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# EXPERIMENTAL VERIFICATION OF DISPLACEMENT-BASED DESIGN PROCEDURES FOR SLENDER RC STRUCTURAL WALLS

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## ABSTRACT

This paper presents the results of experimental studies of large scale wall specimens with rectangular, tee-shaped, and barbell-shaped cross sections to verify of a recently developed displacement-based design procedure. This procedure evaluates the need for special transverse reinforcement at the wall boundaries to provide concrete confinement and suppress buckling of the longitudinal reinforcement. Six wall specimens were tested; three with rectangular cross sections, two with tee-shaped cross sections, and one with a barbell shaped cross section. One of the rectangular walls and the barbell shaped walls included large openings at the base of the wall near the boundary. The specimens were tested under reverse cyclic loading and a constant axial load of approximately $0.10A_g f'_c$. The conclusions of the study are: (1) displacement-based design is a flexible design tool, (2) experimental studies validate the use of displacement-based design, (3) special attention is required for the design of walls with tee-shaped cross sections, and (4) a strut and tie model used in conjunction with a displacement-based design procedure is an effective way to design slender walls with openings in the plastic hinge region.

## KEYWORDS

Displacement-based design, performance-based design, ductility, structural wall, shear wall, openings, unsymmetric, experimental, testing, strut and tie model, effective flange width

## INTRODUCTION

The use of reinforced concrete structural walls is common for resisting lateral forces imposed by wind or earthquakes. In areas of high seismic risk, it is usually not feasible to design a structural wall to remain elastic during a severe earthquake (Wallace and Moehle, 1992); therefore, inelastic deformations are expected, usually at the base of the wall. Allowing inelastic deformations reduces the force that the wall must resist, provides a "fuse" to limit damage to other elements in the structure, and can provide significant damping. In order to exhibit stable, inelastic behavior, the wall must be specially detailed, that is, transverse reinforcement must be provided in regions of high strain.

Prior to 1994, US code provisions regarding the design of reinforced concrete structural walls focused on strength requirements (i.e. ACI 318-89; UBC-1991). Adequate deformability was achieved through the use of heavily confined boundary elements (typically wider than the wall web, resulting in a barbell shaped cross section) whenever the extreme fiber stress due to combined axial and lateral loads exceeds $0.2f'_c$. The confinement within the boundary elements must continue over the wall height until the extreme fiber stress falls below $0.15f'_c$. These boundary elements must be capable of resisting gravity loads and overturning moments without the aid of the wall web. These requirements often made it costly to use a predominantly structural wall system, especially in areas of high seismic risk. Studies have shown that these code requirements are overly conservative for a majority of building systems that utilize reinforced concrete structural walls for lateral load resistance (Ali and Wight, 1991; Wood, 1991; Wallace and Moehle, 1992).

During the 1985 earthquake near Viña del Mar, Chile, the approximately 400 modern reinforced concrete buildings were subjected to strong ground motions. In general, these buildings performed very well. The good performance of these buildings was attributed to the relatively large number of structural walls which reduced the deformations imposed on the walls; therefore, specially detailed boundary regions were not required. Analytical studies were performed to evaluate several Chilean buildings and to extrapolate the findings to U.S. practice (Wallace and Moehle, 1992; 1993; Wallace, 1994). These studies focused on establishing likely response characteristics (wall deformation capacity) of shear wall buildings to estimate design requirements. The analytical studies were compared with experimental studies performed on structural walls of rectangular or nearly rectangular cross section (Carvajal and Pollner, 1983; Oesterle et al., 1976; Shiu et al., 1981; Oesterle, 1986; and Ali and Wight, 1991). The results indicated that the analytical procedure used to estimate the drift capacities tends to yield conservative estimates of wall deformation capacity, and that this procedure provides a simple, effective method for determining the need for confining steel at structural wall boundaries. Subsequent analytical studies of reinforced concrete structural walls has lead to the development of a displacement-based design procedure (Wallace and Moehle, 1992; 1994). The procedure uses lateral drift, as opposed to strength, and directly relates computed building response and wall attributes to the need to provide transverse reinforcement at the wall boundaries based on preventing concrete crushing and suppressing buckling of the longitudinal reinforcement.

Although the displacement-based (performance-based) analytical procedure has been well documented, experimental studies have not been conducted to verify the analytical procedure. In particular, the analytical procedure uses monotonic force-deformation relations (moment-curvature relations) to assess the need for transverse reinforcement at the wall boundaries. In addition, very little experimental information is available to assess the performance of rectangular walls with moderate amounts of transverse reinforcement at the wall boundaries. Essentially no data are available for walls with unsymmetrical cross sections, or for walls with openings at the base. Given these needs, a comprehensive experimental program was undertaken as outlined in the following sections.

## EXPERIMENTAL STUDIES

Experimental verification of the proposed displacement-based design approach involved the testing of six approximately quarter-scale wall specimens. The walls tested were: two solid rectangular walls, two solid tee-shaped walls, one rectangular wall with an opening, and one barbell shaped wall with an opening (Figs. 1 to 3). The walls were 12 ft (3.66 m) high and 4 ft (1.22 m) long, resulting in an aspect ratio of 3. A wall thickness of 4 inches (102 mm) was used. Typical material properties were selected, $f'_c$=4 ksi (27.6 MPa) and $f_y$=60 ksi (414 MPa). Further details can be found in Thomsen and Wallace (1995) for the solid walls and in Taylor and Wallace (1995) for the walls with openings.

A displacement-based design procedure was used to select transverse reinforcement at the wall boundaries. Based on analytical studies of prototype buildings (Thomsen and Wallace, 1995), each wall was designed for 1.5% lateral drift. Transverse reinforcement was provided to ensure that the walls could reach the design curvature. For strains exceeding 0.004, special detailing was used to provide concrete confinement and to suppress buckling of the longitudinal reinforcement. The flanges were considered for the tee-shaped walls and

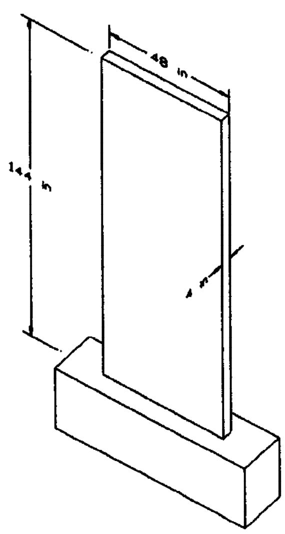

Fig. 1 Rectangular Specimen

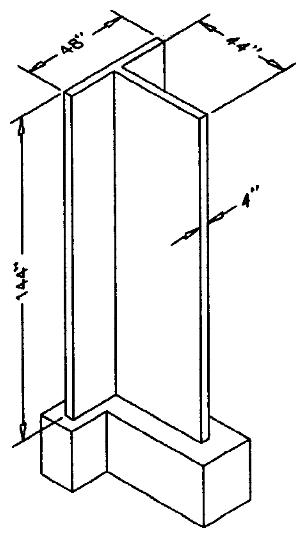

Fig. 2 Tee-Shaped Specimen

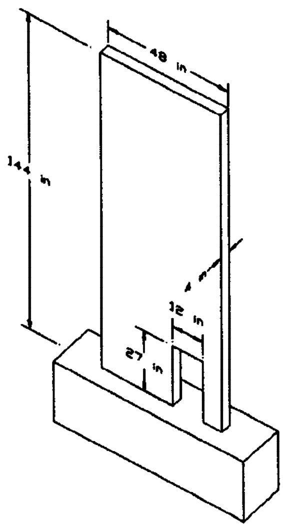

Fig. 3 Rectangular with Opening

the openings were considered for the walls with openings; therefore, the normal strain distributions computed for positive and negative lateral loading differ. Higher concrete compressive strains are predicted for the tee-shaped wall when the flange is in tension, and for the wall with an opening when the "column" is in compression (defined as negative loading in the experimental program for both wall types). Figure 4 shows the reinforcement provided for the rectangular wall with an opening. The reinforcement for the other rectangular wall sections were similar. Figure 4 shows that a greater amount of transverse reinforcement was provided in the column, next to the opening, where greater compressive strains were expected. Analysis of the tee-shaped wall also indicated that considerably greater amounts of transverse reinforcement spaced over a greater depth was needed at the web boundary. In contrast, based on the computed normal strain distribution for the tee-shaped walls, very little transverse reinforcement was needed at the web-flange intersection.

For the solid rectangular wall (RW2), minimum web reinforcement was used (deformed US #2 bars @ 7.5 inches; $\rho = 0.00327$) since the shear stress at wall flexural yielding was only $2.5\sqrt{f_c'}\,psi$ ($0.21\sqrt{f_c'}$ MPa). For the tee-shaped wall, the design shear force was computed from the flexural capacity of the wall assuming all of the flange longitudinal reinforcement was effective as tension steel; therefore, the flexural capacity of the Tee-shaped wall was approximately twice that for the rectangular wall and greater amounts of web steel was provided deformed US #2 bars @ 5.5 inches (6.4 mm @ 140 mm), $\rho = 0.00445$.

Since conventional techniques are not effective for the shear design of discontinuous regions, shear reinforcement for the walls with openings was selected using strut and tie models. A refined model was developed for each wall in each direction. The models indicated that, when the column was in compression, a concentration of horizontal steel was required over the opening to drag the shear force back into the solid panel. Use of a strut and tie model ensured that an adequate load path was provided to carry the applied horizontal load from the top of the wall down to the base. Although the strut and tie model does not assess the need for vertical web reinforcement, equal amounts of horizontal and vertical web steel were provided in the panel region at the base of the wall. Analytical studies (Sittipunt and Wood, 1994) suggest that diagonal reinforcement is the most efficient distribution of web reinforcement. Although less efficient, a uniform mesh of horizontal and vertical reinforcement was used for ease of construction.

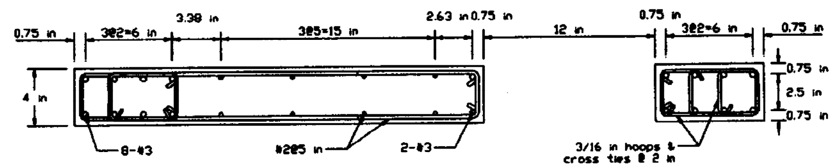

Fig. 4 Cross Section at Base of Wall with Opening

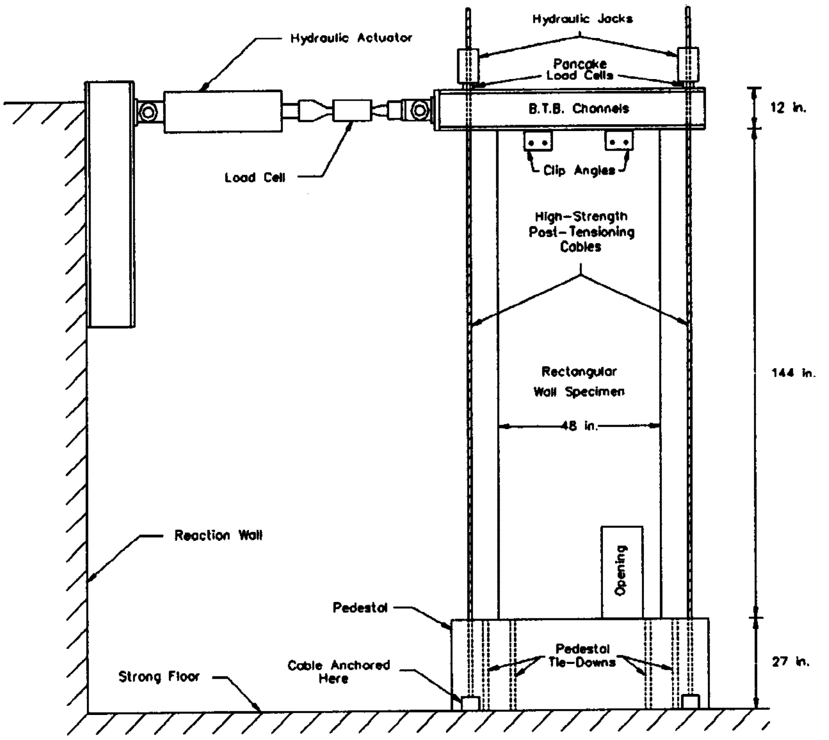


Fig. 5 Test Setup

Instrumentation was provided to measure loads, displacements and strains at critical locations. All measurements were read by a computer data acquisition system at 32 points throughout each cycle. Lateral load was measured with a load cell placed between the actuator and the load transfer assembly. Axial loads were measured with hollow core load cells placed between the top chucks and the jacks. The horizontal displacement profile of each specimen was measured using wire potentiometers at four locations over the wall height. The wire potentiometers were mounted on a steel reference frame connected independently to the floor. Linear potentiometers were provided at each end of the pedestal to determine the vertical displacement of the pedestal, from which the pedestal rotation was calculated. To obtain the rotation at the base of the wall (over approximately the plastic hinge length), wire pots were mounted at the first story height and connected to the pedestal to measure the vertical displacement of each side of the wall. Shear deformations were measured using wire potentiometers mounted in an "X" configuration over the bottom two stories. Vertical strain along the base of each wall was measured three ways. First, steel strain gages were attached to many of the reinforcing bars just above the base of the wall and at the first story level. Second, concrete strain gages were embedded at various locations along the base of the wall. Finally, linear voltage differential transducers (LVDT's) were used to measure vertical displacement along the base of the wall over a gage length of approximately 9 inches (229 mm).

The base of each specimen was fixed to a strong floor and the top was free to rotate (Fig. 5). Out of plane movement was restrained at the top of the walls. A constant axial load of approximately $0.10A_g f'_c$ was maintained for the duration of each test. Reverse cyclic horizontal loading was applied slowly at the top of each specimen in the plane of the wall or web in the case of the tee-walls. Typically, two complete cycles were performed at each drift level; however, four cycles were typically performed at lateral drift levels of 1.0 and 1.5%. The drift levels investigated were approximately 0.10%, 0.25%, 0.50%, 0.75%, 1.0%, 1.5%, 2.0%, 2.5%, and 3.0%.

## EXPERIMENTAL RESULTS

### Lateral Load Versus Top Displacement

Figures 6 through 8 show the applied lateral load versus top displacement relations (note that the vertical axis are different for Fig. 7). The applied lateral load was measured using a load cell mounted between the hydraulic actuator and the top loading assembly. Top displacement was measured using a wire potentiometer. In general, excellent performance was observed for the wall specimens. Lateral drift levels of 2 to 2.5% were applied before significant deterioration was observed. For each specimen, the point of yielding can be identified to occur at approximately 0.75% drift. For the tee-shaped wall, the post-yield stiffness is

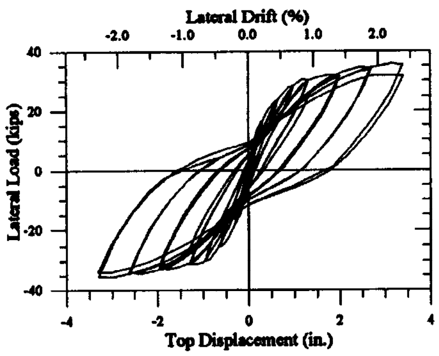


Fig. 6 RW2: Load vs. Disp.

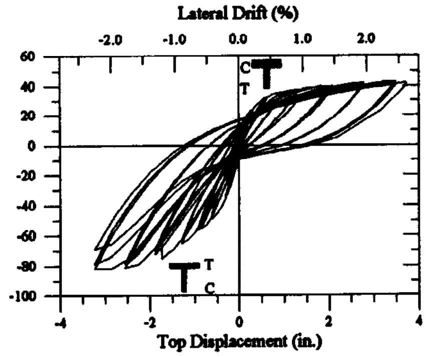


Fig. 7 TW2: Load vs. Disp.

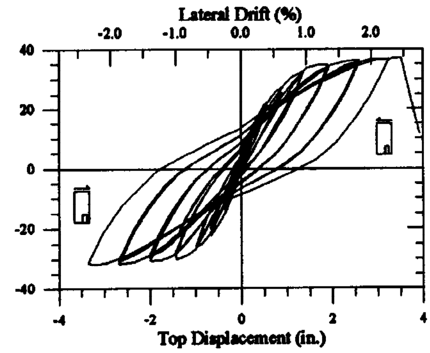


Fig. 8 RW3-O: Load vs. Disp.

significantly greater and the strength is nearly double when the web is in compression due to the large neutral axis depth caused by the large amount of tension steel within the flange. Figure 7 shows that tee-shaped walls do not perform simply as two rectangular walls as may be assumed in design. For the walls with openings, the strength and stiffness in the positive direction are only slightly greater than in the negative direction for each specimen. The similar strengths and stiffnesses were the result a shallow neutral axis depth; thus, the opening had little effect on the flexural behavior of the wall. The full loops in Figs. 6 through 8 indicate that, with proper reinforcement, rectangular walls, tee-shaped walls, and rectangular walls with an opening at the base can exhibit stable hysteretic behavior and significant ductility.

## Strain Profiles at Wall Base

The strain profiles at the base of the wall were calculated from the LVDT's (Figs. 9 through 13). Strains were calculated by dividing the displacement recorded from the LVDT's by the gage length of approximately 9 inches (23 cm). The design strains were taken from moment-curvature analyses based on the curvature demand at the wall base for the design drift level 1.5 % (Wallace and Moehle, 1992). The strain profiles for the four solid walls are nearly linear, thus the assumption of a linear strain distribution is reasonable even though cyclic loads were applied to the wall specimens and the analytical procedure is based on a monotonic moment-curvature analysis.

For the walls with openings, when the lateral drift exceeds approximately 0.75% drift, the strain profiles become nonlinear. The column and the panel act somewhat independently in flexure, resulting in greater curvature in the panel and in the column. This is most prominent in the column (Figs. 10 and 11). This increase in curvature at the base of the column was accompanied by a decrease in curvature at the top of the column, indicating that the column was subjected to reverse curvature.

For the tee-shaped walls with the flange in compression (Fig. 12), very low strains are predicted and measured; therefore, evaluation of this case is not critical. For the T-shaped wall with the flange in tension (Fig. 13), large compression strains developed in the web. It is noted that the design strain distribution plotted

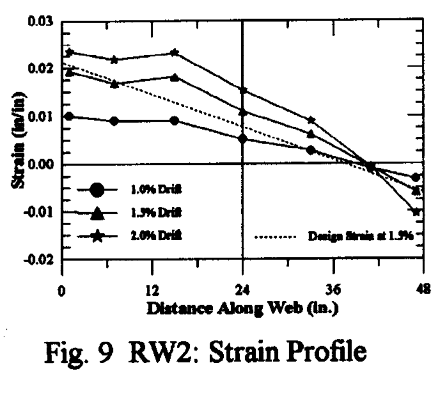


Fig. 9 RW2: Strain Profile

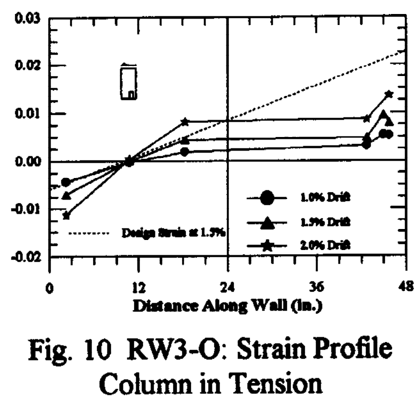


Fig. 10 RW3-O: Strain Profile
Column in Tension

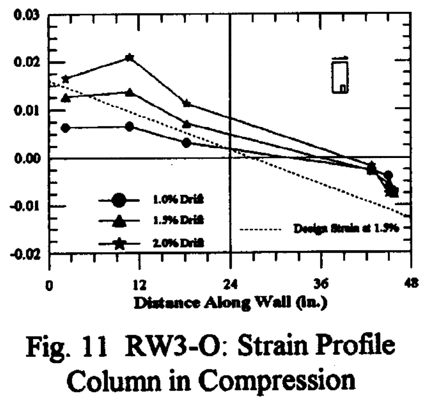


Fig. 11 RW3-O: Strain Profile
Column in Compression

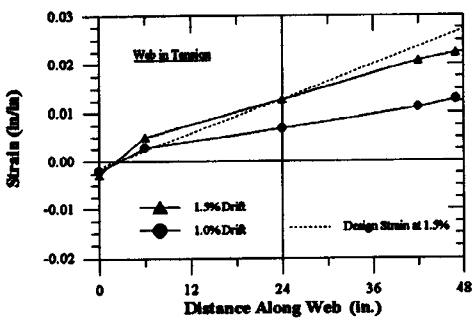


Fig. 12 TW2: Strain Profile - Web in Tension

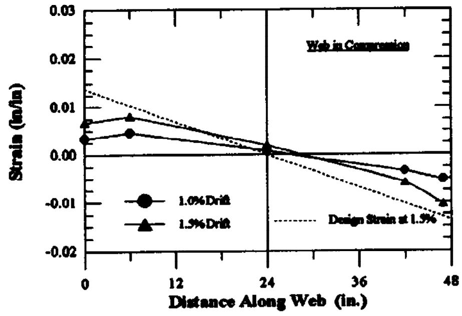


Fig. 13 TW2: Strain Profile - Web in Compression

in Fig. 13 is based on assuming the entire tension flange is effective; experimental results indicate that this was a reasonable assumption. In addition, the experimental results plotted in Fig. 13 have not been corrected for pedestal rotation which accounted for approximately 0.3% drift at an imposed drift level of 1.5%; therefore, the comparison between experimental and analytical strain distributions would be slightly better if this effect were included. The large compressive strains that develop at the web boundary are much greater than those for the rectangular walls due to the contribution of the flange reinforcement as tension steel. This high strain must be identified when evaluating the need for transverse reinforcement; therefore, tee-shaped walls should not be designed simply as two rectangular walls joined together and the influence of the flange must be evaluated. For evaluating detailing requirements at the web boundary (as well as shear reinforcement), a high estimate for the effective flange width should be used to ensure adequate performance.

## Moment - Curvature Relations

The experimental moment-curvature relations for the rectangular wall with an opening are compared with analytical results at the first peak of each drift level in Figs. 14 and 15. In addition, the design ultimate curvature is noted on the figure for reference. The analytical results were calculated using the BIAX (Wallace, 1992) computer program with a Saatcioglu & Razvi (1992) model for confined concrete. For the experimental curves, the moment was taken as applied horizontal load times the height to the point of application above the wall base (150 in; 3.81 m). The experimental curvature was calculated two ways: 1) as the slope of a best fit line through the strains determined from the LVDT's at the base of the wall, and 2) by dividing the rotation at the first story level by an assumed plastic hinge length of 30 inches (762 mm). For the solid walls, in general, very good agreement was observed between analytical and experimental moment-curvature relations (similar to results plotted in Fig. 14); therefore, these results are not shown. For the wall with an opening, the comparisons show that, although the actual strain distributions were not linear, both the peak moments and the curvatures were predicted very well, especially for the case with the column in tension (Fig. 14). For the case with the column in compression (Fig. 15), a reduction in stiffness is noted and the flexural capacity is overestimated using a monotonic moment-curvature analysis. The reduction in stiffness is likely due to the Bauschinger effect caused by cyclic loading. The slightly under predicted ultimate moment

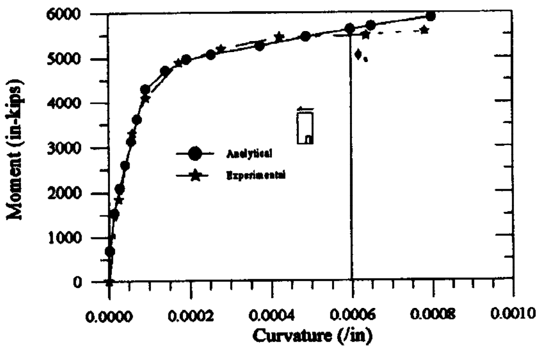


Fig. 14 RW3-O: Moment-Curvature Column in Tension

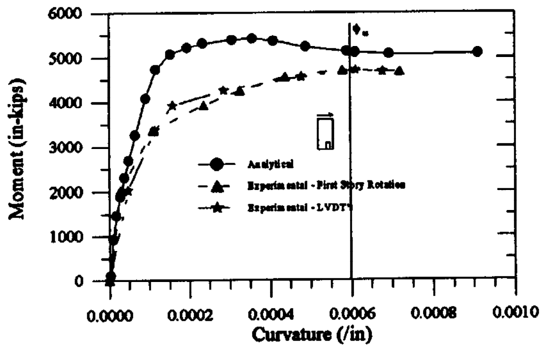


Fig. 15 RW3-O: Moment-Curvature Column in Compression

capacity is attributed to the contribution of the diagonal compression struts to the stress in the longitudinal steel.

### Strut and Tie Models

Although, for solid walls with the same overall dimensions, shear was found to play a small role in the specimen behavior, for the specimens with openings, shear contributed approximately 25% of the overall top lateral displacement. As well, conventional code approaches are ineffective for walls with relatively large openings. Strut and tie models were found to be a flexible and effective tool for the shear design of discontinuous regions. Provided a reasonable model is selected, this method will result in a conservative design. A reasonable model is one in which the orientation of model elements does not vary too far from the elastic principle stresses. Strain gage data indicated that the additional horizontal web reinforcement provided above the opening to drag the shear back into the wall panel at the base of the wall was effective. The strain gage data also revealed that the load path assumed in the design process was not always followed when alternative, stiffer, load paths were available. Although the strut and tie model did not indicate a need for vertical web reinforcement, and analytical studies suggested that diagonal reinforcement should be provided, a mesh of horizontal and vertical web reinforcement was provided. Test results indicate good inelastic performance with this reinforcing arrangement.

## CONCLUSIONS

In areas of high seismic risk, structural walls are not designed to remain elastic during a severe earthquake; therefore, inelastic deformations are expected, usually at the base of the wall. In order to exhibit stable, inelastic behavior, the wall must be specially detailed, that is, transverse reinforcement must be provided in regions of high strain. Experimental results show that, when properly reinforced, slender structural walls can exhibit stable hysteretic behavior and significant ductility.

The experimental studies presented in this paper have shown that displacement-based design methodologies are an effective tool for evaluating structural wall behavior. Using a displacement-based approach results in wall designs that are directly related to the building configuration, as well as the wall aspect ratio, the wall axial load, the wall cross sectional configuration, and the wall reinforcing ratios. This procedure worked well to ensure that the concrete in the boundary elements had adequate confinement to prevent crushing at high strains. The agreement between the predicted and experimental strain distributions and moment curvature relations verify that displacement-based design procedures are appropriate for design and evaluation of slender structural walls. For slender walls with openings, combining a displacement-based design approach for evaluating required transverse reinforcement at the wall boundaries with a strut and tie model to evaluate required shear reinforcement proved effective. Although measured strain distributions were not linear for the walls with openings, comparisons between predicted and measured moment-curvature relations indicate that reliable results can be obtained.

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