NONLINEAR FINITE ELEMENT MODELING OF LOW-RISE SHEAR-CONTROLLED STRUCTURAL WALLS

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ABSTRACT

Commonly-used macroscopic modeling approaches available in the literature generally fail to capture important response characteristics of low-rise walls with shear-controlled behavior, including coupling of shear and flexural deformations, as well as characterization of effective shear stiffness, influence of axial load on shear strength, characterization of shear ductility, and simulation of strength/stiffness degradation in the hysteretic load-deformation response. Based on this shortcoming, a relatively simple finite element modeling approach developed previously by the authors, which was shown to provide accurate response predictions for slender and medium-rise walls, is improved and validated in this study for simulating the hysteretic lateral load response of low-rise walls with responses governed by nonlinear shear deformations. The behavioral characteristics of the constitutive panel (membrane) elements incorporated in the model formulation are based on a fixed-crack-angle modeling methodology, together with robust behavioral models for the shear-aggregate-interlock effects in concrete and dowel action on reinforcing bars. Effects of strain penetration within the wall-foundation interface, as well as strain localization effects were addressed in modeling. Comparison of model predictions with experimentally-measured responses of selected low-rise wall specimens available in the literature revealed that the model provides reasonable predictions of the lateral stiffness, lateral load capacity, ductility, and hysteretic response characteristics of the wall specimens investigated, as well as, the contribution of flexural and shear deformations on wall lateral displacements, the distribution of compressive and shear stresses in concrete, the distribution of horizontal and vertical normal strains on the wall, and the orientation of the cracks developing on the walls.

Keywords: Squat; Low Rise; Wall; Concrete; Model; Shear

1. INTRODUCTION

Presence of structural walls has significant contribution in resisting lateral effects imposed on building structures, especially due to earthquake actions and wind loads. Properly designed and detailed structural walls possess the necessary strength, stiffness, deformation capacity to resist lateral actions. Since the existence of structural walls improves the performance of buildings, it is important to understand and characterize the hysteretic behavior of walls under seismic actions. Therefore, various analytical and experimental studies were conducted to investigate and simulate the lateral load behavior of RC structural walls. However, most of the commonly-used analysis methods do not accurately represent the nonlinear hysteretic response of RC walls with shear-controlled responses. Simulation of hysteretic response becomes complicated when the structural wall is relatively squat (low-rise) wall (when aspect ratio is smaller than approximately 1.5 or 1.0), since flexural yielding before shear failure is not a design requirement for squat walls (or wall segments in a perforated perimeter wall system), and the lateral load behavior of such walls are typically governed by nonlinear shear deformations or by coupling of nonlinear flexural and shear responses. Various experimental and analytical studies were conducted on experimental response evaluation and modeling of walls with coupled nonlinear flexural and shear responses (e.g., Orakcal et al. (2009),

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Massone et al. (2009), Terzioğlu (2011), Kolozvari et al. (2015), Tran and Wallace (2015), Gullu (2013). Recently, for simulating the hysteretic response of shear-dominated squat walls, Delso et al. (2015) proposed a finite element model formulation using a smeared crack approach, and Kim (2016) developed a fiber-based wall model that incorporates diagonal strut elements. Both of these modeling approaches were shown to have limitations in capturing the overall load-displacement response of squat walls, and their accuracy in predicting local deformations was not investigated. Hence, there is still a need for a robust modeling approach to simulate the nonlinear hysteretic behavior of low-rise walls with shear-controlled responses. Accordingly, in this study, the finite element modeling methodology developed by Gullu and Orakcal (2017) was adopted and improved for simulating the lateral load behavior of low-rise shear-controlled structural walls under reversed cyclic loading conditions. The behavior of the constitutive panel elements in the finite element model formulation is described by a previously-developed constitutive relationship named as the Fixed-Strut-Angle Model (Uluchtekin, 2010, Orakcal et al., 2012). Improved constitutive models for shear aggregate interlock and dowel action mechanisms were implemented in the present constitutive model formulation for simulating the transfer of shear stress across cracks (Vassilopoulou and Tassios, 2003, Vintizeleou and Tassios, 1986). The finite element wall model with the improved constitutive formulation was calibrated for wall specimens with varying aspect ratios and geometries, where significant nonlinear shear deformations were observed. The model was calibrated for the squat wall specimens tested by Massone et al. (2009), Orakcal et al. (2009), and Terzioglu (2011), and model predictions were compared with the experimentally-observed hysteretic lateral load vs. displacement responses, as well as contribution of flexural and shear deformations on wall lateral displacements and local deformation (e.g., strain) responses.

2. ANALYTICAL MODEL DESCRIPTION

The finite element modeling approach adopted for RC structural walls involves assembling of 4-node constitutive panel elements for obtaining the overall wall model. To represent the constitutive behavior of the wall model elements, nonlinear hysteretic material relationships along crack orientations and reinforcement directions are used in combination with behavioral response characteristics including compression softening, tension stiffening, hysteretic biaxial damage, as well as constitutive shear stress transfer mechanisms across cracks. The so-called Fixed Strut Angle Model (FSAM) originally proposed by Uluchtekin (2010) was selected as the baseline constitutive panel model in the finite element model assembly. Its simple formulation and adequate accuracy makes it a feasible candidate for implementation. The original assumption in the baseline FSAM formulation is that crack directions coincide with principal stress directions in concrete, implying zero shear stress developing along the crack surface. After cracking, principal stress directions in concrete are fixed along the fixed strut (crack) directions, whereas principal strain directions are free to rotate. This base formulation neglects shear aggregate interlock effects in concrete along crack surfaces, and dowel action on reinforcing bars. The behavior of concrete along the fixed compression struts within each panel element is represented using biaxial stress–strain relationships, whereas the behavior of reinforcing steel is described by uniaxial stress–strain relationships applied along the directions of reinforcing steel bars. The hysteretic stress–strain relationship for concrete by Chang and Mander (1994) is adopted in the model formulation (Figure 1(a)) whereas the reinforcing steel stress–strain relationship proposed by Menegotto and Pinto (1973) is used to represent uniaxial hysteretic behavior of reinforcing bars (Figure 1(b)). The constitutive model implemented for concrete was modified by adding compression softening (defined by Vechio and Collins, 1993), hysteretic biaxial damage (defined by Mansour et al., 2002) and tension stiffening effects (defined by Belarbi and Hsu, 1994) into the formulation. These modifications to the constitutive model allow simulating the behavioral features of concrete under biaxial loading conditions.

Prior to formation of the first crack in the FSAM, the stress–strain behavior of concrete is assumed to follow the monotonic envelope. It is assumed that the principal stress and strain directions coincide, and the stress–strain model for concrete is applied in the principal strain direction in a rotating manner (Figure 2(a)). The first crack in the RC panel forms in a perpendicular direction to the principal tensile strain, as the principal tensile strain exceeds the monotonic cracking strain value of concrete. This
implies that for subsequent loading, the first “Fixed Strut” direction is already assigned, and it is parallel to the first crack. The direction of the first strut (crack) remains constant during later loading stages, where the principal strain directions continue to rotate, whereas the principal stress directions, along which the concrete stress–strain relationship is applied, are fixed (Figure 2(b)). Since principal stress and crack directions coincide, zero shear stress develops along the crack directions, implying zero shear aggregate interlock and zero dowel action, which is the underlying assumption in the original FSAM formulation. Using the uniaxial constitutive model adopted for concrete, concrete stress values are determined after the calculation of the strains that are parallel and perpendicular to the fixed strut direction. The stresses in reinforcing steel are calculated by the implementation of the hysteretic uniaxial constitutive model for reinforcing steel applied along the orthogonal rebar directions. The average stresses on the panel element are provided by superposition of stresses developing in concrete and reinforcing steel.

As the tensile strain exceeds the cyclic cracking strain of concrete, the second crack is formed perpendicular to the first crack because of the zero shear stress assumption along the crack directions. Before formation of the second crack, the constitutive behavior proceeds in the form of a single fixed strut mechanism. The first and second cracks being perpendicular is an example of the so-called “orthogonal crack” modeling approach, which has also been adopted in other constitutive panel model formulations. The second crack, being perpendicular to the first fixed strut, implies the formation of the second “fixed strut” direction. These fixed struts work under tension or compression, depending on the loading direction. The biaxial behavior of concrete after formation of the second crack is shown in Figure 2(c).

During further stages of loading, principle stress directions are fixed along the fixed strut directions while principal strain values are able to rotate freely. The applied strain field is transformed into strain components in the fixed strut directions instead of principal strain directions in order to determine the principle stresses in concrete. To obtain the principle stresses in concrete, calculated strain values are used in the uniaxial constitutive model for concrete and the concrete stresses are reduced by compression softening and biaxial damage parameters.

The constitutive material model for reinforcing steel is applied along the orthogonal rebar directions and superposition of stresses developing in concrete and reinforcing steel gives the resultant average stresses on the panel element.
The original formulation of the FSAM (Ulugtekin, 2010), in which the aggregate interlock and the dowel action mechanisms are ignored, is modified herein, since the assumption that no shear stress transfer occurs across crack surface may lead to overestimation of shear deformations and prediction of premature sliding shear failures. To remedy this shortcoming of the original FSAM, for representing shear transfer across cracks, shear aggregate interlock mechanisms along crack surfaces (Figure 3 (a)) associated with frictional shear stresses and clamping effects of reinforcing steel bars were implemented in the present model formulation, using the hysteretic constitutive model proposed by Vassilopoulou and Tassios (2003) which relates the shear stress on the crack surface with crack slip deformation. Figures 3(b) and 3(c) show the hysteretic features of the constitutive shear aggregate interlock behavior, with empirically-defined shear stress capacities. The shear stress capacities associated with frictional and clamping aggregate interlock mechanisms along a crack surface are defined using Equations 1 and 2, respectively:

\[
\tau_{fr, max} = (\mu) \left( f_c \right)^{2/3} \left( \sigma_e \right)^{1/3} \tag{1}
\]

\[
\tau_{cl, max} = (\mu) \left( f_c \right)^{2/3} \left( \rho \sigma_s \right)^{1/3} \tag{2}
\]

where \( \mu \) is the friction coefficient, \( f_c \) is the concrete compressive strength, \( \rho \) is the reinforcing bar ratio, \( \sigma_e \) and \( \sigma_s \) are the normal compressive stress in concrete and axial bar stress perpendicular to the crack surface, respectively. Shear friction coefficient \( \mu \) was recommended to be taken as 0.44 by Vassilopoulou and Tassios (2003). In the interlock model, when the normal stress perpendicular to the crack surface is tensile (crack is open), the frictional shear stress due to aggregate interlock is assumed to be zero.

In the present formulation of the FSAM, a robust constitutive model was also implemented to represent dowel action on reinforcing bars (Figure 4(a)), as proposed by Vintzeleou and Tassios.
(1986), defined as a dowel force vs. deformation relationship for cracked concrete (Figure 4(b)). The dowel force capacity for a reinforcing steel bar is obtained empirically using Equation 3:

\[
V_{\text{max}} = 1.30 (D_b)^2 (f_c f_y)^{1/2}
\]  

where \(f_c\) is the concrete compressive strength, and \(D_b\) and \(f_y\) are the bar diameter and bar yield strength, respectively. Details of the constitutive shear transfer mechanisms across cracks (shear aggregate interlock and dowel action) are available in Vassilopoulou and Tassios (2003) and Vintzeleou and Tassios (1986).

![Figure 3](image_url)  
Figure 3. a) Shear aggregate interlock mechanism along a crack, b) Hysteretic model for shear aggregate interlock for small crack slip, c) Hysteretic model for shear aggregate interlock for large crack slip (Vassilopoulou and Tassios, 2003)

![Figure 4](image_url)  
Figure 4. a) Dowel mechanism along a crack, b) Hysteretic model for dowel action on reinforcement (Vintzeleou and Tassios, 1986)

3. EXPERIMENTAL VALIDATION OF THE MODEL

3.1 Experimental Programs on Squat Walls

For experimental validation of the model, experimental results obtained for four of the eleven squat wall specimens tested by Terzioglu (2011) were first used. These specimens were characterized by their aspect ratios (0.33, 0.5 and 1.0), axial load levels (0 to 5%\(A_g f'_c\)), and reinforcement configurations at boundary and web regions. A representative figure for the geometry and reinforcement details of one of the four wall specimens used for validation of the model is presented in Figure 5(a). The concrete compressive strength varied between 19 and 35 MPa for these specimens, whereas the reinforcing bar yield strength varied between 440 MPa and 575 MPa. Two wall pier specimens having a shear span-to-depth ratio of 0.44 tested by Massone et al. (2009) were used for further validation of the model. Figure 5(b) shows the dimensions and the reinforcement details of the two identical specimens with different concrete compressive strength values and different axial load levels. The concrete compressive strength of these specimens varied between 28.3 and 32.0 MPa, whereas the rebar yield strength was 424 MPa.
Properties of the wall specimens, named with the initials SW tested by Terzioglu (2011) and WP tested by Massone et al. (2009), used to verify the analytical models are summarized in Table 1. The first set of specimens (Terzioglu, 2011) incorporated a RC pedestal at the bottom for connection of the specimen to the laboratory strong floor, and a RC load transfer beam at the top for connection of the specimen to the loading actuator. Lateral loading on the specimens was applied by an actuator, one end of which was fixed to a reaction wall. All walls were tested under reversed cyclic lateral loads, while the axial loads on the specimens were kept constant. The two squat wall specimens tested by Massone et al. (2009) had a different test setup from the other specimens. They were tested under zero rotation imposed on both top and bottom pedestals, a constant axial load, and cyclic lateral load applied at mid-height level of the specimens, creating a double-curvature loading condition where the bending moment on each wall is zero at wall mid height and maximum (with reverse signs) at the top and bottom cross-sections of the wall, representing the actual loading conditions on wall piers and spandrels in a perforated perimeter wall of a building.

Table 1. Geometry and reinforcement details for wall specimens investigated

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Height (cm)</th>
<th>Length (cm)</th>
<th>Aspect Ratio</th>
<th>$P_y/A_f'c$</th>
<th>Web Rein. (Transverse / Longitudinal)</th>
<th>Reinforcing Bar</th>
<th>$\rho$ (%)</th>
<th>Boundary Reinforcement</th>
<th>Reinforcing Bar $\rho_b$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW-T2-S3-4</td>
<td>75</td>
<td>150</td>
<td>0.50</td>
<td>0.00</td>
<td>$\phi8 @ 125$ mm</td>
<td>0.68</td>
<td>4 $\phi 16$</td>
<td>5.15</td>
<td></td>
</tr>
<tr>
<td>SW-T4-S1-6</td>
<td>50</td>
<td>150</td>
<td>0.33</td>
<td>0.00</td>
<td>$\phi8 @ 125$ mm</td>
<td>0.68</td>
<td>4 $\phi 14$</td>
<td>3.95</td>
<td></td>
</tr>
<tr>
<td>SW-T5-S1-7</td>
<td>150</td>
<td>100</td>
<td>1.00</td>
<td>0.00</td>
<td>$\phi8 @ 125 / 250$ mm</td>
<td>0.68 / 0.34</td>
<td>4 $\phi 22$</td>
<td>9.75</td>
<td></td>
</tr>
<tr>
<td>SW-T1-N5-S1-10</td>
<td>75</td>
<td>150</td>
<td>0.50</td>
<td>0.05</td>
<td>$\phi8 @ 250$ mm</td>
<td>0.34</td>
<td>4 $\phi 16$</td>
<td>5.15</td>
<td></td>
</tr>
<tr>
<td>WP-T5-N5-S1</td>
<td>137</td>
<td>122</td>
<td>0.44</td>
<td>0.05</td>
<td>$\phi13 @ 305$ / 330 mm</td>
<td>0.278 / 0.227</td>
<td>2 $\phi 13$</td>
<td>1.33</td>
<td></td>
</tr>
<tr>
<td>WP-T5-N10-S2</td>
<td>137</td>
<td>122</td>
<td>0.44</td>
<td>0.10</td>
<td>$\phi13 @ 305$ / 330 mm</td>
<td>0.278 / 0.227</td>
<td>2 $\phi 13$</td>
<td>1.33</td>
<td></td>
</tr>
</tbody>
</table>

The finite element model configurations used for modeling and analysis of the test specimens were calibrated for several aspects of the squat wall specimens, including the geometric properties, reinforcement attributes, material characteristics, and loading conditions. As-tested properties of the materials employed in the construction of the specimens are used to calibrate the constitutive material parameters applied in the model formulation. In calibration of the parameters for concrete and
reinforcing steel in tension, tension stiffening effects were considered. Wall models generated also incorporated zero-length rotational springs at the wall-pedestal interfaces (Figure 6), calibrated to represent strain penetration effects within the pedestals, similarly to the approach used by Massone et al. (2009). To define the rotational spring stiffness, a linear strain distribution was assumed along the development length of the longitudinal reinforcing bars at wall boundaries, the integration of which was used (together with as assumed neutral axis depth) to calculate an interface rotation under a specific bending moment. As well, strain localization effects, as proposed by Coleman and Spacone (2001) were incorporated by the constitutive material calibrations in the modeling approach. Model predictions were compared with test results both at global (lateral load–displacement) and local (deformation, strain, stress) response levels for the wall specimens.

![Figure 6. Interface crack and rotational spring (Massone et al., 2009)](image)

3.2 Model Results and Comparison with Test Data

Experimentally-obtained responses for squat wall specimens with shear-controlled responses were compared with predictions of the analytical model. Among the wall specimens investigated, specimen SW-T2-S3-4 had an aspect ratio of 0.5, and there was no axial load applied on this specimen. The measured and predicted lateral load vs. top displacement responses for specimen SW-T2-S3-4 are compared in Figures 7(a) and 7(b). The initial stiffness of the wall specimen was reasonably captured by the analytical model, although the stiffness after cracking was overestimated. The lateral load capacity of the specimen was well-predicted in both positive and negative loading directions. The model underestimated the ductility for the wall; however, degradation of the lateral load and pinching behavior were represented fairly well. Specimen SW-T5-S1-7 differentiated with its aspect ratio of 1.0, high boundary reinforcement ratio, and observed shear-flexure interaction behavior. As depicted in Figures 7(c) and 7(d), the model predictions for this specimen, in terms of lateral load capacity, stiffness, ductility, pinching characteristics, and degradation in lateral load are again reasonable.

![Figure 7. Lateral load vs. top displacement responses for a) SW-T2-S3-4 (Test), b) SW-T2-S3-4 (Model) c) SW-T5-S1-7 (Test), d) SW-T5-S1-7 (Model)](image)

SW-T4-S1-6 was the specimen with the lowest aspect ratio of 0.33, while specimen SW-T1-N5-S1-10 had an aspect ratio of 0.5 and it was tested under an axial load level of 5%Acf. The model slightly underestimates the stiffness and ductility of specimen SW-T4-S1-6, and reasonably represents its lateral load degradation characteristics (Figures 8(a) and 8(b)). The lateral load capacity of the specimen was overestimated by the model especially in the negative loading direction. Reasonably accurate lateral load capacity predictions were achieved in both positive and negative loading...
directions by the model for the specimen SW-T1-N5-S1-10 that was subjected to axial load during testing, which demonstrates that the model successfully represents the influence of axial load on the lateral load capacity of these walls (Figures 8(c) and 8(d)). This specimen experienced sudden strength degradation in the positive loading direction due to crushing of concrete, whereas degradation in the analysis results was more gradual, as is typically the case with the model results. The model underestimates the ductility of this specimen and predicts less pronounced pinching behavior.

As shown in Figures 9(a) and 9(b) for specimen WP-T5-N5-S1, which was tested under double-curvature and subjected to an axial load corresponding to 5% of its axial load capacity, the model exhibited good accuracy in predicting the stiffness, lateral load capacity, and ductility characteristics of the wall, in both positive and negative loading directions. Degradation in lateral load, although captured by the model, was more gradual in the analytical results. A similar correlation was obtained for specimen WP-T5-N10-S2, subjected to an axial load corresponding to 10% of its axial load capacity. The response was well-predicted by the model in terms of stiffness, lateral load capacity, and ductility (Figures 9(c) and 9(d)). Experimentally observed sudden degradation in lateral load was captured by the model, although the model predicted relatively more gradual degradation. It is a significant attribute that the overall response prediction of the model for the axially-loaded wall specimens is accurate, because the model is apparently successful in representing the influence the axial load on the shear-controlled response characteristics of walls. Considering that seismic design codes and performance assessment guidelines neglect the influence of axial load in calculation of the shear strength and stiffness of RC walls, mainly due to lack of experimental data, availability of a modeling approach considering interaction between axial load and shear capacity is promising towards improvement of code provisions on wall shear strength.

Analytical model predictions were compared with test results at also local (deformation) response levels. It is shown in Figure 10 that the model can successfully simulate the lateral load vs. shear and flexural deformation contributions of top displacement for specimens SW-T2-S3-4 and SW-T5-S1-7 with aspect ratios of 0.5 and 1.0, respectively. Test results and model predictions are compared up to the data point where local deformation measurements were available. As shown in the figures, the model can reliably predict the contribution of flexural and shear deformations to wall lateral displacement, as well as the flexural and shear stiffness values for the wall specimens with aspect...
ratios of 0.5, and 1.0. Both experimental and analytical results show that the contributions of shear deformations to wall lateral displacements are significant, and flexural deformation components of the response are closer to linear elastic behavior, which is expected for squat walls with shear-controlled lateral load responses.

Furthermore, test results and model predictions for the distribution of average horizontal normal strain (across wall length) values along wall height, at increasing drift levels, are compared in Figure 11 for two representative wall specimens. Horizontal normal strains are significant response quantities for low-rise walls, since these strains are direct indications of whether the horizontal web reinforcement in the wall yields or not, which further determines whether the wall experiences diagonal tension failure or diagonal compression failure, respectively (Terzioglu, 2011). Both test measurements and model predictions presented in Figure 11 indicate that the average horizontal normal strains tend to increase towards their maximum values at wall mid-height, due to the lateral constraining effect of the top and bottom pedestals. As well, the maximum values predicted by the model for the average horizontal normal strain on each wall are in reasonable agreement with the experimentally-measured values, indicating that the model is also reliable for prediction of yielding in horizontal web reinforcement, which in turn influences the wall shear capacity and failure mode.

Model predictions for the distribution of vertical normal strains along the height and the width of the walls are also presented, although they cannot be compared with experimental observations due to
absence of vertical strain measurements during the tests. Figures 12(a) and 12(b) representatively illustrate the vertical strain profiles estimated by the model along the height of the specimens with aspect (and shear-span-to-depth) ratios 0.5 and 1.0, corresponding to different drift levels in the positive loading direction. The vertical strain values shown in the figures were obtained at the left boundary of each wall specimen, which is subjected to tensile strain under loading in the positive direction. The vertical tensile strain values gradually decrease with increasing distance from the base of the wall specimens, since these walls were tested under single curvature (cantilever) loading condition, with zero bending moment at the top of the wall. Model predictions for the distribution of vertical normal strains along the web of the same wall specimens (at the base of each wall) are presented in Figures 12(c) and 12(d). The magnitudes of the vertical strains predicted by the model are relatively low, which is expected since the lateral load behavior of these walls is predominantly shear-controlled and the contribution of flexural deformations to the lateral displacement of the walls are limited. As also depicted in the figures, the distribution of the vertical strains predicted by the model along the web of the walls is not compatible with the plane-sections-remain-plane assumption, which is also expected considering the low aspect ratio of the walls investigated.

![Figure 12](image)

Figure 12. Vertical strain profile along height at wall boundary for a) SW-T2-S3-4, b) SW-T5-S1-7, Vertical strain profile along web at wall base for c) SW-T2-S3-4, d) SW-T5-S1-7

Again, for illustrative purposes, Figures 13(a) and 14(a) show representative crack orientations (with solid lines representing open cracks and dashed lines representing closed cracks) predicted by the model for two wall specimens with shear-span-to-depth ratios of 0.5 and 1.0, at the drift level corresponding to their lateral load capacity in the positive loading direction. As shown in the figure, the cracks are mostly diagonal within the web region of the walls, representing formation of a diagonal strut under shear-dominated behavior. However, they tend to be more horizontal at the base of the walls and at the wall boundaries, where flexural deformations are somewhat more influential. The vertical strain contours presented in Figures 13(b) and 14(b) are predicted by the model for the same two wall specimens at their lateral load capacity, and demonstrate how the distribution of the vertical strains is compatible with both the direction of loading and the bending moment distribution along the height of the walls.

Figures 13(c) and 14(c) present stress contours for the principal compressive stresses in concrete (also incorporating shear aggregate interlock stresses on crack surfaces) predicted by the model for the same wall specimens at their lateral load capacity. The diagonal struts, formation of which constitutes the primary shear resisting mechanism of squat walls with shear controlled responses, can be clearly observed in the principal compressive stress contours.

Finally, Figures 13(d) and 14(d) show stress contours for the in-plane shear stresses predicted for the two wall specimens. As illustrated in the figures, the magnitude of the local shear stresses is larger along the diagonal struts, where the principal compressive stresses in concrete are also of larger magnitude. The results demonstrate the significant contribution of shear stresses to the principal compressive stresses in concrete, which is to be expected for squat walls with predominant shear behavior.
4. CONCLUSIONS

A nonlinear finite element modeling approach was adopted and validated for simulating the hysteretic lateral load response of low-rise walls with aspect ratios not larger than 1.0, the response of which is governed by nonlinear shear deformations. The behavioral characteristics of the constitutive panel (membrane) elements used in the model formulation are based on a fixed-crack-angle modeling methodology. Improved constitutive models for shear aggregate interlock and dowel action mechanisms were implemented in the constitutive model formulation for simulating shear stress transfer across cracks. Rotational springs at the wall-pedestal interfaces representing strain penetration effects within the pedestals were also implemented and strain localization effects were incorporated by the constitutive material calibrations in the modeling approach.

The finite element model with the refined constitutive formulation was calibrated for selected wall specimens with shear-controlled nonlinear behavior, with varying aspect ratios, geometries, and reinforcement configurations, and tested under different loading configurations. The model was shown to provide reasonable predictions of the lateral stiffness, lateral load capacity, ductility, pinching, and lateral load degradation characteristics of the wall specimens investigated. As well, the model provided reasonable estimates for flexural and shear deformation contributions to wall response and the distribution of average horizontal normal strains on the walls. Reasonable model predictions were also obtained for the distribution of compressive and shear stresses in concrete, the distribution of horizontal and vertical normal strains on the wall, and the orientation of the cracks developing on the walls with shear-dominant responses.

The modeling approach adopted in this study is believed to be a significant improvement towards reliable prediction of the response of squat walls under reversed-cyclic loading conditions. Numerical stability of the model, as well as the capability of the model to effectively simulate degrading (softening) lateral load responses, makes it a feasible candidate for simulation of shear-controlled wall responses, as well as general responses under combined shear and flexural actions.
5. REFERENCES


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