A NEW DESIGN APPROACH TOWARD A DUCTILE REINFORCED CONCRETE SQUAT WALL

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Abstract

Reinforced concrete (RC) squat walls, with a height-to-length ratio of 2.0 or less, have high strength and stiffness which make them a popular seismic force-resistant system for low-rise buildings, parking structures, and nuclear power plants. However, extensive studies on squat shear walls have shown that squat walls have limited drift ductility because a flexural yielding mechanism is difficult to achieve. This is mainly because the failure of a squat wall initiates from the compression failure of diagonal struts across the web. This research investigates a new and simplistic reinforcing detail for squat walls to achieve a ductile seismic behavior. While ACI 318-19 requires a mesh of steel bars to reinforce squat walls, the new detail fortifies the squat walls by several steel cages which contain vertical bars enclosed by transverse hoops. Each steel cage is similar to that used in RC columns and can be easily prefabricated to significantly reduce the onsite assembly work. Each cage is much like a well-confined column, which prevents failure in the concrete diagonal struts before the longitudinal rebars yield. Ductility of the new squat walls are much enhanced when longitudinal rebar yielding occurs first. Both ACI compliant and proposed walls with aspect ratios of 0.5 and 1.0 were tested under large displacement reversals. Similar to prior research results, ACI compliant squat walls exhibited a fast deterioration in shear strength at low drift ratios and failed in a sliding shear failure mode after severe damage occurred near the wall base due to failure of intersected compression struts under cyclic loading. On the other hand, the new squat walls showed excellent ductility without sliding shear failure. The proposed new design allows squat walls to develop a ductile seismic behavior which is essential to promoting levels of safety during seismic events. The enhanced ductility also warrants a higher strength reduction factor, ϕ, of greater than 0.6, which is currently specified for squat walls by the ACI code.

Keywords: Reinforced Concrete, Squat Wall, ACI, Ductility

1. Introduction

Shear walls with a height-to-length aspect ratio of less than 2.0 are classified as squat shear walls. The American Concrete Institute's Building Code (ACI 318-19) [1] requires a steel rebar mesh (distributed vertical and horizontal steel rebars) to reinforce the squat shear walls. Squat walls have high shear strength and stiffness which make them a popular seismic force-resisting system for low-rise buildings, parking structures, and nuclear power plants. However, results of numerous tests on squat shear walls [2-7] reveal insufficient drift ductility as the flexural yielding mechanism is difficult to achieve. Most tested squat shear walls suffer from sliding shear failure, which prevents them from achieving reasonable drift ductility. The sliding shear failure initiates from the crushing of the unconfined concrete at a wall's web; then, the concrete crushing propagates through a sliding plane which runs parallel to the wall base, causing the squat walls to abruptly lose their shear strength. Consequently, ACI code Section 21.2.4.1 assigns a low strength reduction factor, ϕ, of 0.6 to shear-controlled low-rise walls [1]. Numerous studies have explored the potential of enhancing drift ductility and delaying (or avoiding) the sliding shear failure by providing additional shear-friction steel bars, enhanced interface resistance (with rough, untreated or a grooved interface between the wall base and footing), fiber-reinforced concrete (FRC), diagonal reinforcement, high-strength steel, or high-strength concrete. However, the drift ductility improvement, which is the motivation of this study, has always been considered minor.

A commonly used efficient method to confine concrete is to provide steel transverse reinforcement to enclose the longitudinal steel bars (which is similar to the steel cage of a reinforced concrete column); therefore, the concrete strength and ductility are improved. ACI code Section 18.7.5.4 specifies the amount and spacing for enhancing the confinement effectiveness for columns in special moment frames. The required amount of confinement reinforcement depends on the strength of the concrete and the confinement reinforcement, total concrete area, enclosed concrete area, spacing of longitudinal steel bars, spacing of transverse reinforcement, and applied axial force [1].

In this study, multiple steel cages were used to reinforce the squat shear walls. Each steel cage consists of vertical steel bars enclosed by transverse hoops. Each cage is much like a well-confined column which prevents the diagonal concrete struts from failure before the longitudinal rebars yield. The volume ratio of the transverse hoops is calculated according to ACI code Section 18.7.5.4 [1] for confined columns. This was done to confine the web of a squat wall, which always fails first then leads to a sliding failure in a conventionally reinforced squat wall. The proposed methodology not only improves ductility, but also eliminates the steel cage on-site assembly by prefabrication.

2. Experimental Program

2.1 Specimens description

To verify the proposed methodology, two series of scaled squat shear walls were tested having an aspect ratio of 0.5 or 1.0. In each wall series, a specimen reinforced by the proposed reinforcing detail was designed to have similar amount of total vertical steel area as the counterpart specimen designed by ACI provisions. However, the two walls have different horizontal steel reinforcement configurations. Fig. 1 and Table 1 summarize the details of each specimen. Specimen designations were selected to refer to test variables, such as design criteria (ACI or proposed), and wall aspect ratio (0.5 or 1.0). For example, Specimen SW-A-0.5 represents a squat wall (SW) designed by ACI (A) requirements, and the wall's aspect ratio is 0.5. The 0.5-aspect-ratio walls have a thickness, length, and height of 102 mm, 1016 mm, and 508 mm, respectively. While the 1.0-aspect-ratio walls have a thickness, length, and height of 102 mm, 1016 mm, and 1016 mm, respectively. In the first series, two 0.5-aspect-ratio specimens (SW-A-0.5 and SW-P-0.5) were constructed according to ACI specifications and the proposed detailing, respectively. Both walls had a similar total amount of vertical steel area but different horizontal configurations. For specimen SW-A-0.5, the target design shear stress was selected as $1.2\sqrt{f'_c}$ (MPa). Considering a design concrete compressive strength of 34.5 MPa, and using the ACI equation in Section 18.10.4.1: $V_n = A_{cv}(\alpha_c\lambda\sqrt{f'_c}+\rho_t f_y)$ where α_c = 0.25 for a squat wall with a

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height-to-length ratio of less than 1.5, the wall web was reinforced by #10M rebars with a spacing of 102 mm in both the vertical and horizontal directions. Each boundary element was designed according to ACI code Section 18.10.6.4 [1]. Accordingly, the boundary element consisted of four #13M and two #10M vertical steel bars confined by #10M hoops with a spacing of 32 mm. In specimen SW-P-0.5, the volume ratio of transverse hoops was calculated according to confining requirements for columns in the ACI code Section 18.7.5.4 [1]; hence, the wall was reinforced by four 254-mm wide steel cages. The two boundary cages consisted of four #13M and two #10M vertical rebars which were enclosed by #10M hoops with a spacing of 32 mm up to two-thirds of the wall's height. The spacing was relaxed in the upper one-third of the wall's height by using #10M hoops at a spacing of 51 mm. The hoop's spacings of the middle two cages were similar to that in the boundary cages but with six #10M vertical rebars in each cage.

The other series of specimens (SW-A-1.0 and SW-P-1.0) had a height-to-length ratio of 1.0, designed according to ACI provisions and the proposed details, respectively. The SW-A-1.0 wall's web had a steel mesh of #10M rebars at a spacing of 127 mm in both the vertical and horizontal directions. The quantity of steel reinforcement was calculated based on a selected target design shear stress of $0.83\sqrt{f'_c}$ (MPa) with a design concrete compressive strength of 34.5 MPa. The two boundary elements consisted of four #16M vertical steel rebars enclosed by #10M hoops at a spacing of 70 mm. For specimen SW-P-1.0, the wall was reinforced by four steel cages: The two middle cages had a width of 273 mm and the two outer cages had a width of 235 mm. The boundary cages' width was narrower than that of the middle cages to satisfy the maximum allowed center-to-center spacing of the laterally supported longitudinal rebars as per ACI code Section 18.10.6.4 (f) [1]. All steel rebars had a nominal yield strength of 420 MPa.

Table 1 – Specimen information

Specimen[1]	f_{cm}[2] **(MPa)**	**Web reinforcement**		**Boundary reinforcement**	
		Horizontal, ρ_t (%)	**Vertical, ρ_l (%)**	**Horizontal, ρ_t (%)**	**Vertical, ρ_l (%)**
SW-A-0.5	34.5	1.38 (#10@102)	1.38 (#10@102)	4.4 (#10@32)	2.47 (4#13; 2#10)
SW-P-0.5	34.5	4.4[3] (#10@32)	1.65 (6#10 per cage)	4.4[3] (#10@32)	2.55 (4#13; 2#10)
SW-A-1.0	28.3	1.1 (#10@127)	1.1 (#10@127)	2.0 (#10@70 in.)	5.6 (4#16)
SW-P-1.0	39.3	4.4[3] (#10@32)	1.5 (6#10 per cage)	4.4[3] (#10@32)	3.8 (6#13; 2#10)

[1]: SW refers to shear walls; A denotes ACI compliant design; P refers to proposed details, 0.5 or 1.0 represents the wall height-to-length ratio.
[2]: f_{cm} is the measured concrete compressive strength.
[3]: ρ_t =4.4% (#10@32 mm) from the wall base to $2h/3$. ρ_t =2.75% (#10@51 mm) from $2h/3$ to h.

2.2 Test setup and instrumentation

The test setup consisted of a specimen cast monolithically with a fixing block and loading block as shown in Fig. 2. The fixing block had a thickness, length, and height of 1100 mm, 1625 mm, and 1830 mm, respectively; while the loading block had a thickness, length, and height of 660 mm, 660 mm, and 1400 mm, respectively. The fixing block was post-tensioned by six threaded rods to the strong floor, while the loading block was tightened to a 1500-kN servo-controlled actuator. The loading block was restrained from out-of-plane movements by a lateral bracing frame on both sides perpendicular to the loading direction. The surface contact between the frame plate and the loading block was lubricated by grease to minimize the friction and resistance provided by the lateral bracing system. Linear variable differential transformers (LVDTs) were used to measure the displacements of the two blocks and the specimen. The drift ratio of the wall was computed by the difference between the displacement corresponding to the line of force action (center of the actuator's

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head), which is represented by point A in Fig. 2, and the displacement of the wall base which is point B in Fig. 2, divided by the distance between the two points. The loading protocol is also shown in Fig. 2.

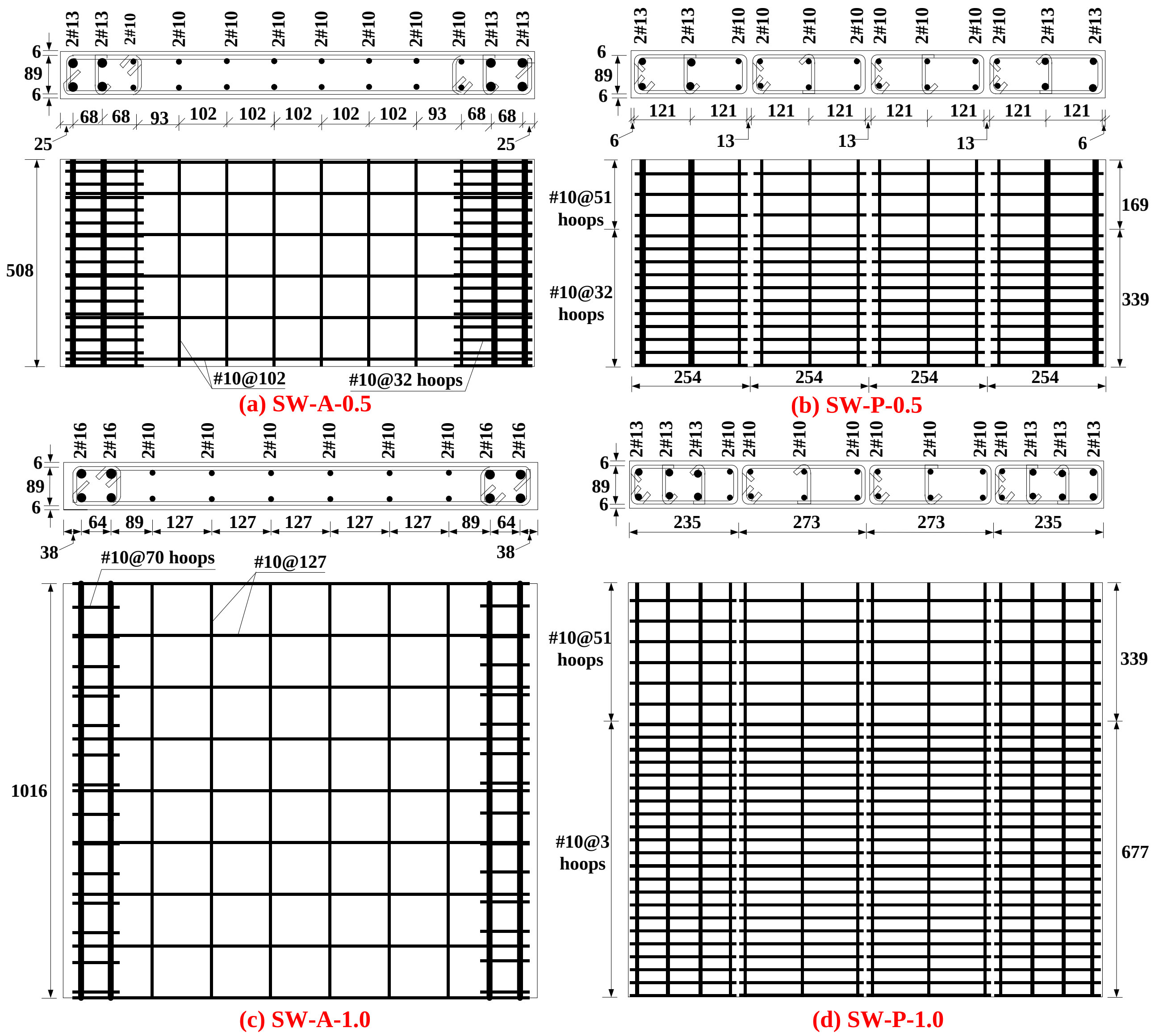

Fig. 1 – Specimen reinforcement details (units: millimeter). All bars sizes are in metric size.

3. Experimental results

3.1 Cracking patterns and damage progress

The cracking pattern of each test specimen is summarized in Fig. 3 and Fig. 4. In specimen SW-A-0.5, the onset of web concrete crushing commenced at 1.25% drift ratio; then, its shear strength dropped to 65% of the maximum attained shear force. The web concrete crushing propagated from the wall's web to the boundaries at 1.5% drift ratio and ultimately formed a sliding plane near the wall base. Beyond 1.75% drift ratio, the shear strength was primarily resisted by the dowel action of vertical rebars as shown in Fig. 3. On the other hand, there was no concrete crushing in specimen SW-P-0.5 up to 4% drift ratio because the concrete was well

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confined by the transverse reinforcement, and only minor concrete cover spalling was observed at 2% drift ratio. As shown in Fig. 5, the shear strength plateaued up to 2% drift ratio. In specimen SW-A-1.0, the sliding plane was formed at 1% drift ratio, the concrete crushing propagated around the sliding plane at 1.25% drift ratio where the shear strength dropped to 85% of its peak value. In contrast, specimen SW-P-1.0 maintained its shear strength up to 2% drift ratio, and only minor concrete cover spalling was observed at 2% drift ratio.

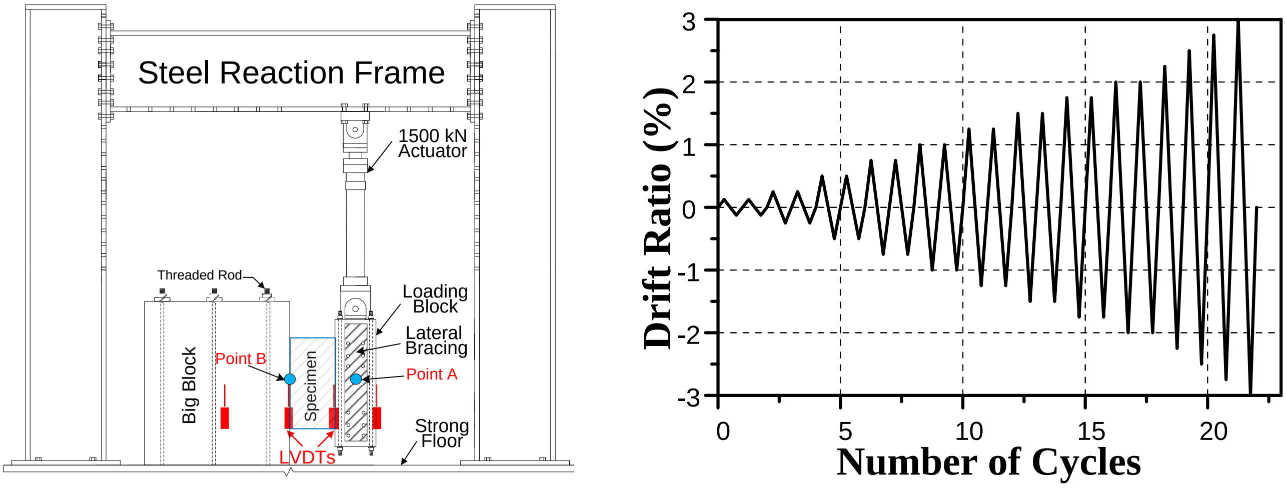

Fig. 2 – Test setup and loading protocol.

3.2 Hysteretic loops and energy dissipation

Shear stress versus drift ratio responses of the test specimens are shown in Fig. 5. Specimen SW-A-0.5 reached its peak shear stress of $0.99\sqrt{f'_c}$ (MPa) at 1.25% drift ratio (positive direction); then, the shear stress abruptly dropped when the sliding shear failure occurred. A significant concrete crushing was observed at sliding plane as shown in Fig. 3. On the other hand, the shear stress in specimen SW-P-0.5 was maintained at a stress of $0.92\sqrt{f'_c}$ (MPa) up to 2.5% drift ratio (positive direction). This indicates a doubled ductility as compared to specimen SW-A-0.5. Specimen SW-P-0.5 also showed a much more gradual strength degradation than specimen SW-A-0.5. It maintained 81% of its peak stress when reaching 3% drift ratio. Afterward, the wall's strength gradually degraded as the concrete between the steel cages cracked, leading to a gradual separation of the steel cages. Nonetheless, the concrete inside the steel cages remained confined after the concrete cover spalled. These results confirm that confining the concrete at the wall web eliminates the sliding shear failure and enhances the drift ductility of squat walls. Likewise, specimen SW-A-1.0 reached its peak shear strength of $0.63\sqrt{f'_c}$ (MPa) at 0.75% drift ratio but suddenly dropped to 30% of its peak shear stress at 1% drift ratio. On the other hand, specimen SW-P-1.0 maintained its shear strength of $0.7\sqrt{f'_c}$ (MPa) until 2% drift ratio. Beyond this point, its shear strength slightly dropped and maintained approximately 75% of its peak value until 4% drift ratio.

Fig. 6 illustrates the normalized shear strength backbone curves of the tested specimens. The ACI compliant 0.5-aspect-ratio wall dropped to 65% of its peak strength at 1.25% drift ratio and continued losing its strength down to 40% of the peak value at 2% drift ratio because the concrete of the wall's web crushed and sliding shear failure occurred. On the other hand, the 0.5-aspect-ratio wall with the proposed detailing reserved its peak shear strength up to 2.5% drift ratio which is twice that attained by the ACI compliant wall since the concrete was well-confined. Similarly, the ACI compliant 1.0-aspect-ratio wall dramatically lost its shear strength after 1% drift ratio down to only 25% of its peak value, while the wall with the proposed detailing maintained a ductile behavior up to 4% drift ratio with only a 19% shear strength drop after 2% drift ratio.

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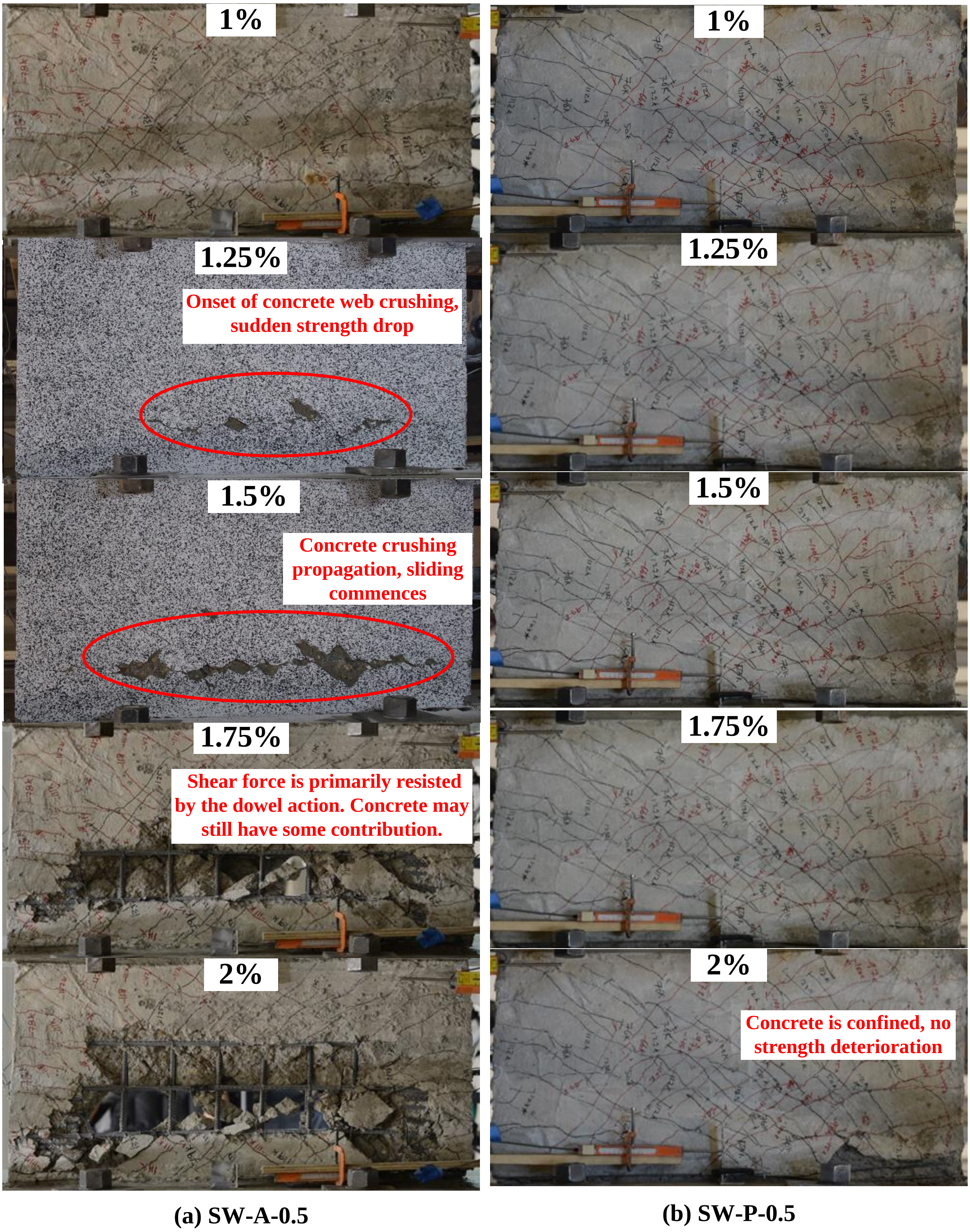

Fig. 3 – Damage pattern in specimen SW-A-0.5 and SW-P-0.5.

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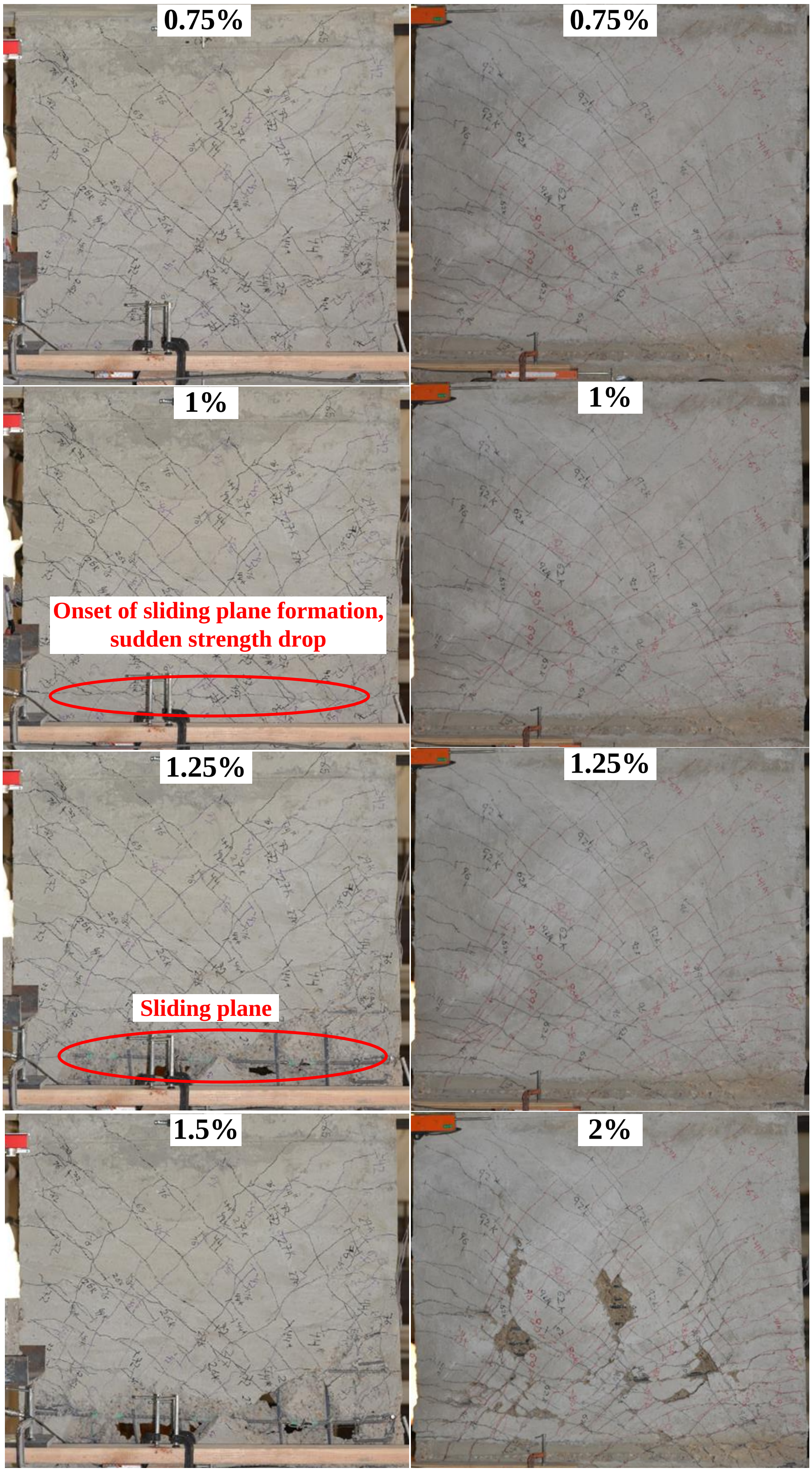

(a) SW-A-1.0 **(b) SW-P-1.0**

Fig. 4 – Damage pattern in specimen SW-A-1.0 and SW-P-1.0.

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The specimens' cumulative energy curves are shown in Fig. 7. Specimen SW-P-0.5's dissipated energy was 60% more than that of SW-A-0.5 at 2% drift ratio, and its dissipated energy was more than 2.6 times at 4% drift ratio. Also, specimen SW-P-1.0's dissipated energy is 3.8 times that of specimen SW-A-1.0 at 2% drift ratio, and 10 times greater than that dissipated by SW-A-1.0 at 8% drift ratio. The cumulative energy of the ACI compliant walls were terminated at 2% drift ratio because they lost more than 60% of their strength, and the walls' displacement was only due to the wall rigid body movement (wall slip) which is the result of the sliding shear failure.

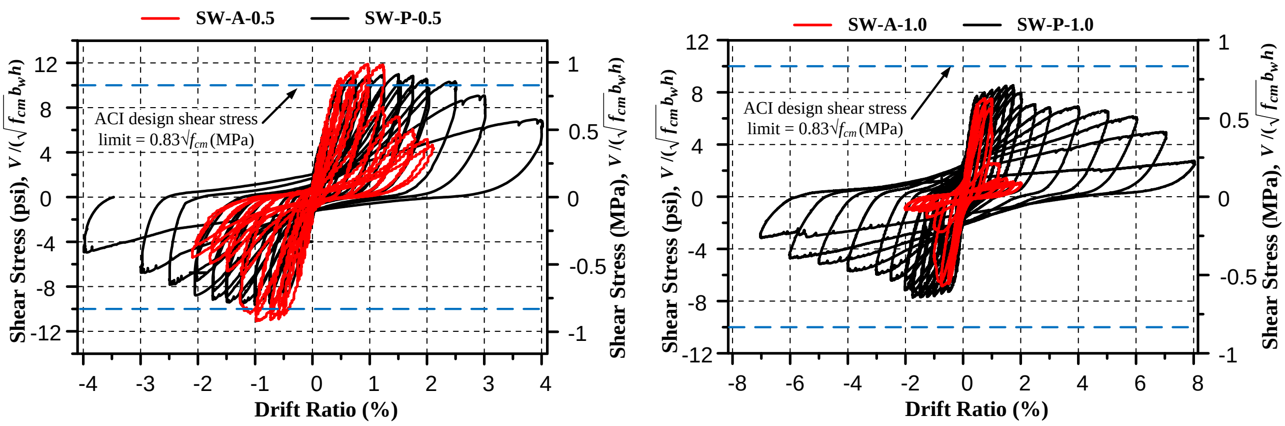

Fig. 5 – Shear stress responses.

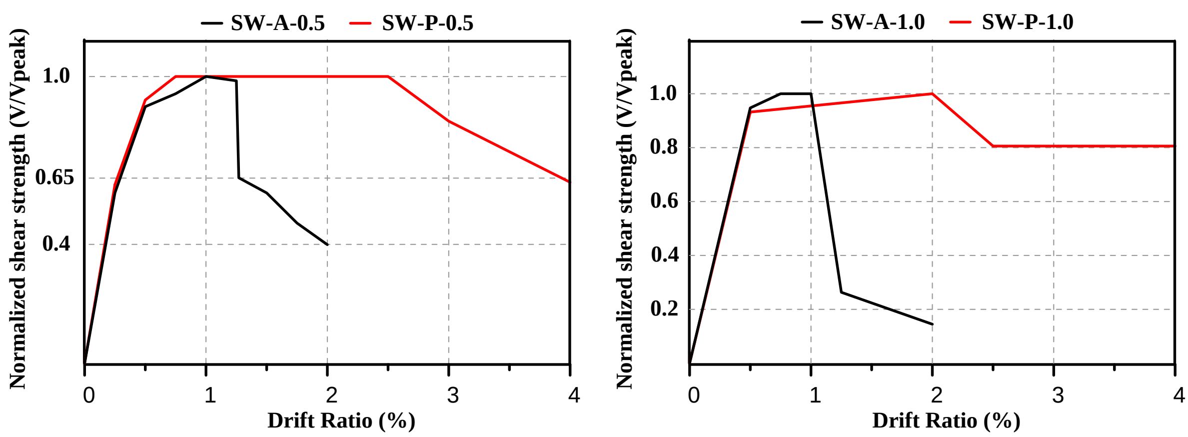

Fig.6– Normalized shear strength backbone curves of 0.5 and 1.0 aspect ratio walls.

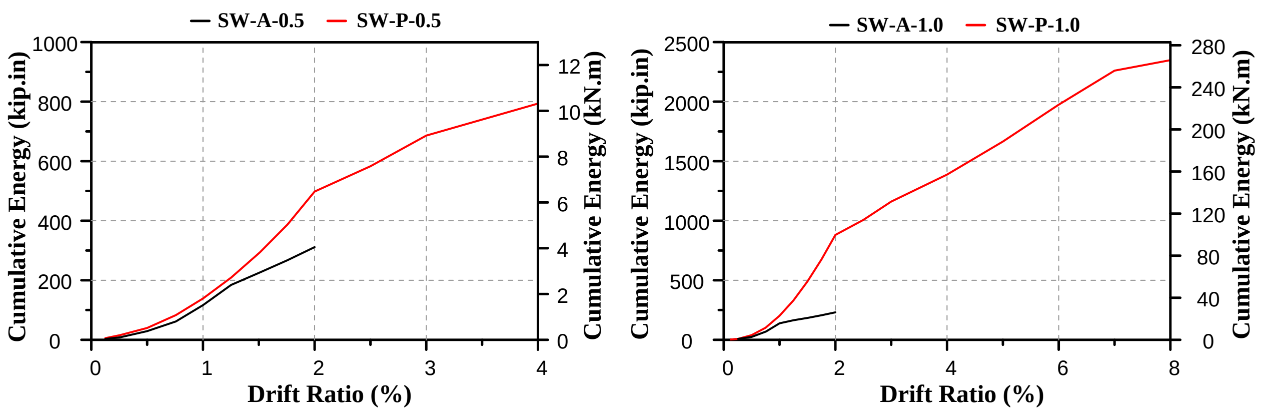

Fig.7– Cumulative energy curves of 0.5 and 1.0 aspect ratio walls.

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4. Summary and Conclusions

Extensive prior experimental studies have revealed a low drift capacity and a brittle failure mode of RC squat shear walls. This is mainly due to the sliding shear failure caused by the concrete crushing of the web. To enhance the shear behavior of squat walls, a new design methodology was investigated. This new design confines the wall web by using multiple steel cages similar to that used for columns in special moment frames. The proposed design was verified by testing four reduced scaled specimens having an aspect ratio of 0.5 or 1.0 and reinforced according to both the ACI requirements and by the proposed design. The experimental test results indicate that squat walls designed by the new design methodology sustained twice the drift ductility compared to that of the ACI compliant walls, and the strength loss was very gradual as opposed to a sudden drop. This is because the concrete in the web was well-confined which eliminated the sliding shear failure. The confined concrete in the web had greater deformation capability which forced the longitudinal rebar yielding to occurs first. The proposed new design allows squat walls to develop a much more ductile seismic behavior and energy dissipation capability which is essential to promote increased levels of safety during seismic events. The enhanced ductility also warrants a higher strength reduction factor, ϕ, of greater than 0.6 that currently specified for squat walls by the ACI code.

5. References

[1] American Concrete Institute: *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI318R-19)*, Farmington Hills, Michigan, 2019.

[2] Baek, J., Park, H., Lee, J., & Bang, C. (2017): Cyclic loading test for walls of aspect ratio 1.0 and 0.5 with grade 550 MPa shear reinforcing bars. *ACI Structural Journal*, 114(4), 969. doi:10.14359/51689680

[3] Barda F., Hanson J. M., and Corley W. G. (1977): Shear Strength of Low-Rise Walls with Boundary Elements, *ACI Special Publications*, Reinforced Concrete in Seismic Zones, SP-53-8, pp.149-202. (Also, Research and Development Bulletin, No. RD043.01D, Portland Cement Association.) Buffalo, NY, 2009, 291 pp.

[4] Gulec, C. K., and Whittaker, A. S. (2009): Performance-Based Assessment and Design of Squat Reinforced Concrete Shear Walls, Report No. MCEER-09-0010, *Multidisciplinary Center for Earthquake Engineering Research*, Buffalo, NY, 2009, 291 pp.

[5] Hidalgo P. A., Ledezma C. A., and Jordan R. M. (2002): Seismic Behavior of Squat Reinforced Concrete Shear Walls, *Earthquake Spectra*, Vol. 18, No. 2, pp. 287-308.

[6] Pauley T., Priestley M. J. N., Synge A. J. (1982): Ductility in Earthquake Resisting Squat Shear walls, *ACI Journal*, Vol. 79, No. 26, pp. 257-269.

[7] Wood, S. L., (1990): Shear Strength of Low-Rise Reinforced Concrete Walls, *ACI Structural Journal*, V. 87, No. 1, Jan.-Feb., pp. 99-107.

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