinforcement may buckle in- or out-of-plane depending on the ratio of wall thickness to concrete side cover, axial load, and shear demand [5].



Figure 5. ASCE/SEI 41-06 Column shear strength versus ductility relation [1]

Test data from the eight shear walls in addition to forty-seven other test specimens [7][8][9][10][11][12] [13] [14][15] are investigated to assess whether the relationship presented in Fig. 5 for columns is applicable to structural walls. These well-detailed walls were selected from a comprehensive database which includes over two hundred test specimens. For this study, well-detailed walls were defined as walls with at least one-half of the area of boundary transverse reinforcement required by ACI 318-14 S18.10.6.4 for Special Boundary Elements and a ratio of vertical hoop/crosstie spacing to longitudinal boundary bar diameter (s/d) less than 8 (consistent with criteria used in ASCE 41-06 [16] to assess the level of confinement of boundary elements). These walls also have shear span ratios ranging from 1.0 to 3.0, web thickness larger than 75 mm, axial load levels, $P/A_g f_c$, less than 25%, and medium to high ductility reinforcing steel.



Figure 6. Wall curvature ductility versus $V_{\text{max}}/V_{n,ACI}$



In this study, wall curvature ductility $\mu_{\phi} = \frac{\phi_u}{\phi_y}$ and plastic rotation θ_{pl} were calculated based on the following relations [17]:

$$\delta_u = \delta_v + \theta_{pl} (h_w - l_p/2) \tag{1}$$

$$\theta_{pl} = (\phi_u - \phi_y) l_p \tag{2}$$

where ϕ_u is the ultimate curvature, ϕ_y is the yield curvature, δ_u is the top lateral displacement at significant strength loss, δ_y is the top lateral displacement at yield, h_w is the wall height, $l_p = \frac{l_w}{2}$ is the plastic hinge length for well-detailed walls, and l_w is the wall length. Values of δ_u and δ_y were determined from the test data.

A relationship for wall curvature ductility versus $V_{\max}/V_{n,ACI}$ and $V_{\max}/[A_{cv}\sqrt{f_c}]$ is shown in Figures 6 and 7, respectively, where V_{\max} is the maximum shear determined from test data and $V_{n,ACI}$ is the nominal shear capacity according to ACI 318-14 S18.10.4. All specimens with $V_{\max}/V_{n,ACI} \leq 1.0$ have failure type of "flexure" and curvature ductility $\mu_{\phi} \geq 10$. In addition, most of the walls with shear failure modes have $V_{\max}/V_{n,ACI} \geq 1.5$ and curvature ductility $\mu_{\phi} \leq 5$. Wall shear capacity is also dependent on the axial load level. The test with design axial load of $0.2A_g f_c$, i.e., RW-A10-P20-S60, has shear capacity of approximately $1.75V_{n,ACI}$.



Figure 7. Wall curvature ductility versus $V_{\text{max}} / [A_{cv} \sqrt{f_c}]$

The relationship between wall plastic rotation and $V_{\text{max}}/V_{n,ACI}$ is shown in Fig. 8. The figure indicates an average plastic rotation capacity of 0.005 for walls with shear failures and plastic rotation capacities between 0.015 and 0.035 for most walls with flexure failures. These are the results for walls with aspect ratio up to 3.0; larger roof drift ratios are likely for more slender walls in very tall buildings [18].

0.08



Figure 8. Wall plastic rotation versus $V_{max}/V_{n,ACI}$

Plastic Rotation

5. Conclusions

An experimental research program consisting of eight large-scale reinforced concrete shear walls and an investigation on shear strength and deformation capacity for well-detailed walls having shear span ratio between 1.0 and 3.0, web thickness larger than 75 mm, axial load level, $P/A_g f_c$, less than 25%, and medium to high ductility reinforcing steel was conducted. Based on the study, it can be concluded that walls with $V_{\text{max}}/V_{n,ACI} \leq 1.0$ have failure type of "flexure" and have curvature ductility $\mu_{\phi} \geq 10$. Most of the walls with shear failure modes have $V_{\text{max}}/V_{n,ACI} \geq 1.5$ and curvature ductility $\mu_{\phi} \leq 5$. Furthermore, wall shear capacity increases with an increase in axial load. For one of the specimens tested, the shear capacity was $1.75V_{n,ACI}$ for an axial load of $0.2A_g f_c$. Walls with shear failures have modest plastic rotation capacity and walls with flexure failures have plastic rotation ranging approximately from 0.015 to 0.035.

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