

NONLINEAR MODELING OF RC AND SRC STRUCTURAL WALLS

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ABSTRACT

Prediction of the inelastic response of RC and SRC walls and wall systems requires a reliable modeling approach that includes important material and response parameters (e.g., confinement, slip, nonlinear shear behavior, neutral axis migration). Although various macroscopic models have been proposed to evaluate the response of RC walls, the models are not available in commonly used analysis programs. A research project was undertaken at UCLA to investigate and to improve on an effective modeling approach for the reliable prediction of the inelastic response of RC and SRC walls that incorporates refined constitutive laws and various response parameters. The Multi-Component-in-Parallel Model is being calibrated and updated using extensive experimental data on RC and SRC walls, and will be implemented into a widely available analytical platform. The objective of this paper is to summarize the ongoing analytical project and to discuss the effectiveness and reliability of the model for RC walls. Model calibration with experimental data and investigation of model sensitivity to modeling parameters are emphasized.

Introduction

Reinforced concrete (RC) and steel reinforced concrete (SRC) structural walls are effective for resisting lateral loads imposed by wind or earthquakes. The walls provide substantial lateral strength and stiffness, as well as the inelastic deformation capacity needed to meet the demands of the earthquake ground motions. Extensive research, both analytical and experimental, has been carried out to study the behavior of both isolated and coupled walls and of frame-wall systems. In order to predict the inelastic response of such structural systems under seismic loads, the hysteretic behavior of the structural members and their interaction should be accurately described by reliable analytical models.

Analytical modeling of the inelastic response of structural walls can be accomplished by using either a microscopic finite element model based on a detailed interpretation of the local behavior, or by using a macroscopic model based on capturing overall behavior with reasonable accuracy. Although the finite element method provides a powerful tool, due to the lack of

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completely reliable basic models and the complexities involved in the analysis and interpretation, difficulties may arise in the implementation and interpretation of the microscopic models. Macroscopic models, on the other hand, are based on a simplified idealization and their validity is restricted to the conditions upon which the derivation of the model is based. In general, relative simplicity and reasonable accuracy are emphasized as prerequisites of a wall model for use in design and evaluation of wall and frame-wall structural systems.

Although suitable analytical models have been proposed for realistic and practical prediction of the hysteretic behavior of RC beam members, a reliable model for a practical analysis of RC structural walls is not available in commonly used analysis programs. Use of a beam-column element at the wall centroidal axis is a commonly used modeling approach (Figure 1). In this case, a column is used to model properties of the wall. The column is commonly located at the wall centerline with rigid links used on girders. The main limitation of a beam-column model lies in the assumption that rotations occur about the centroidal axis of the wall. Thus, important features of the observed behavior (i.e., variation of the neutral axis along the wall cross section during loading and unloading, rocking of the wall, etc.) are disregarded and the resulting effects on the structural system (e.g., outriggering interaction with the frame surrounding the wall) are not properly considered. Various macroscopic models have been proposed in order to capture these experimentally observed features in predicting the inelastic response of RC structural walls. As a result of extensive studies, the Multi-Component-in-Parallel Model (MCPM, also referred to as Multiple-Vertical-Line-Element Model, MVLEM) proposed by Volcano, Bertero and Colotti (1988), has been shown to successfully balance the simplicity of a macroscopic model and the refinements of a microscopic model. The model captures important features (e.g., refined material behavior, shifting of the neutral axis, rocking of the wall, the effect of a fluctuating axial force), which are commonly ignored in simple models. Although extensive research has been carried out for the description and development of the MCPM, the physical phenomena underlying the response of the model to quasi-static and dynamic loading have not been rigorously studied, the model has not been calibrated with reliable experimental data, and important modeling parameters have not been clearly identified by sensitivity studies. The reliability of the model for shear is not well known, and improvements may be needed.

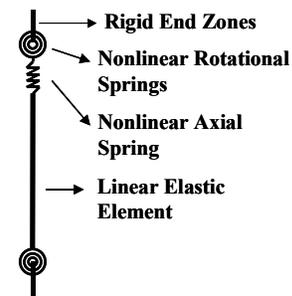


Figure 1. Beam-Column Element

Given these shortcomings, a research project was undertaken to update and calibrate the MCPM for RC and SRC wall systems. The coupling of flexural and shear responses (current approaches assume independence), will also be considered. The model will also be extended for investigating the response of unsymmetrical (e.g., T-shaped) wall sections, implemented into the analytical platform (OpenSees) being developed by the Pacific Earthquake Engineering Research (PEER) Center (<http://opensees.berkeley.edu/OpenSees/developer.html>), and used to study relationships between system and local responses. A discussion on accuracy and effectiveness of the model in predicting the stiffness, strength, and cyclic response of RC structural walls is presented in this paper. Calibration with experimental studies on RC structural walls and

sensitivity studies on model parameters are also emphasized.

Background on MCPM

Prior work (Vulcano et al., 1988) identified that wall flexural response can accurately be predicted by the MCPM when refined hysteretic constitutive laws are adopted. The model offers the flexibility to incorporate various hysteretic material relations and important response parameters (e.g., confinement, slip, nonlinear shear behavior, neutral axis migration). The model in Figure 2 is proposed to simulate the response of the generic MCPM wall element.

A structural wall is modeled as a stack of m MCPM elements, which are placed one upon the other (Figure 2). The flexural response is simulated by a multi-uniaxial-element-in-parallel-model with infinitely rigid beams at the top and bottom (floor) levels: the two external truss elements represent the axial stiffnesses of the boundary columns of the wall, whereas the interior elements represent the axial and flexural stiffnesses of the central panel.

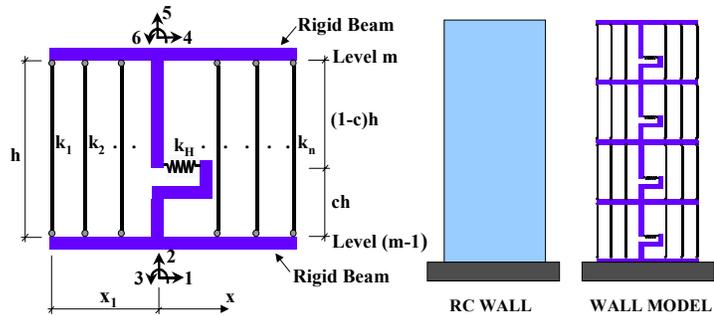


Figure 2. MCPM Element and Wall Model

The number of the axial elements (n) can be increased to obtain a more refined description of the cross section. A horizontal spring, with stiffness k_H and nonlinear hysteretic behavior (e.g., an origin-oriented hysteresis model, OOHM) simulates the shear response of the wall member.

Flexural and shear modes of deformation of the wall member are uncoupled. The relative rotation occurs around the point placed on the central axis of the element of at height ch . A suitable value of the parameter c can be selected on the basis of the expected curvature distribution along the inter story height h , i.e. $0 \leq c \leq 1$, if the curvature sign does not change along h .

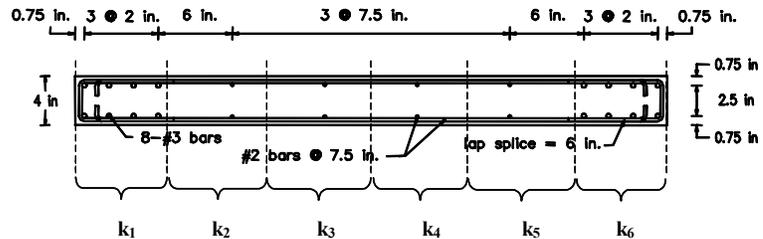


Figure 3. Tributary Area Assignment

Hysteretic constitutive laws for unconfined and confined concrete and for steel are incorporated in the model to define the stiffness and strength properties of the uniaxial elements. The stiffness of the axial elements are also based on the concrete and steel tributary area assigned to each element (Figure 3). The reinforcing steel stress-strain behavior is described by a nonlinear model, such as that described by of Menegotto and Pinto (1973). For this discussion, the monotonic envelope curve of concrete in compression follows the so-called modified Kent and Park model (Park et al., 1982). The implemented concrete model describes the concrete stress-strain relation under an arbitrary cyclic strain history. The effect of concrete confinement on the monotonic envelope curve in compression, successive stiffness degradation, the effect of tension stiffening, and the hysteretic response under cyclic loading in compression and tension have been

addressed (Yassin, 1994). As well, the model provides a flexible approach to implement new material relations. The origin-oriented-hysteresis-model for the horizontal spring is calibrated for a trilinear backbone curve with values of pre-cracked, post-cracked and post yield shear stiffness of the wall cross section. Unloading and reloading occur along straight lines passing through the origin. The reliability of the model for shear is questionable, and additional work is focusing on improving the shear response by linking shear strength and stiffness with flexural ductility. Figure 4 illustrates hysteretic constitutive laws for steel and concrete, and the OOHM for the horizontal spring.

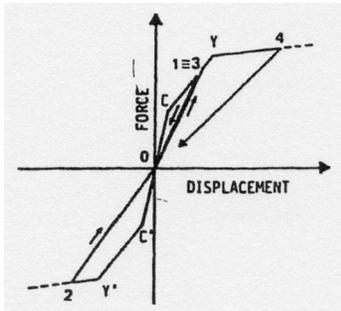


Figure 4 (a). OOHM

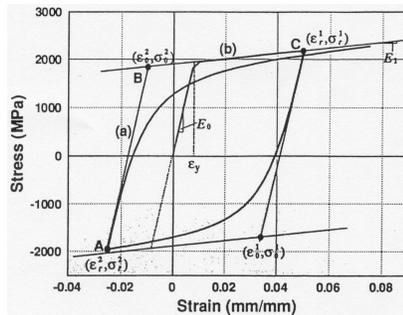


Figure 4 (b). Steel

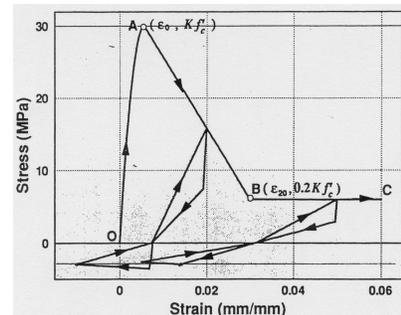


Figure 4 (c). Concrete

Available Experimental Studies

The wall specimens tested by Thomsen and Wallace (1995) and Taylor, Cote and Wallace (1998) are being used to assess and calibrate the model. Geometry of the wall specimens is

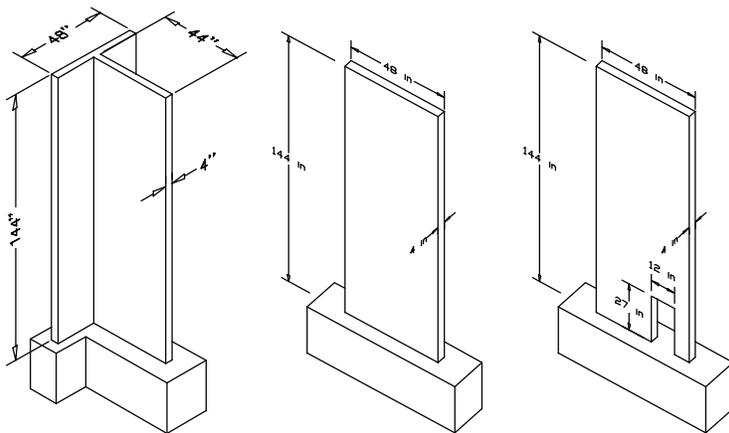


Figure 5. RC Wall Geometry

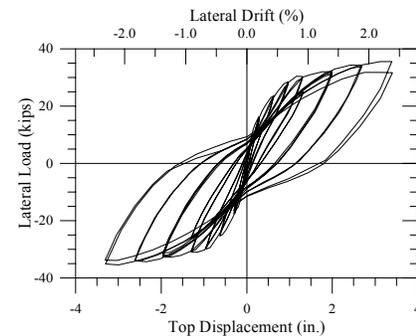


Figure 6. Specimen RW2

shown in Figure 5. Cyclic lateral loads were applied at the top of the approximately 1/4 scale walls. A constant axial load of approximately $0.10A_g f'_c$ was maintained for the duration of testing. The walls were based on a prototype building designed using flexural strength requirements of the Uniform Building Code ("Uniform", 1994); however, detailing requirements were determined using a displacement-based evaluation (Wallace, 1995). Capacity design was used to ensure the location of primary inelastic flexural deformations would occur at the base of

the wall and that adequate shear strength was provided. A strut-and-tie model was used to assess load path and shear strength requirements for the walls with openings. Design and reinforcement details for the walls are presented in detail in the referenced report/paper. Lateral load versus lateral displacement response at the top of the wall is presented in Figure 6 for the rectangular specimen RW2. Substantial instrumentation was used to record important response quantities. In addition, the ongoing research project at UCLA involves testing of three approximately 1/3 scale structural steel reinforced concrete (SRC) walls to address strength, stiffness and detailing issues. Upon completion, the tests will provide the data to assess and calibrate the model to study SRC wall response.

Comparison with Experimental Results

In this paper, the comparison of analytical and experimental results is limited to the RC wall specimen RW2 tested by Thomsen and Wallace (1995). Additional studies will be performed for more RC and SRC wall specimens. The model wall RW2 had a rectangular cross section with dimensions of 48 in. x 4 in. (1220 mm x 100 mm), and a height of 12 feet (3.66 m). The height and length dimensions of the wall result in an overall aspect ratio (h_w/l_w) of 3.0. Reinforcing details of the specimen are illustrated in Figure 7. Instrumentation was used to measure displacements, loads, and strains at critical locations in the wall specimen. Load cells were used to measure axial and lateral loads, and strain gages were used to measure strains in the concrete and the reinforcing steel. A constant axial load of $0.07A_gf'_c$ was maintained throughout the testing. Reversed cyclic loads were applied at the top of the wall under displacement control.

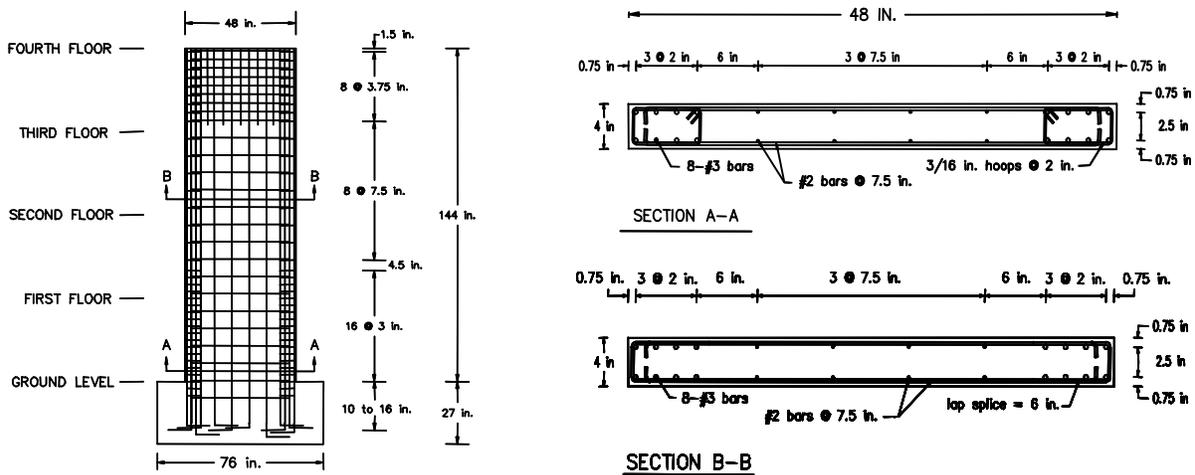


Figure 7(a). Specimen RW2 – Profile

Figure 7(b). Specimen RW2 – Cross Section

The first drift level to be applied was approximately 0.1%, followed by 0.25%. The drift level was then increased in 0.25% increments up to 1.0%. The next drift level applied was 1.5% and another cycle with a drift level at 1.0% followed. The drift was then increased in 0.5% increments up to 2.5%. Measured lateral load versus lateral displacement response of the wall specimen is presented in Figure 6.

The geometry and material models used for the MCPM model were selected to represent

the geometry and material properties of the tested specimen. A cyclic, displacement-controlled, analysis was performed to simulate the test conditions. The analytical model developed, consisted of 8 MCPM members stacked upon each other ($m=8$). To allow for a refined description of the flexural stiffness, and to adequately represent concrete confinement, 22 cross section elements were defined along the length of the wall ($n=22$). A value of 0.4 was selected for the 'c' parameter governing relative height of rotation at each member, in accordance with previously conducted research (Vulcano et al., 1988). Figure 8 shows the discretization of the wall cross section, i.e., the tributary area on the cross section assigned to each vertical element. The number of MCPM members and vertical bars along the wall length are parameters associated with the model and the sensitivity of the response with regard to each of these parameters is discussed in the following section.

The reinforcing steel stress - strain relationship described by the Menegotto and Pinto (1973) model was calibrated to reasonably model the experimentally observed properties of the longitudinal reinforcement at the boundary regions and the uniformly distributed web reinforcement.

A steel yield strength of $f_y = 63$ ksi (434 MPa) and tangent modulus of $E_s = 29,000$ ksi (200,000 Mpa) were assigned to the steel stress-strain model. The strain-hardening ratio was defined to be 0.02.

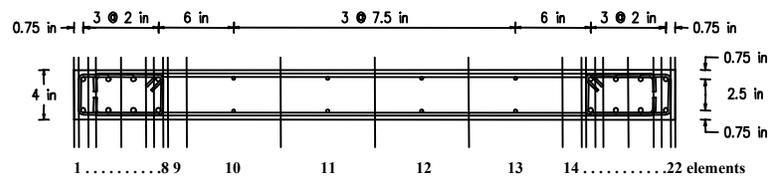


Figure 8. Discretization of Wall Cross section

The monotonic concrete stress-strain relation defined by the modified Kent and Park model was calibrated using mechanical properties of the concrete used for the construction of the test specimen. The compressive concrete strength was assigned as $f'_c=5.5$ ksi (37.9 MPa) in accordance with the observed mean compressive strength at the first story height (0 to 3 ft) of the specimen, whereas the strain at peak compressive stress, ϵ_{0c} , was set equal to 0.0025 in/in. The concrete rupture strength was set equal to $f_r = 7.5\sqrt{f'_c}$ as 0.56 ksi (3.83 MPa). The tension stiffening modulus E_{ts} was assigned a value of 10% of the initial tangent modulus (i.e., $E_{ts} = E_c/10$). In order to account for confinement, the volumetric ratio of the transverse reinforcement at the boundary regions of the specimen and spacing of the hoops were defined as model parameters.

The origin-oriented-hysteresis model used to simulate the shear response of the wall members, based on using a trilinear force-displacement backbone curve with pre-cracked, post-cracked and post-yield shear stiffness, and values of cracking and yield shear force. The pre-cracked shear stiffness was calculated from the relationship $k_H = GA_{eff}/h$, where G is the shear modulus of concrete, A_{eff} is the effective shear area of the cross section, and h is the height of each element. The post-crack and post-yield stiffness values were arbitrarily set at 40% and 1% of the pre-cracked shear stiffness, respectively. Nonlinear quasi-static analysis of the model subjected to a prescribed displacement history was performed using MATLAB (version

5.3.1.29215a) via application of an incremental-iterative displacement controlled solution technique. A lateral top displacement history was applied at the horizontal degree of freedom at the top of the wall model to mimic the history applied to the test specimen.

The analytical results obtained for the lateral load–top displacement relationship are compared to the experimentally observed behavior for wall specimen RW2 (Figure 9(a)). The analytical model reasonably captures overall the cyclic response. Stiffness degradation, hysteresis shape, residual displacements, and pinching behavior are clearly represented in the analytical results. Specifically, the capacity of the wall is predicted very closely at lateral drifts greater than 1.4%. However, a number of discrepancies exist between the calculated and measured responses. The model overestimates the tangent stiffness of the wall for drift ratios less than 0.5%. The initial tangent stiffness estimate by the model is approximately 40% higher compared to experimental results. The unloading stiffness at maximum drift levels was slightly higher than measured response, leading to overestimation of the residual (permanent) displacements, and the

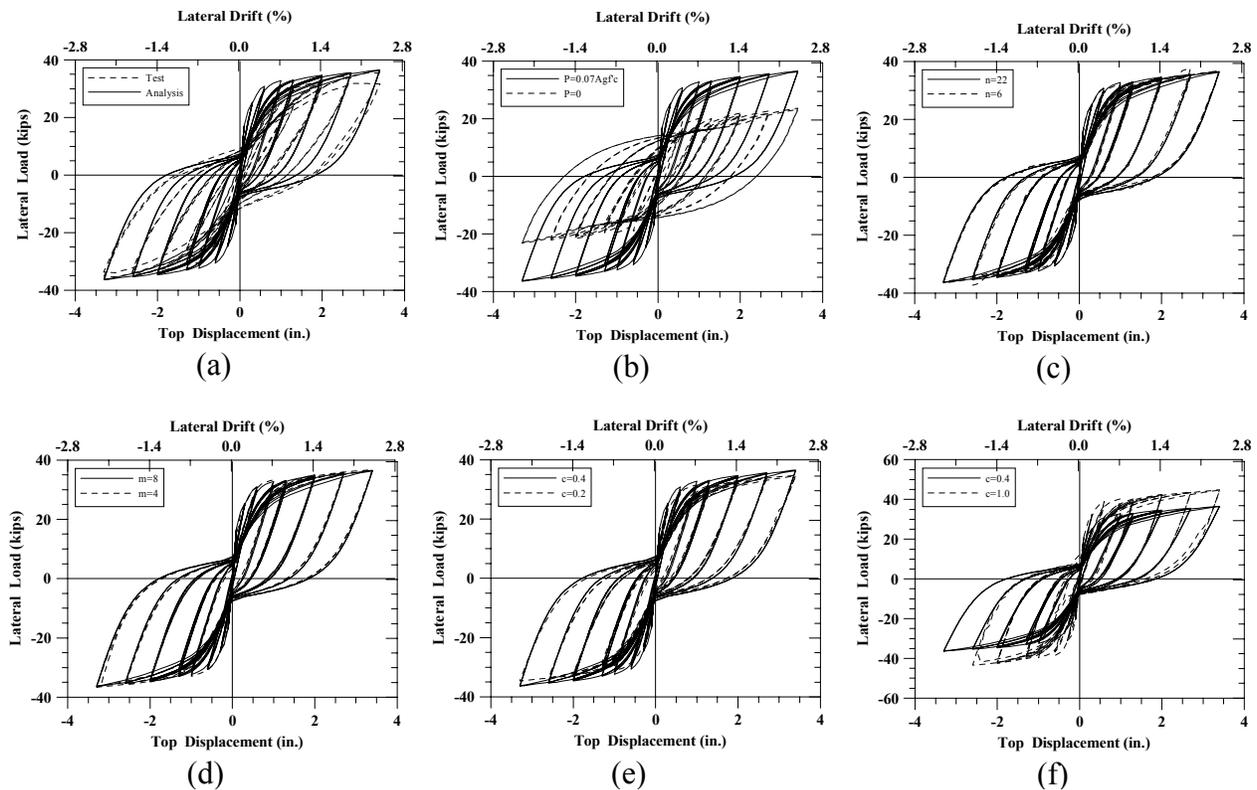


Figure 9. Analytical Results, Experimental Correlation and Parametric Studies

unloading stiffness close to the zero-displacement level is not well represented. Furthermore, a more pronounced and sudden pinching behavior (increase of stiffness going from unloading to reloading due to closure of flexural cracks in concrete) is observed in the analytical response. The hysteresis shape and pinching behavior in the analysis results is highly influenced by the level of axial load, which also affects the stiffness and strength of the wall considerably (Figure 9(b)).

The initial pre-cracked lateral stiffness of the wall model is overestimated by a factor of approximately 40%. By examining the measurements taken from vertical LVDT's at the base of

the wall and concrete strain gages embedded in boundary regions of the wall (the concrete gages measured a lower level of curvature from strains over a smaller gage length even in the pre-cracked range of loading), it was concluded considerable micro-cracking was experienced by the wall, even at very low drift ratios (Figure 10). In addition, the load-displacement measurements clearly exhibited a hysteresis shape, even in the pre-cracked range, which is consistent with the formation of micro-cracks (Figure 11). It is concluded that the results of the analytical model are reasonable, and that the influence of cracking on lateral stiffness occurs earlier than predicted, possibly due to differential shrinkage of concrete at the wall-pedestal interface, or other reasons yet to be investigated.

The model overestimates the unloading stiffness at the maximum drift levels, resulting in slightly higher predictions of residual (permanent) displacements for zero lateral load. Further calibration of the material properties, or implementation of different material models, will be considered to assess approximate values of initial stiffness to account for cracking, which may result in improved correlation. As well, the unloading curves of the analytical results displayed a higher level of curvature, underestimating the lateral stiffness close to the zero-lateral displacement level. The shape of the unloading curve was found to be largely influenced by the curvature properties of the steel stress-strain relationship (Menegotto and Pinto model) defined for the longitudinal reinforcement. Revision of the parameters defining the cyclic curvature of the steel constitutive model through parametric studies or implementation of more refined steel stress-strain relations, may improve response correlations. The analytical model is also found to predict a more pronounced pinching behavior than observed in the experimental behavior. This inconsistency will be improved by implementing an axial stiffness model to consider the contact stresses in concrete due to progressive opening and closing of cracks.

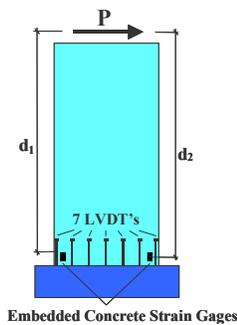


Figure 10. Instrumentation

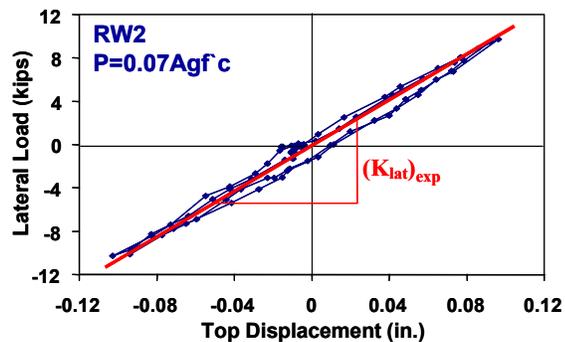


Figure 11. First Two Cycles of Loading

Parametric Studies

The shape of the loading and unloading curves are found to be influenced by parameters associated with constitutive material laws, identifying the need to conduct sensitivity studies on influence of material parameters on response. Apart from the parameters associated with constitutive material laws, the only parameters associated with the analytical model are the number of cross section elements used along the length of the wall cross section (n), the number of MCPM elements stacked on top of each other along the height of the wall (m), and the parameter defining the point of relative rotation along the height of each MCPM element (c). A

refined model configuration with 8 MCPM elements along wall height and 22 elements along wall length is used in the analysis for comparison with the experimental results described in the previous section. The c parameter was selected as 0.4, as suggested by Vulcano et al. (1988). Sensitivity of the response to the variation of these parameters was investigated.

It has been observed that the calculated response is not very sensitive to the selection of either the number of MCPM elements along the height or the wall or the number of vertical elements along wall length, provided that reasonable values are selected in order to adequately represent the overall wall geometry. Figure 9(c) shows a comparison of the lateral load – top displacement response predicted using for 8 MCPM elements stacked along wall height with either 22 cross section elements or 6 cross section elements along the length of the wall. Figure 9(d) represents a comparison of the same response predicted using either 8 or 4 MCPM elements to model the wall, with 22 cross section elements used along the wall length. The comparisons indicate that analytical response obtained using a smaller number of MCPM elements or cross section elements, is essentially the same as that obtained using the more refined models. Accordingly, a model with 4 MCPM elements and 6 cross section elements is sufficient to predict the response of the wall specimen reliably. Significantly increasing the number of vertical elements or the MCPM elements to extensive levels does not improve the prediction of the global response; however, use of more elements is valuable in terms of obtaining more detailed information on local behavior, such as the state of stress and strain in the concrete or steel at various locations along the wall length, or the moment curvature response at various locations over the wall height.

The sensitivity of analysis results on the parameter defining the point of relative rotation along the height of each MCPM element (c) is illustrated in Figures 9(e) and 9(f). Figure 9(e) compares the predicted lateral load – top displacement response for values of $c = 0.2$ and $c = 0.4$. Only a slight difference between the wall strength and stiffness of the two models is observed. To observe a more distinct difference, Figure 9(f) compares the response for $c = 0.4$ and an illustrative value of $c = 1.0$. The difference between predicted wall strengths is evident; therefore, increasing the values of parameter c leads to a higher prediction of the wall strength, and a slightly higher prediction of the lateral stiffness. However, this variation does not influence the shape of the hysteresis curve significantly. The strength of the wall was accurately predicted by using a value of $c = 0.4$ to 0.5, which was also recommended based on prior research (Vulcano et al., 1988).

Conclusions and Future Work

The intent of this research is to investigate and to improve on an effective modeling approach for the reliable prediction of the inelastic response of RC and SRC structural walls. Overall, it was verified that the model is effective in predicting the cyclic load-displacement response of RC walls. The analytical model was able to simulate important behavioral features including shifting of the neutral axis along the wall cross section and the effect of axial force, which are commonly ignored in simple models. The model adequately captured the measured global response with reasonable accuracy. Characteristics of the cyclic response, including stiffness degradation, shape of the hysteresis curve, residual displacements and pinching behavior were clearly represented in the analysis results. The model provided a good prediction of lateral

strength of the wall and lateral stiffness at high drift ratios. However, further sensitivity studies on material parameters and implementation of more refined constitutive laws are needed to address possible model improvements.

Additional work is focusing on improving the shear response, and possibly linking shear strength and stiffness with flexural ductility. As well, the model will be evaluated and refined for SRC walls to address significant design and behavior issues in SRC construction. The model will eventually be implemented into an analysis program to study system response.

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