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Evaluation of Shear-Flexure Interaction Behavior of Reinforced Concrete Wall

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Abstract

Reinforced concrete (RC) wall is a critical structural member that resists lateral loadings, such as earthquake and wind. RC wall having moderate height to length ratio, 1.50-2.50, has the altered shear-flexure interaction (SFI) behavior, so shear and flexural failure mechanisms occur almost concurrently. Therefore, an experimental study of a moderate RC wall was conducted as a comprehensive study of the wall's coupled nonlinear shear-flexure behavior under cyclic loading. The experimental results show that the RC wall failed in flexure mechanism, indicated by crushing of the flexural compression zone, and followed by immediate shear failure, notified by the occurrence of web crushing. In addition to the experiment, an analytical model using SFI-MVLEM element in OpenSees software was performed to verify the experimental results. The analytical results show that the model is able to simulate reasonably well the coupled nonlinear shear-flexure behavior of the RC wall subjected to cyclic loading.

Introduction

Reinforced concrete (RC) wall is a critical structural member in concrete buildings to resist lateral loadings, such as earthquake and wind. In typical structures, it can take the form of individual shear walls or core walls. RC walls are generally classified into three categories, namely squat, moderate, and slender walls for those having height to length ratio (h_w/l_w) of 0.25-1.50, 1.50-2.50, and greater than 2.50, respectively. Squat RC wall tends to fail in the shear-controlled mechanism, whereas slender RC wall tends to fail in the flexure-controlled mechanism. On the other hand, in moderate ratio of RC walls, shear and flexural failure mechanisms occur almost concurrently. This is further referred to as shear-flexure interaction (SFI) [1]. Most of present concrete buildings construct RC walls that can be categorized as moderate to slender RC walls. While the behavior of slender RC walls can be predicted quite well using flexural theory, it may not be the case for moderate RC walls. Therefore, it emphasizes the necessity of the comprehensive study of the coupled nonlinear shear-flexure behavior of such RC walls under cyclic loading either by experimental study or by analytical study.

The nonlinear behavior of RC walls can be analyzed in the form of a macroscopic model consisting of fiber-panel-based elements. The multiple vertical line element model (MVLEM) has been extensively used in the nonlinear analysis of RC walls. The MVLEM was proposed by Orakcal et.al [2], and it is well-known due to its accuracy and stability. This model was extensively developed to influence the coupled shear-flexure behavior leading to the new shear-flexure interaction multiple vertical line elements model (SFI-MVLEM) [1]. The SFI-MVLEM has also been

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used in the previous study of SFI behavior of moderate RC walls [3]. It was found that the model could predict reasonably accurate the ultimate load and displacement of moderate RC walls.

This current study focuses on investigating the SFI behavior of moderate RC walls by performing an experimental study at the Structural Engineering Laboratory of Petra Christian University. In addition, the hysteretic curve obtained from the experiment was also compared with analytical result using the SFI-MVLEM element in OpenSees software.

SFI-MVLEM Model

Kolozvari [1], in 2013, developed SFI-MVLEM model of RC walls by replacing the uniaxial element of MVLEM with an m-number of RC panels. Furthermore, the effect of dowel action was also included in the new model. The analytical model of the element is shown in Figure 1. Each SFI-MVLEM element has six degrees of freedom (DOFs), which are located at the center of the top and bottom rigid beams. These DOFs were introduced in order to capture the normal strain in the vertical direction and shear strain. On the other hand, normal strain in the horizontal direction was evaluated by involving a total of m-number additional DOFs, which were added to each RC panel. Moreover, *ch* indicates the center of relative rotation between the two rigid beams at the top and bottom of the SFI-MVLEM element.

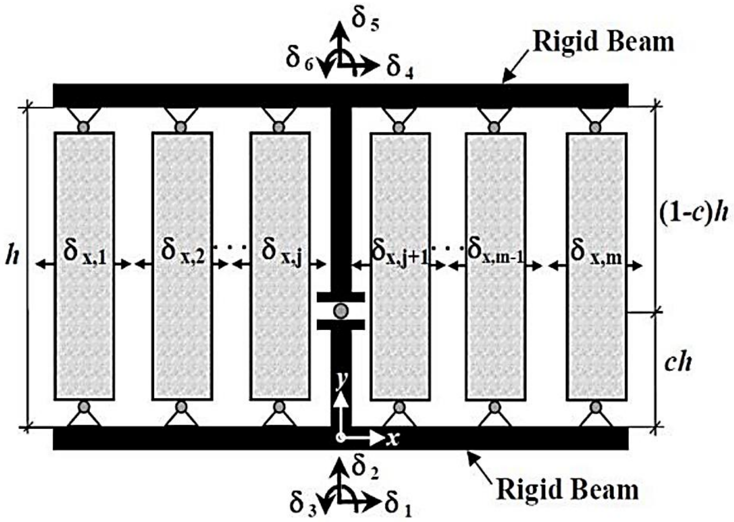
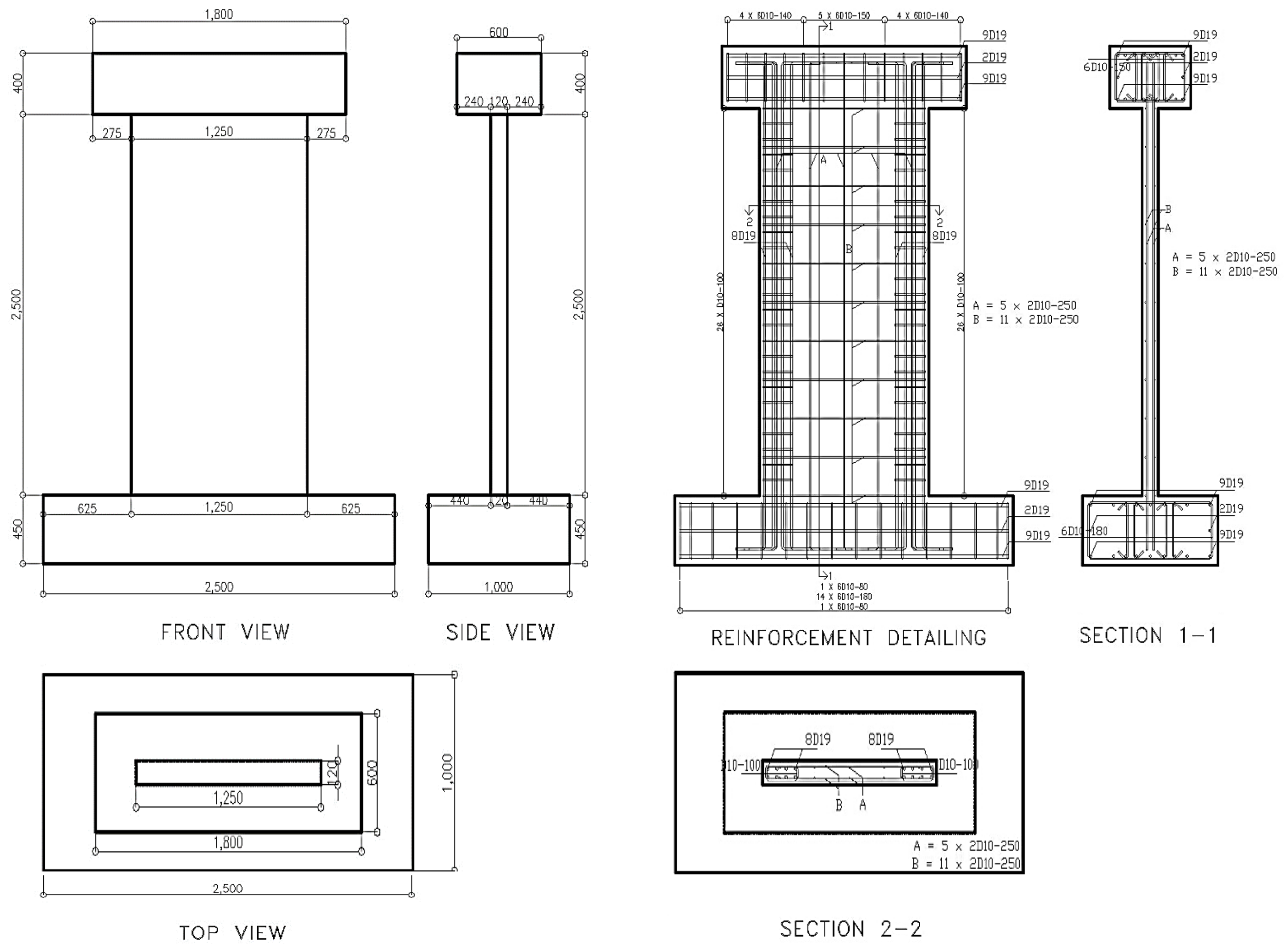


Figure 1. SFI-MVLEM Element [1]

OpenSees software was used to perform the analytical model. The program can be divided into three main steps. First of all, geometrical data of RC walls need to be identified including overall height, wall thickness, and boundary elements. Thereafter, the ratio of vertical, horizontal, and confining reinforcement embedded in the wall need to be specified. Finally, the last step was to identify other material parameters for concrete and steel, i.e. compressive strength (f'_c), modulus of elasticity (E_c), and ultimate strain (ε_c) for concrete and yield strength (f_y), modulus of elasticity (E_s), and strain hardening ratio (b) for steel reinforcement. For SFI-MVLEM element, by default, *Concrete CM* material derived from the model proposed by Chang and Mander [4] was selected to model the behavior of concrete. Moreover, a steel constitutive model, *Steel MPF*, proposed by Menegotto and Pinto [5], and further enhanced by Filippou et al. [6] was chosen to represent the behavior of steel reinforcement in the SFI-MVLEM element.

Laboratory Experiment

A rectangular RC structural wall was cast-in-place in the Structural Engineering Laboratory of Petra Christian University. The specimen has h_w/l_w of 2.0 and it was designed to fail in a flexural-shear mechanism by designing such that the flexural and shear strengths of the RC wall were similar. Reinforcement configuration for the RC wall consists of both vertical web reinforcement ratio (ρ_v) and horizontal web reinforcement ratio (ρ_h) of 0.52%, which satisfies the minimum requirement prescribed in ACI 318-14 code [7]. Steel bar flexural reinforcements of 8D19 were used at each of the boundary elements of the RC wall. The top and bottom beams in the specimen were designed to be stiff and strong enough to resist loadings without any significant deformation. The details of specimen's dimensions and reinforcement are shown in Figures 2a and 2b, respectively. The specimen was constructed using concrete with compressive strength of 25 MPa. Furthermore, Table 1 summarizes the mechanical properties of the reinforcing bars as determined from tensile tests.



(a) (b)

Figure 2. (a) Dimensions and (b) Reinforcement Details of the RC Wall Specimen

Table 1. Properties of Steel Bars

ϕ_d (mm)	f_y (MPa)	f_u (MPa)
D19	481.150	636.176
D10	463.483	651.027

Test Setup

The test setup (see Figure 3) simulated a cantilever RC structural wall with a fixed base and applied loadings at the top of the wall, in this case was the top beam. To simulate a fixed base, prestressing bars were used for clamping the bottom beam to the laboratory strong floor to prevent accidental sliding and overturning. Furthermore, restraining blocks were also utilized at both ends of the bottom beam to prevent unintended lateral movement. Two vertical hydraulic cylinders with capacity of 600 kN each and a spreader beam were used to generate static vertical loading to simulate gravity loading in the actual RC wall. To simulate earthquake cyclic loading, a double-acting horizontal hydraulic cylinder with capacity of 2000 kN and a displacement controller were used in this test setup.

Instrumentation

In the RC wall test, all displacements were measured using a series of linear variable displacement transformers (LVDTs) (see Figure 4a). The in-plane lateral displacement of the top beam was measured by two LVDTs attached at both ends of the top beam and the data would be used to plot the force-drift curve of the specimen. A set of LVDTs were put at the bottom beam to monitor horizontal movement and up-lift, if any. Moreover, a series of LVDTs mounted to the edges of the wall along the wall height were used to monitor flexural deformation. Two pairs of LVDTs attached diagonally to the wall web were used to monitor shear deformation. In addition, two LVDTs attached to the wall base were used to monitor sliding shear deformation.

Strains in the reinforcing bars were measured using strain gauges that were installed on the reinforcing bars at certain locations (see Figure 4b). For the main flexural reinforcement, four strain gauges were installed at different positions at wall base to investigate whether the bars were yielded during testing. For the web reinforcement, strain gauges were installed at several locations - bottom, middle, and top of the wall to investigate the strain in those bars.

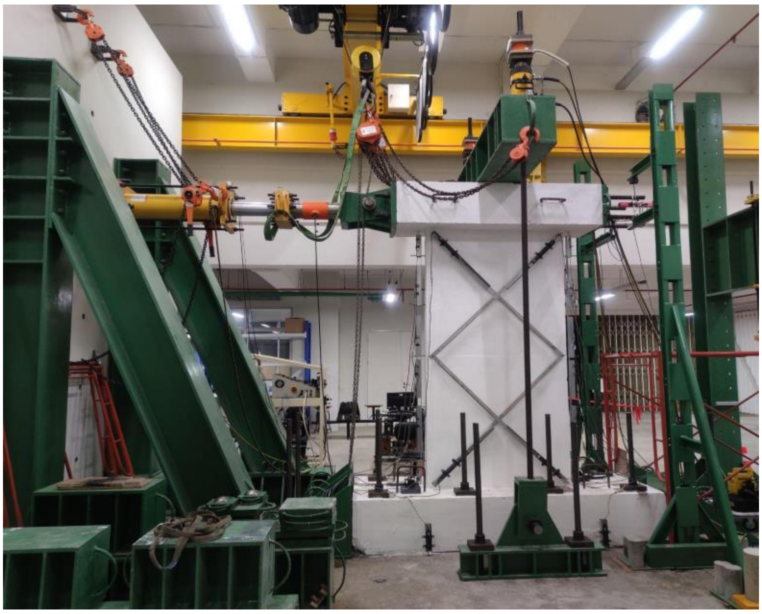
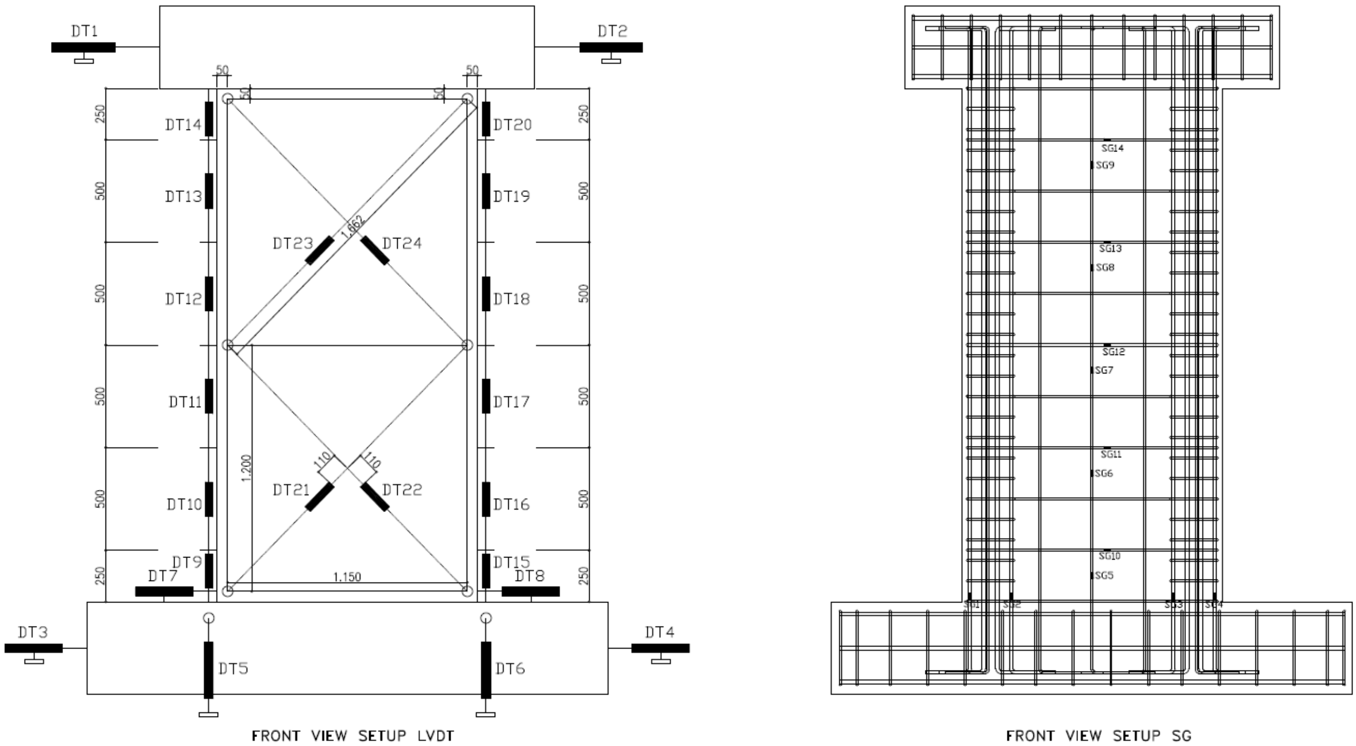


Figure 3. Overall Test Setup of the RC Wall Specimen



(a) (b)

Figure 4. (a) LVDT and (b) Strain Gauge Setup

Test Procedure

An axial loading of 5% of the concrete cylinder compressive strength was applied initially to the RC wall using the vertical hydraulic cylinders and spreader beam. This ratio was chosen since it was in the range of possible axial load in RC structural walls during an earthquake. The lateral load was applied in a cyclic manner using the horizontal hydraulic cylinder with displacement controller. The specimen was subjected to a loading history shown in Figure 5.

Each cycle has positive and negative drift amplitudes which were increased gradually in every odd cycle. The expected failure mode for the specimen was combination of flexure-shear mechanism with residual strength ratio about 0.40-0.75 of the peak value according to ASCE 41-17 [8]. At each peak amplitude, crack patterns were drawn to capture crack propagations. Displacements of the specimen and strains in reinforcing bars were recorded throughout testing.

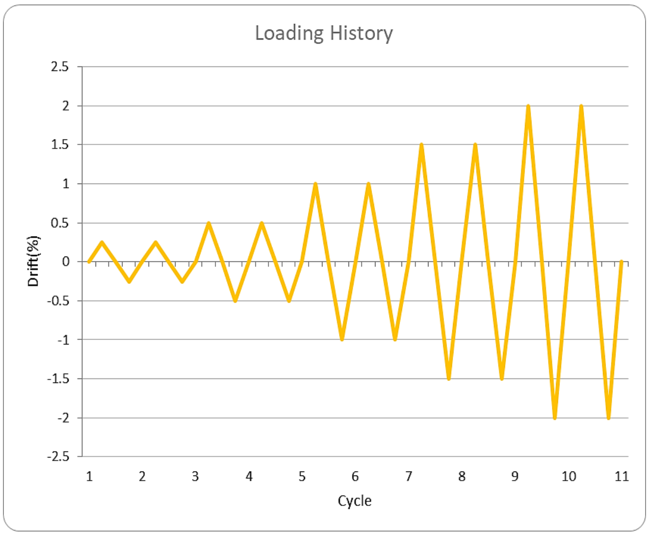


Figure 5. Experimental Loading History

Results and Discussion

This section exhibits the results of the RC wall laboratory testing and discusses the performance of the RC wall based on the regulating code provisions, ACI 318-14 [7] and ASCE 41-17 [8]. Furthermore, the outcomes from the experimental study will be compared to analytical results from OpenSees software using shear-flexure interaction multiple vertical line element model (SFI-MVLEM).

Crack Patterns and Force-Drift Relationship

The crack patterns propagation during testing is shown in Figure 6. For the crack propagation, diagonal cracks started to develop in the RC wall web from the second cycle onwards. In the boundary elements, significant horizontal cracks started to occur at the lower part of the wall in the fourth cycle. In the sixth cycle, crushing of the compression zone at the bottom of wall began to occur. The specimen reached its peak strength in the ninth cycle. Afterwards, the specimen showed a sudden reduction in performance after the occurrence of diagonal web crushing in the last cycle as shown in Figure 7. This sudden failure was also accompanied by concrete crushing at the bottom part of the compression boundary element.

The force-drift ratio curve obtained from the experiment is shown in Figure 8. The recorded maximum force and associated drift ratio are +513 kN and +2.00%, respectively. From the force-drift ratio curve, it can be seen that the specimen failed in an altered shear-flexural mechanism. Horizontal web reinforcements yielded first at drift of 1.00% and then followed by yielding of flexural reinforcements at drift of 1.50% which were shown by strain gauges data presented later in the paper. The failure mode in the altered shear-flexural failure mechanism was verified by other significant indications at the end of the testing as shown in Figure 7 previously. It was found that concrete at the lower extreme compression zone were crushed and it was accompanied by web crushing that occurred at the final cycle indicating combination of flexure-shear failure. Note that the specimen tested had similar strength in the positive and negative directions at the same cycle number. This finding means that the diagonal cracks and horizontal cracks that developed owing to loading in one direction did not affect the strength in the other direction as long as web crushing or lower boundary element crushing had not occurred. The same finding was reported by Teng and Chandra [9] in a previous study.

The experimental result shows that the RC wall specimen satisfies the criteria of ASCE 41-17 [8] for collapse prevention state of RC structural walls failing in flexure or shear. According to ASCE 41-17 [8], collapse prevention performance level means that the building is on the verge of partial or total collapse as a result of earthquake damage.

ASCE 41-17 [8] specifies a drift ratio of 2.00% as the limit for the collapse prevention performance level for RC structural walls. At this respective drift ratio, the residual strength of the specimen failing in flexure with shear stress ratio ($V/[t_w l_w \sqrt{f'_c}]$) above 0.5 (in MPa) has to be at least 40% of its peak strength while the residual strength of the specimen failing in shear has to be at least 20% of its peak strength. The specimen tested in this study has residual strength of 41% of its peak strength at drift level of 2.00% which is above the acceptance criteria of ASCE 41-17 [8] for the collapse prevention performance level.

Cycle No. -2; -0.25%; -186 kN

Cycle No. -4; -0.50%; -267 kN

Cycle No. -6; -1.00%; -372 kN

Cycle No. +9; +2.00%; +513 kN

Figure 6. Crack Patterns Propagation of the RC Wall Specimen during Testing. Note: +9; +2.00%; +513 kN indicates the ninth cycle in positive direction, drift ratio, and lateral force at the respective drift ratio. Positive direction is from left to right.

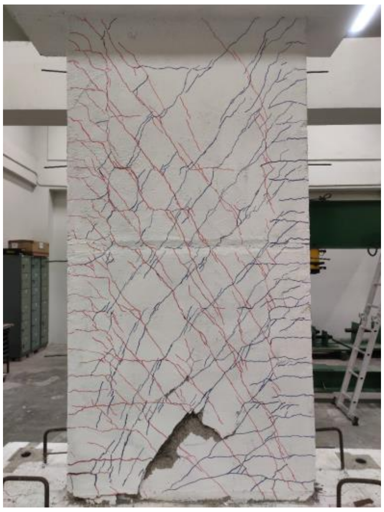


Figure 7. Final Crack Patterns of the Specimen at the End of Testing

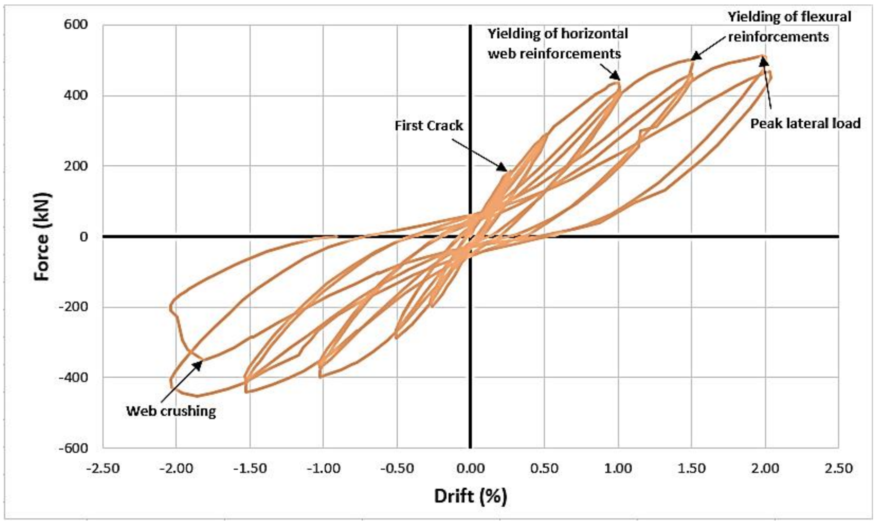


Figure 8. Force-Drift Ratio Curve of the Specimen

Lateral Deformations and Strains

Generally, the contribution of flexural deformation and shear deformation to the total wall drift ratio depends on h_w/l_w of the wall. Figure 9 shows the deformation components due to flexural, shear, and sliding shear in the RC wall specimen. For h_w/l_w of 2.0, the contribution of flexural deformation was dominant among the other deformation components at the early stages or before formation of major diagonal crack (drift ratio lower than 1.00%). For the next consecutive cycles, the contribution of shear deformation began to increase gradually until the specimen failed.

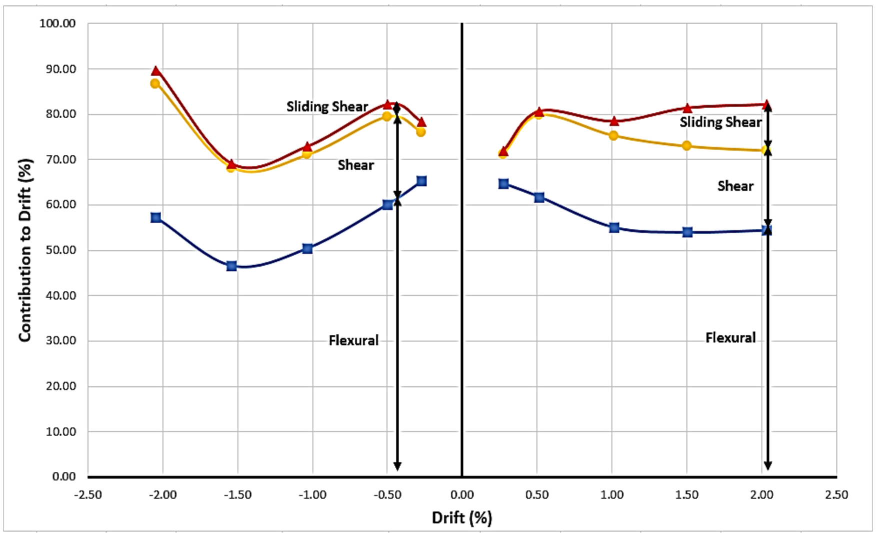


Figure 9. Contribution of Wall Deformation Components to Total Drift

The strain distributions of the vertical and horizontal reinforcement bars are shown in Figures 10 and 11. The strains were plotted from strain gauges installed at the bottom of vertical bars and from those installed at the middle of horizontal bars. These strains were obtained to observe whether the steel bars had yielded during testing at various stages. At the initial drift ratio of 0.25%, the strains in both vertical bars (Figure 10) and horizontal bars (Figure 11) were still low, except for the horizontal bar strain gauge at the height of 1.25 m, possibly caused by the development of diagonal cracks in the wall mid-height section at early cycles. At later testing stages, values of the strain in steel bars gradually increased with each increment of the drift ratio. Sudden increase in strain values was spotted at drift ratio of 1.00%, especially in the horizontal bar at the mid-height section where major diagonal cracks occurred. According to the strains plotted in Figures 10 and 11, it is shown that the horizontal bars along wall height yielded prior to the vertical bars at the bottom of wall which yielded at drift ratio of 1.50%. Subsequently, the strains in all bars continued to increase beyond the yield limit, until reaching maximum lateral force at drift ratio of 2.00%. At this point, both vertical and horizontal bars yielded beyond yield limit which indicates a combination of flexure-shear failure.

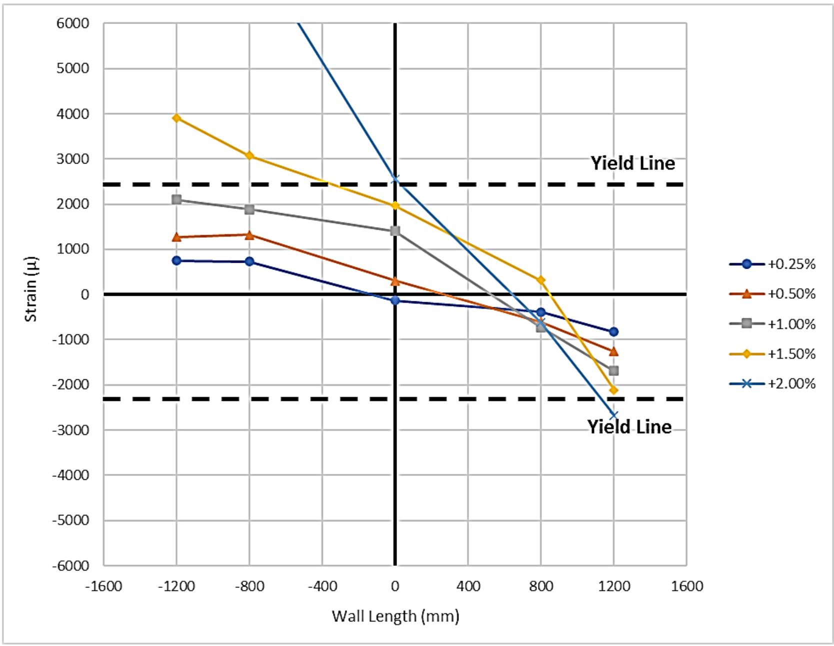


Figure 10. Strain in Vertical Bars at the Bottom of RC Wall Specimen at Various Drift Ratios

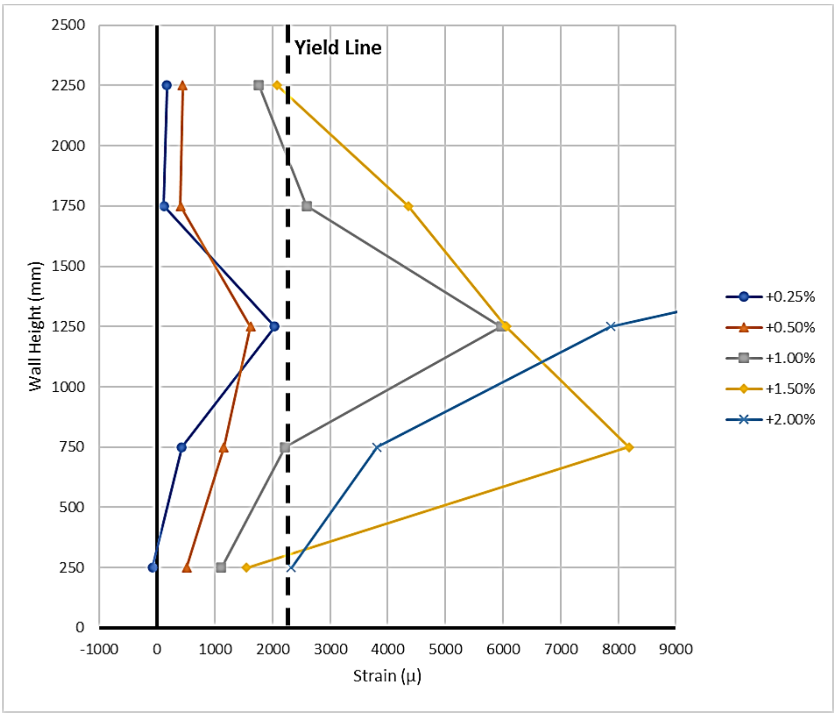


Figure 11. Strain in Horizontal Bars at the Middle of RC Wall Specimen at Various Drift Ratios

Comparison to Analytical Model and Building Code Provisions

Figure 12 shows the comparison of the RC wall specimen's hysteretic curve obtained from the experiment and from the analytical model. Visually, it can be observed that the model predicts higher overall stiffness of the wall with the presence of steeper gradient. Hence, the peak lateral force of the wall from the model occurred at 1.00% drift whereas from the experiment, the peak lateral force occurred at 2.00% drift. However, the model is able to predict well the peak lateral force of the specimen both in positive and negative directions. Furthermore, the model is also able to capture the strength degradation of the RC wall beyond 1.00% drift. Nevertheless, the model failed to capture significant strength degradation due to web crushing in the last cycle of loading. In addition, considering the shape of the overall force-drift curve, the model also has less pinching effect as compared to the experiment which this case was also reported in a previous study [3].

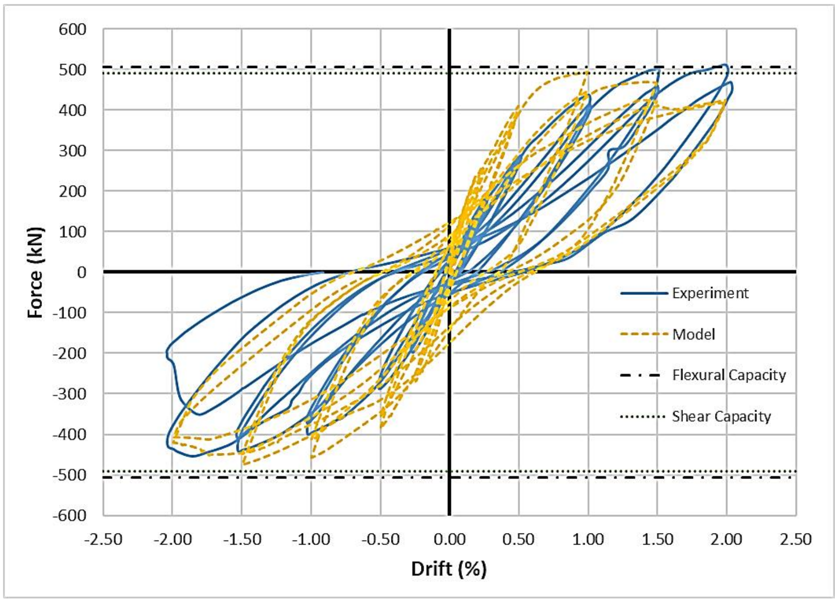


Figure 12. Comparison of RC Wall Specimen's Hysteretic Curves Obtained from the Experiment and Analytical Model

In accordance with the provisions of ACI 318-14 [7], the nominal flexural and shear strengths of the specimen are 506.37 kN and 491.53 kN, respectively. Thus, the experimental peak strength (513 kN) is slightly above the code predictions. From a previous study [10], it was concluded that building code provisions predict quite well the flexural strength of RC walls whereas they tend to underestimate the shear strength. Since these nominal values are very close to each other for the specimen tested in this study, and hence it was expected that the specimen would fail in a combination of flexure-shear failure. From a design perspective, ACI 318-14 [7] does not specifically require to calculate the design shear force of special RC walls based on the probable moment like in the case of special moment frames. Therefore, the nominal shear strength could be lower than the nominal flexure strength under certain circumstances. From this experiment, it was shown that web crushing could occur in the collapse prevention stage (2.00% drift) for such RC walls which is not desirable for ductile performance of special RC walls. This issue has been addressed in the newer version of ACI 318-19 [11] that the design shear force for special RC walls should be amplified to account for flexural overstrength at critical sections where yielding of flexural reinforcement may occur.

Conclusions

A comprehensive experimental study regarding the nonlinear SFI behavior was conducted in the Structural Engineering Laboratory of Petra Christian University. The specimen tested was a moderate RC wall having h_w/l_w of 2.0 and both ρv and ρh of 0.52%. In addition to the experiment, an analytical model using SFI-MVLEM element in OpenSees was performed to be compared to the experimental results. From the results, some concluding remarks can be listed as follows:

1. The RC wall specimen failed in altered shear-flexure mechanism. The flexure failure was indicated by crushing of the flexural compression zone whereas the shear failure was notified by web crushing that occurred almost concurrently.
2. The experimental result shows that the specimen meets the criteria of ASCE 41-17 [8] for collapse prevention stage. The specimen tested in this study has residual strength of 41% of its peak strength at drift level of 2.00% which is above the acceptance criteria of ASCE 41-17 [8].
3. The deformation of the specimen having h_w/l_w of 2.0 was contributed greatly by flexural deformation at the early stages (drift ratio below 1.00%). For the next consecutive cycles, the contribution of shear deformation increased gradually until the specimen failed.
4. At peak lateral load (drift ratio of 2.00%), both vertical and horizontal bars yielded beyond yield limit which indicates a combination of flexure-shear failure.
5. The analytical model from SFI-MVLEM element could predict reasonably well the hysteretic curve as well as the peak lateral load of the RC wall specimen tested in this study. However, the model failed to predict the web crushing failure that occurred in the last cycle. Moreover, the model also has less pinching effect as compared to the experiment which this case was also reported in a previous study [3].
6. The RC wall specimen tested in this study has very close nominal flexure and shear strengths according to ACI 318-14 [7]. Hence, a combination of flexure-shear failure might occur. From a design perspective, it is more

desirable to have a ductile flexural failure without web crushing which affects the performance of RC walls at collapse prevention stage. To address this issue, ACI 318-19 [11] has introduced an amplification factor for the design shear force of special RC walls.

Acknowledgements

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References

1. Kolozvari, K.I., *Analytical Modeling of Cyclic Shear-Flexure Interaction in Reinforced Concrete Structural Walls*, (Ph.D. Dissertation), University of California, Los Angeles, 2013.
2. Orakcal, K., Wallace, J.W., and Conte, J.P., Flexural Modeling of Reinforced Concrete Walls Model Attributes, *ACI Structural Journal*, 101(5), 2004, pp. 688–698.
3. Purnomo, J. and Chandra, J., Evaluation of a Reinforced Concrete Wall Macroscopic Model for Coupled Nonlinear Shear-Flexure Interaction Response, *Civil Engineering Dimension*, 20(1), 2018, pp. 41-50.
4. Chang, G.A. and Mander, J.B., *Seismic Energy Based Fatigue Damage Analysis of Bridge Columns: Part I-Evaluation of Seismic Capacity*, State University of New York, Buffalo, NY, NCEER-94-0006, 1994, pp. 222.
5. Menegotto, M. and Pinto, E., Method of Analysis for Cyclically Loaded Reinforced Concrete Plane Frames Including Changes in Geometry and Non-Elastic Behavior of Elements under Combined Normal Force and Bending, *Proceeding of IABSE Symposium on Resistance and Ultimate Deformability of Structures Acted on by Well Defined Repeated Loads*, 1973, pp. 15–22.
6. Filippou, F.C., Popov, E.P., and Bertero, V.V., *Effects of Bond Deterioration on Hysteretic Behavior of Reinforced Concrete Joints*, Earthquake Engineering Research Center, University of California, Berkeley, CA, UCB/EERC-83/19, 1983.
7. ACI Committee 318, *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*, American Concrete Institute, Farmington Hills, MI, 2014, 519 pp.
8. ASCE 41-17, *Seismic Evaluation and Upgrade of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia, 2017.
9. Teng, S. and Chandra, J., Cyclic Shear Behavior of High Strength Concrete Structural Walls, *ACI Structural Journal*, 113(6), 2016, pp. 1335-1345.
10. Chandra, J., Liu, Y., and Teng, S., Analytical Study on High Strength Concrete Shear Walls, *36th Conference on Our World in Concrete and Structures*, Singapore, 2011, pp. 221-230.
11. ACI Committee 318, *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*, American Concrete Institute, Farmington Hills, MI, 2019, 623.