

FLEXURAL AND SHEAR RESPONSES IN SLENDER RC SHEAR WALLS

Leonardo M. MASSONE¹, Kutay ORAKCAL² and John W. WALLACE³

SUMMARY

A review of experimental data obtained from slender reinforced concrete wall tests was conducted to assess the contributions of flexural and shear deformations to inelastic lateral displacements. Evaluation of the test results indicates a significant coupling between inelastic flexural and shear deformations, even at shear force levels of about one-half of the nominal shear strength of the specimens. Common column-type analytical models for walls, such as the Multiple-Vertical-Line-Element-Model (MVLEM) incorporate uncoupled deformation components for shear and flexure, which is inconsistent with the experimental observations. Therefore, the MVLE model was modified to allow coupling of the flexural and shear deformation components based on membrane behavior, via adaptation of the Modified Compression Field Theory (MCFT). Results from two different wall tests, for a slender wall and a short wall, were compared with the model results. In the case of the slender wall, experimental and model results compare favorably, although shear deformations are underestimated. The overall lateral load versus top displacement response was captured for the short wall test, although additional studies are needed to address observed discrepancies.

INTRODUCTION

Reinforced concrete structural walls are commonly used to resist the actions imposed on buildings due to earthquake ground motions. To resist such actions, properly proportioned and detailed slender walls are designed to yield in flexure, and to undergo large flexural deformations without loss of lateral load capacity. Therefore, the ability to model the cyclic behavior and failure modes of structural walls is an important aspect of engineering design, particularly as the profession moves forward with design and evaluation approaches that emphasize performance based seismic design.

Recent research has shown that the lateral force versus deformation response of slender walls in flexure can be captured reasonably well using simple analytical models (e.g., Thomsen [1]), and improved predictions can be obtained using more detailed models (e.g., Orakcal [2]). However, such models usually consider uncoupled shear and flexural responses, which is inconsistent with observations, even for relatively slender walls (Massone [3]).

¹ Graduate Research Assistant, Dept of Civil & Environ. Engineering, University of California Los Angeles, USA

² Graduate Research Assistant, Dept of Civil & Environ. Engineering, University of California Los Angeles, USA

³ Associate Professor, Department of Civil & Environ. Engineering, University of California Los Angeles, USA

Accordingly, this paper provides a review of experimental data obtained from select slender reinforced concrete wall tests for determining the relative contributions of inelastic flexural and shear deformations in wall lateral displacements, and summarizes a proposed modeling approach to incorporate coupling of wall flexural and shear responses. Preliminary model results are presented and compared with test results obtained from tests on a slender and short wall to evaluate the modeling approach.

EXPERIMENTAL EVIDENCE OF FLEXURE-SHEAR INTERACTION

Deformations associated with flexure and shear were determined for two, well-instrumented, approximately quarter-scale slender wall specimens with rectangular cross sections (RW2 of Thomsen [1], and SRCW1 of Sayre [4]) to assess their relative contributions to displacement responses. An overview of this effort is provided in the following section; additional information is available in the paper by Massone [3].

Test overview

The two slender wall specimens used to assess the relative deformations associated with shear and flexure were proportioned and detailed using capacity design and displacement-based design approaches, respectively. The wall specimens were tested in an upright position. An axial load of approximately $0.10A_g f'_c$ was applied to the wall specimens using hydraulic jacks mounted on top of the load transfer assembly. The axial stress was held constant throughout the duration of each test. Cyclic lateral displacements were applied to the walls by a hydraulic actuator mounted horizontally to a reaction wall at 12.5 feet (3.81 m) above the base of specimen RW2, and to a reaction frame at 16 ft (4.88 m) above the base of specimen SRCW1.

Specimen RW2 was 12 ft (3.66 m) tall and 4 in. (102 mm) thick, with a web length of 4 ft (1.22 m). The longitudinal reinforcement at wall boundaries consisted of 8 - #3 ($A_b = 0.11 \text{ in}^2 = 71 \text{ mm}^2$) bars, whereas web reinforcement consisted of two curtains of deformed #2 ($A_b = 0.049 \text{ in}^2 = 32 \text{ mm}^2$) bars placed horizontally and vertically, with a spacing of 7.5 in. (189 mm) on center. Specimen SRCW1 was a structural steel reinforcement at wall boundaries consisted of a W6x9 section ($A_{b1} = 2.68 \text{ in}^2 = 1729 \text{ mm}^2$) surrounded by 8 - #4 ($A_{b2} = 1.60 \text{ in}^2 = 1032 \text{ mm}^2$) bars. The web reinforcement consisted of two curtains of horizontal and vertical #3 bars with a spacing of 6 in. (152 mm) on center.

Design concrete compressive strengths were 4,000 psi (27.6 MPa) for specimen RW2 and 5000 psi (345 MPa) for specimen SRCW1. Grade 60 (414 MPa) bars were used for longitudinal and web reinforcement of the specimens. For SRCW1, A572, Grade 50 (345 MPa) W6x9 sections were used.

Instrumentation was used to measure displacements, loads, and strains at critical locations for each wall specimen. Wire potentiometers (WPs) were mounted to a rigid steel reference frame to measure lateral displacements along the height of the wall (Figure 1). Linear potentiometers (LPs) were mounted horizontally and vertically on the wall foundation to measure any horizontal slip of the pedestal along the strong floor as well as rotations caused by uplift of the pedestal from the strong floor. Measurements from the WPs used to record lateral displacements were corrected to remove the contribution of pedestal slip and rotation to lateral displacements.

Axial (vertical) displacements at the wall boundaries were measured using two WPs mounted directly to the wall ends. These measurements were used to calculate wall story rotations by dividing the difference in relative axial displacements by the distance between the potentiometers. Shear deformations within the first two (RW2) and three (SRCW1) levels of the wall specimens were determined using

measurements from WPs placed diagonally on the walls, in an "X" configuration (Figure 1).

Linear variable differential transducers (LVDTs), oriented vertically over the wall length just above the wall – pedestal interface, were used to obtained average axial strain and allow for the determination of section curvature. The strains in the reinforcing steel also were measured through the use of strain gauges near the wall base and at other locations (Thomsen [1] and Sayre [4]).



Figure 1 General instrument configuration. RW2 and SRCW1.

Determination of flexural and shear deformations

A common approach used to determine average (story) shear deformations for shear wall tests is to use measurements from displacement gauges placed diagonally in an "X" configuration (e.g., Thomsen [1], see Figure 2). However, as shown by Massone [3], measurements obtained from the diagonal gauges are influenced by flexural deformations if the center of rotation of the story does not coincide with the geometric center of the story height. The average shear deformations calculated using an "X" configuration may result in over-estimation of the shear deformations if not corrected to account for flexural deformations.



Figure 2 Uncorrected shear deformation measure at wall base.

Flexural deformation

To determine the contribution of the flexural deformations to the lateral displacement at the top of a story, the location of the centroid of curvature distribution (center of rotation) of the story must be estimated. The flexural displacement at the top of the first story, for a given curvature distribution is calculated as:

 $Uf = \underline{\alpha} \underline{\theta} h$ (Eq-1)

where $\underline{\theta}$ is rotation over story level, h is the story height, and $\underline{\alpha}$ is the relative distance from the top of the first story to the centroid of the curvature distribution. In this study, a value of $\underline{\alpha} = 0.67$ was used, which is consistent with prior research (Thomsen [1]).

Shear deformation: Corrected X configuration

As proposed by Massone [3], the uncorrected story shear displacement measured through the use of the "X" configuration of WPs and the story flexural displacement measured using the vertical WPs at the wall boundaries can be used in combination to obtain a "corrected" average story shear displacement (Us_Xcorrected) as:

Us_Xcorrected = Us_Xoriginal +
$$\left(\frac{1}{2} - \underline{\alpha}\right) \cdot \underline{\theta} \cdot \mathbf{h}$$
 (Eq-2)

where Us_Xoriginal obtained using only the diagonal WP (X configuration) measurements would give a biased estimation of the story shear displacement, due to contribution of flexural deformations.

Force versus displacement relations - Shear and Flexure

Using the methodology described above to separate the contributions of shear and flexural deformations in measured wall displacements, lateral load versus top and story deformation relations can be determined. Figure 3 plots the applied story shear force versus measured flexural (Figure 3a) and shear (Figure 3b) displacements within the first and second stories of specimen RW2. Figure 4 plots the same relationships for the first through third stories of specimen SRCW1. Relations derived from the experimental data are compared to analytical results for linear elastic analyses for a "fully-cracked" section stiffness for flexure and an elastic shear stiffness. The cracked section stiffness is obtained from a sectional analysis, as a secant stiffness to the point of first yield of reinforcement. For simplicity, a linear elastic shear stiffness is assumed.



Results for specimen RW2 are presented in Figure 3. At the second story level of specimen RW2, shear displacements were evaluated using only the diagonal potentiometer (X configuration) measurements without correction given in Eq-2, because vertical potentiometers were not provided during testing along the second story height (Figure 1). The story shear force versus flexural displacement relationships for specimen RW2 (Figure 3a) reveal that: (1) the cracked stiffness obtained from a moment versus curvature analysis approximates the effective stiffness prior to yield reasonably well, (2) yielding of flexural reinforcement occurs at a lateral load close to that associated with the lateral load to reach the wall nominal moment (29.4 kips = 131 kN), and (3) yielding occurs primarily in the first two levels. The story shear force versus shear displacement relationships (Figure 3b) reveal that: (1) inelastic shear behavior occurred in the first story despite a nominal shear capacity ($V_n = 62$ kips; 276 kN) of approximately twice the applied story shear (~30 kips = 133 kN), (2) inelastic deformations were limited to essentially the first story, and (3) the elastic shear stiffness approximately represents the measured shear stiffness in regions where flexural yielding was not observed (i.e., the second level). The observed results clearly demonstrate coupling of wall inelastic shear and flexural responses; inelastic flexural deformations appear to have led simultaneously to inelastic shear deformations.

Similar relationships for specimen SRCW1 are presented in Figure 4. Vertical and diagonal wire potentiometers were provided along the bottom three stories of specimen SRCW1 (Figure 1); therefore, the shear displacement measurements at the first, second and third story levels of SRCW1 were corrected based on Eq-2. The story shear force versus flexural displacement relationships in Figure 4(a) reveal findings similar to those for specimen RW2, except that slip between the structural steel section and the concrete appears to have contributed significantly to a loss of stiffness within the first story of specimen SRCW1. The story shear force versus shear displacement relationships (Figure 4b) reveal that inelastic shear behavior was experienced within the first and second stories of the wall, despite a nominal shear capacity ($V_n = 146.3$ kips; 651 kN) of approximately twice the applied story shear (~70 kips = 310 kN). The observed inelastic shear deformations in the first story level, and also to a lesser degree in the second

story level, clearly demonstrate the coupling of inelastic flexural and shear deformations. As well, the flexural and shear force-deformation relationships reach yielding at approximately the same time. This behavior, which has also been observed in prior studies (e.g., Takayanagi [5]), has been established for the tests evaluated using an unbiased methodology for evaluating shear deformations, and the results verified through the use of redundant measurements for multiple test specimens (Massone [3]).



Figure 4 Story deformations: SRCW1.

FLEXURE AND SHEAR INTERACTION MODEL FOR COLUMN-TYPE ELEMENTS

Several analytical models have been proposed to consider coupling between flexural and shear components of RC wall response. One basic approach, proposed by Takayanagi [5], involves adopting a shear force - displacement relationship with shear yielding defined at the same lateral load level as that required to reach flexural yield. Another methodology involves implementing the finite element method (FEM) together with the so-called Modified Compression Field Theory (MCFT, Vecchio [6]) to model reinforced concrete membrane behavior. An approach based on adopting this idea for a fiber model, was proposed by Pentrangeli [7] to couple wall shear response with flexural and axial responses.

The analytical model proposed in this study is based on applying the methodology developed by Pentrangeli [7], to a Multiple-Vertical-Line-Element wall model (MVLEM, see Orakcal [2]). Model results are compared to experimental data from tests on slender and short RC walls to investigate the validity of the model.

Base model: Multiple Vertical Line Element Model (MVLEM)

The Multiple Vertical Line Element Model (MVLEM) resembles a two-dimensional fiber model, simplified such that element rotations (curvatures) are concentrated at the center of rotation defined for each element, instead of using a displacement field (e.g., a linear curvature distribution) as in a generic

displacement-based fiber model or finite element model implementation. A structural wall is modeled as a stack of MVLE's, which are placed one upon the other (Figure 5b). The axial and flexural response of each MVLE is simulated by a series of uniaxial elements (or macro-fibers) connected to infinitely rigid beams at the top and bottom (e.g., floor) levels (Figure 5a). The plane-sections-remain-plane assumption is applied to calculate the strain level in each uniaxial element according to values of displacement and rotation at the degrees of freedom of each MVLE. The stiffness properties and force-displacement relationships of the uniaxial elements are defined according to uniaxial constitutive stress-strain relationships implemented in the model for concrete and steel and the tributary area assigned to each uniaxial element (Figure 6). A horizontal spring placed at the center of rotation (at height ch) of each MVLE, with a prescribed nonlinear force-deformation behavior, simulates the shear response of the element.



Figure 5 Multiple-Vertical-Line-Element (MVLE) Model (Orakcal [2]).



Figure 6 Discretization of wall cross section (Orakcal [2]).

Shear - Flexure Interaction

In MVLE models, flexural and shear modes of deformation are uncoupled (i.e., flexural deformations do not affect shear strength or deformation). As noted previously in this paper, experimental results indicate that inelastic shear deformations are observed simultaneously with inelastic axial – flexural deformations, even when the overall wall response is expected to be dominated by flexure (e.g., when the nominal shear strength of the wall exceeds significantly the lateral load required to produce flexural yielding). Therefore, for a MVLEM, where deformations due to flexure and shear are uncoupled, linear elastic shear response will be predicted for slender walls, which is inconsistent with experimental results

Constitutive Material Models

A uniaxial monotonic stress-strain relationship, such as a simple bilinear relationship or the stress - strain model proposed by Menegotto [8] can be used to model the cyclic behavior of the reinforcing steel. To obtain a reliable model for panel (membrane) behavior, the biaxial constitutive relationship adopted for concrete should consider the effects of compression softening (reduction in principal compressive stress due to the application of tensile stress in the orthogonal direction), tension - stiffening (post - peak tensile stresses in concrete due to bond stresses between reinforcing steel and concrete between cracks), and possibly the reduction in the post – peak concrete tensile stress capacity to account for loss of aggregate interlock capacity. In this study, the uniaxial constitutive model by Menegotto [8] for steel and the biaxial constitutive model by Vecchio [6] for concrete were implemented (Figure 7).



Figure 7 Constitutive material models. (a) concrete, and (b) steel.

Modeling RC Panel Behavior

The Modified Compression Field Theory (MCFT) is used to model a RC panel element with membrane actions acting on it, i.e., uniform normal and shear stresses are applied in the in-plane direction. The constitutive stress - strain models for materials are applied along the principal directions of the strain field (i.e., principal strain directions 1 and 2), to obtain the stress field associated with the principal directions. It is assumed that the principal stress and strain directions coincide (Vecchio [6]).

As established, the MCFT approach applies to a two dimensional panel element under membrane actions, and is appropriate for implementation into a 2D finite element formulation. In this study, the intent is to develop a simplified version of the model that adopts an approach similar to that adopted for a column-type element; specifically, for the uniaxial elements of the MVLE as described in the previous section.

Proposed Model

In the procedure description, the uniaxial elements located within each MVLE (also called strips or fibers) are denoted by element (i) and the MVLEs are denoted by element (j):

1. The deformations or strains within the components of each MVLE (j) are determined from the six

prescribed degrees of freedom, (u_x , u_y and θ at both ends) shown in Figure 8. Assuming that the shear strain is uniform along the section and that plane sections remain plane, the axial strain (ε_y) and shear distortion (γ_{xy}) components of the strain field can be calculated for the entire section (for all the strips (i)) based on the prescribed degrees of freedom selected for the current analysis step. Accordingly, each strip (i) (Figure 8) has two input variables, axial strain (ε_y) and shear distortion (γ_{xy}), based on element (j) deformations. The transverse strain within each strip (ε_x) is initially estimated to complete the definition of the strain field, allowing stresses and forces to be determined from the constitutive material relationships and geometric properties (dimensions and steel area) for each strip. The output variables associated with the input strains ε_y and γ_{xy} are the axial stress, σ_y , and the shear stress, τ_{xy} , for each strip.



Figure 8 Trial displacement state at section (j) for prescribed top displacement.

- 2. A numerical procedure (Newton's method) is employed to linearize and iterate on the unknown quantity ε_x (horizontal stress in each strip i), to achieve horizontal equilibrium for a given σ_x within each strip. Due to a lack of information and as an initial approximation, the horizontal stress σ_x within each strip was assumed to be equal to zero, which is consistent with the boundary conditions of a wall with no transverse loads applied over its height. The orientation of principal strain (or stress), α , is used as an iterative parameter (instead of ε_x) for convenience
 - a. For a trial value of principal orientation angle (α), together with the prescribed values of axial strain (ε_y) and shear distortion (γ_{xy}), the strain field (horizontal strain ε_x , and the principal strains ε_1 and ε_2) is defined for each strip (i). It is assumed that the same orientation angle (α) applies for the principal directions of both the strain (ε_1 , ε_2) and the stress fields (σ_{c1} , σ_{c2}). Using the constitutive material relationships implemented for concrete and steel, and compatible strains for the two materials (assuming perfect bond), the stresses in concrete along the principal directions and stresses in steel along the vertical and horizontal directions are determined. As noted earlier, a uniaxial stress-strain model is used for the reinforcing steel; therefore, stresses in steel are calculated in horizontal and vertical directions (based on ε_x and ε_y , based on the assumption that reinforcement is provided in the vertical and horizontal directions).

b. Stresses in concrete are transformed from the principal directions to the x-y directions resulting concrete forces and superimposed with the forces in the reinforcement based on the concrete and steel areas within each strip. The resultant gives average normal and shear stresses in the strip (i) as:

$$\tau_{xy} = -\frac{\sigma_{c1} - \sigma_{c2}}{2} \cdot \sin(2 \cdot \alpha)$$
 (Eq-3)

$$\sigma_{x} = \sigma_{cx} + \rho_{x} \cdot \sigma_{sx} = \frac{\sigma_{c1} + \sigma_{c2}}{2} - \frac{\sigma_{c1} - \sigma_{c2}}{2} \cdot \cos(2 \cdot \alpha) + \rho_{x} \cdot \sigma_{sx}$$
(Eq-4)

$$\sigma_{y} = \sigma_{cy} + \rho_{y} \cdot \sigma_{sy} = \frac{(\sigma_{c1} + \sigma_{c2})}{2} + \frac{(\sigma_{c1} - \sigma_{c2})}{2} \cdot \cos(2 \cdot \alpha) + \rho_{y} \cdot \sigma_{sy}$$
(Eq-5)

c. Equilibrium is checked in the horizontal direction (σ_x) for each strip (i), until equilibrium is achieved for the specified angle α .

3. Once horizontal equilibrium is achieved (within a specified tolerance) within each strip, vertical stresses in the strips are assembled to determine the total resisting axial force and bending moment of each MVLE, whereas the shear forces in the strips are assembled to determine the total resisting shear force of the element.

4. Consequently, global equilibrium is checked for the overall MVLE wall model by comparing the applied and resisting forces, and global iterations are performed on the model degrees of freedom as described in Orakcal [2] until global equilibrium is satisfied.

An important aspect of the proposed approach involves the solution strategy. In the procedure described above, it is assumed that for a prescribed deformation configuration for each MVLE (i.e., for prescribed displacements at the element degrees of freedom), an internal iteration is required to solve for the principal direction angle (α), until horizontal stress equilibrium (e.g., $\sigma_x=0$) is reached at each strip. Another way to approach the same problem is to define the angle (α) for each strip as an additional element degree of freedom (in addition to the six degrees of freedom already defined). In this case, the internal iterations required to solve for α within each strip (Step 2 in the procedure) would be redundant. However, the addition of another external dof would increase the size of the stiffness matrix defined for each MVLE. The proposed procedure may require more iterations to converge compared with the case when the internal variable (α -value) for each strip is defined as an additional degree of freedom at the element level. However, the proposed approach was selected primarily because it provides a more general format, that is, it is likely to be easier to implement into existing elements (e.g., fiber) or programs without significant modifications.

PRELIMINARY MODEL RESULTS

Analytical results obtained with the proposed approach for monotonic loading are compared with test results for two reinforced concrete wall tests. The two specimens selected for study represent relatively extreme cases of wall behavior, a slender wall with response governed by flexure (RW2) and a short wall with response governed by shear (H13). Therefore, they provide good test cases to examine the model performance.

Specimen RW2, described earlier, is a slender wall with a shear aspect ratio of three (M/Vl_w) , and a nominal shear strength approximately twice that of the lateral load required to reach flexural yield. Specimen H13 tested by Hidalgo [9] is a short wall $(M/Vl_w = 0.5)$ with fixed-fixed boundary conditions at top and bottom of the specimen with a lateral load applied at the top level. Specimen H13 is 55.1 in. (1.4 m) tall and 3.9 in. (0.1 m) thick, with web length of 55.1 in. (1.4 m). Boundary vertical steel has a total

area of 1.24 in^2 (8 cm²) at each boundary. The distributed web reinforcement has a steel area ratio of 0.26% in both vertical and horizontal directions. The compressive strength of concrete used in construction of the H13 is 2.63 ksi (18.1 MPa), and the steel yield stress is 53.7 ksi (370 MPa). No axial load is applied on the specimen. The geometry of the specimen and the relatively large amount of boundary vertical reinforcement resulted in a response governed by shear (Hidalgo [9]).

The analytical model consists of 8 MVLEs stacked on top of each other, with 8 uniaxial elements defined along the length of the wall for each MVLE. The center of rotation of the MVLEs was defined at a value of c=0.4 for RW2, and a value of c=0.5 for H13. Three analysis cases are presented in Table 1.

| | Table 1: | Modeling Cases | |
|---|--------------------------|--|---------|
| | Shear-flexure behavior | f _{cr} , psi (MPa) | Case No |
| 1 | Uncoupled – Linear Shear | $f_{cr} = 4\sqrt{f'_c} \left(1/3\sqrt{f'_c}\right)$ | 1(a) |
| 2 | Coupled | $f_{cr} = 4\sqrt{f'_{c}} \left(1/3\sqrt{f'_{c}}\right)$ | 2(a) |
| | | $f_{cr} = 0$ | 2(b) |

Note: f_{cr} is the tensile strength of the concrete, and f'c is the compressive strength of the concrete.

Analyses 1(a) and 2(a) include tensile strength of concrete, whereas 2(b) excludes the contribution of concrete tensile strength (f_{cr}). For specimen RW2 all three cases (1a, 1b, 2b) were evaluated. For specimen H13, only the case defined as 2(a) (coupled model and expected materials properties) was evaluated.

Slender wall test – RW2

Figure 9 compares the analytical and experimental lateral load – top displacement responses for the slender wall. As observed in the figure, a better prediction of the lateral load – top displacement relation is obtained for case 2(b) (proposed shear – flexure coupling model neglecting tensile strength of concrete). For the same material properties (cases 1(a) and 2(a)), the lateral load – top displacement responses are not very different for linear elastic shear response or for coupling of the flexural and shear responses, since the overall response of the wall is primarily flexural. Figure 10 compares analytical and experimental lateral load – "flexural" displacement responses at the first and second story levels. The best correlation is obtained for the coupled shear and flexure response with zero concrete tensile strength. (case 2(b)). Figure 11 compares analytical and experimental lateral load – "shear" displacement responses at the first and second story levels. Again, the best correlation is obtained for the coupled shear and flexure response displacement responses at the first and second story levels. Again, the best correlation is obtained for the coupled shear and flexure response with zero concrete tensile strength (case 2(b)). Results presented are preliminary, and additional studies for improved material relations and to incorporate cyclic behavior are underway.



Figure 9 Load-Top displacement relation.



Figure 10 Load-Flexural displacement curve. (a) 1st and (b) 2nd story.



Figure 11 Load-Shear displacement curve. (a) 1st and (b) 2nd story.

Results of analyses that consider coupling of the shear and flexural responses (cases 2(a) and 2(b)), yield inelastic shear and flexural displacements at various story levels; however, inelastic shear deformations are concentrated within the bottom story of the wall, where large inelastic flexural deformations are experienced. Although discrepancies are observed between analysis and test results, the analytical model is successful in representing the behavior associated with the coupling of shear and flexural responses. Refinement of the analytical model to incorporate updated material relationships, alternative strain and stress conditions (presently uniform shear strains and zero horizontal stresses along wall length), as well as cyclic behavior, may result in improved response predictions.

Short wall test - H13

The proposed analytical model can be extended to medium-height and short walls as long as the adopted material laws are representative of the physical behavior, and strain and stress conditions adopted in the model (uniform shear strains and zero horizontal stresses) are refined. In such cases, nonlinear shear deformations govern the overall response of the wall. Preliminary model results are compared with test results.

For specimen H13, only the analytical lateral load – top displacement relationship was compared with experimental results, since no other experimental data were available. Figure 12 plots the analytical and experimental lateral load – top displacement responses. As observed in Figure 12, the model proposed model captures general trends observed in the test of H13, and produces results that are substantially better than results produced with a linear (uncoupled) shear model; however, significant discrepancies are observed between experimental and analytical results. In particular, significant discrepancies in the responses are noted after concrete cracking (at a lateral load of approximately 35 kips (155 kN)). Better correlation might be possible for more refined modeling options. This is an area of current research by the authors.



Figure 12 Load-Top displacement relation.

CONCLUSIONS

Test results for reinforced concrete and structural steel reinforced concrete walls were investigated to establish force deformation relations at various story levels. The deformations were disaggregated into flexural and shear components via a procedure described. The results showed that inelastic shear and flexural deformations initiated simultaneously, at essentially the same level of applied lateral top load and displacement, despite wall nominal shear strengths of approximately twice the lateral force required to initiate flexural yielding. The findings indicate the presence of coupling between inelastic flexural deformations and inelastic shear deformations. The experimental results investigated were from tests on slender walls with responses dominated by flexural behavior; however, similar findings would be expected for shorter walls with responses governed by shear.

An analytical model that couples wall flexural and shear responses was proposed. The model incorporates RC panel behavior described by the Modified Compression Field Theory (MCFT) into the fiber-based Multiple Vertical Line Element Model (MVLEM). Model results were compared with selected test results for a slender and a short wall. In the case of the slender wall, good correlation between analytical and experimental results was observed. The inelastic shear deformations of the slender wall were somewhat underestimated; however, shear yielding and nonlinear shear behavior were successfully represented. Model results for the short wall captured the general behavior observed in the experiment; however, more significant discrepancies between the analytical and experimental responses were observed. The model results are promising; ongoing work will focus on refinement of the analytical model and implementation of the model into a widely available analysis platform.

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