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CYLIC TESTING OF REINFORCED CONCRETE STRUCTUAL WALLS WITH ORDINARY BOUNDARY ELEMENT DETAILING

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Abstract

The use of reinforced concrete walls to resist lateral demands from earthquakes is common. Two large-scale reinforced concrete walls were tested under a constant axial gravity load of $0.053A_gf'_c$ with reversed-cyclic, quasi-static lateral loading and overturning moment applied at the top of the wall. The walls had rectangular cross-sections, with differing amounts of boundary longitudinal reinforcement, and without ACI 318-11 Special Boundary Elements. The primary objective of the study was to ascertain the deformation capacity (plastic hinge rotation capacity) of walls with modest-to-light detailing. Damage initiated with spalling of cover concrete and buckling of wall longitudinal reinforcement, followed by eventual fracture of wall longitudinal reinforcement and crushing of the concrete core. Minimal strength degradation was evident in both specimens tested through loading cycles that produced 1.0% plastic hinge rotations. Significant strength degradation and damage occurred during loading cycles to 1.5% plastic hinge rotation. Failure occurred during the third loading cycle at 1.5% plastic hinge rotation capacities are similar to that obtained in recent tests of thin walls with ACI 318-11 Special Boundary Elements, likely due to the thicker wall sections and lower axial loads reducing the compressive strains at a given plastic hinge rotation and reducing the likelihood of lateral instability.

Keywords: reinforced concrete, wall, structural wall, shear wall, boundary element



1. Introduction

Reinforced concrete structural walls are often used in reinforced concrete buildings to resist lateral demands from earthquakes and wind. During strong ground motions, a first-mode-dominant response is common in buildings, causing yielding at the base of reinforced concrete walls to concentrate over a plastic hinge length that extends vertically upward from the base of the wall. Lumped plasticity models, where plasticity is concentrated at the base of the structure, may be used to capture this behavior (e.g., [2]). In this modeling approach, plastic hinge rotation is proportional to the design lateral deformations at the top of the structure, which are used to assess to assess the need for boundary element detailing when conducting displacement-based design of walls ([3], [4], and [5]). In regions of the United States expected to be subjected to strong ground motions, Special Boundary Elements (SBEs) detailed in accordance with ACI 318-11 Section 21.9.6.4 are typically required at the lower stories of mid-rise and high-rise walls. The intent of SBEs is to provide a level of lateral confinement of boundary element concrete and a level of restraint against buckling of longitudinal reinforcement that will promote a ductile response of the wall in bending. Because significant yielding is not expected to occur above the plastic hinge region, SBEs are not required at this location in the wall, and boundary transverse reinforcement requirements are instead based on ACI 318-11 Section 21.9.6.5. In this study, two reinforced concrete structural walls without SBEs were tested to failure under constant axial load and quasi-static, reversedcyclic loading (shear and moment). The primary objective of this study was to assess the performance of flexure-controlled walls with modest axial loads and with boundary transverse reinforcement required by ACI 318-11 Section 21.9.6.5.

2. Test Specimens

Two specimens, referred to as W1 and W2, were designed and tested in the UCLA Structural Earthquake Engineering Laboratory. The test specimens were designed to be roughly one-half-scale based on an assumed prototype. The cross-sections for both walls were rectangular with an asymmetric layout of reinforcement at the boundaries (Figure 1). The primary test variables were the quantity of wall boundary longitudinal and transverse reinforcement, which led to variation in the longitudinal and transverse reinforcement ratios and the s/d_b ratio, which is the ratio of the vertical spacing of boundary transverse reinforcement, s, to the bar diameter of the boundary longitudinal reinforcement. The s/d_b ratio indicates an increased likelihood for buckling of boundary longitudinal reinforcement. The s/d_b ratios at each end of the wall were 5.3 and 4.6 for W1 and 6.4 and 16.0 for W2; therefore, the likelihood of bar buckling is significantly increased at one boundary of W2 relative to the other wall boundaries.

An intermediate level of wall boundary transverse reinforcement is sometimes referred to as an Ordinary Boundary Element (OBE) (as used in [6]). An OBE provides a lower level of lateral confinement of concrete and restraint against buckling of longitudinal reinforcement than that of an ACI 318-11 Section 21.9.6.4 Special Boundary Element (SBE). An OBE is required when an SBE is not required and the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$ (f_y in ksi) or $2.76/f_y$ (f_y in MPa). This limit corresponds to $\rho = 0.0067$ for reinforcement with a specified yield strength of 60-ksi = 414-MPa, which is common is the United States. In regions of the United States expected to be subjected to strong ground shaking, OBEs are often used in mid-to-upper-level stories in high-rise reinforced concrete buildings.

At both wall boundaries of W1 and at one boundary of W2, reinforcement was provided to satisfy provisions for an intermediate level of wall boundary transverse reinforcement (i.e., for an OBE), which requires 203-mm maximum vertical spacing of this reinforcement. Assuming 13-mm diameter hoops and ties at 203-mm spacing for the prototype led to 6.4-mm diameter hoops and ties at 102-mm spacing for the test specimens. At one boundary of W2, light boundary reinforcement excluding hoops and ties was provided. In regions of the United States expected to be subjected to strong ground shaking, the upper-level stories in a typical high-rise wall are often lightly-reinforced, sometimes without boundary hoops and ties. Per ACI 318-11 Section 21.9.6.5(a), minimum wall boundary transverse reinforcement is permitted when the ratio of longitudinal reinforcement at the wall boundary is less than or equal to $400/f_y$ (f_y in ksi) or $2.76/f_y$ (f_y in MPa. This led to the use of 14 - 9.5mm Grade 60 ($f_y = 60$ -ksi = 414-MPa) longitudinal bars at the W2 wall boundary ($\rho = 0.0061$) with minimum



boundary transverse reinforcement. Per ACI 318-11 Section 21.9.6.5(b), this minimum reinforcement consisted of 9.5-mm U-bars engaging wall boundary longitudinal reinforcement and spliced to the 9.5-mm web transverse reinforcement at both ends of the wall. For W1, the horizontal web bars were embedded (without hooks) into the cores of the ordinary boundary elements. It is noted that for both W1 and W2, longitudinal and transverse wall web bars were selected to meet the code minimum reinforcement ratio, which is $\rho_l = \rho_t = 0.0025$ per ACI 318-11 Section 21.9.2.1.

Both of the walls tested in this study were used previously for testing of steel reinforced concrete (SRC) coupling beams [7]. This resulted in damage at the beam-to-wall connections, which were located slightly above mid-height of the walls at each end. This damage was repaired prior to testing of the walls, as shown in the photos in Figure 2. The repairs at this region included removal of the embedded steel beams for W2, removal of damaged concrete, grouting of damage regions, and the use of steel plates and post-tensioned threaded rods to provide confinement and prevent spalling of concrete. Because the coupling beam centerlines were located 1676-mm above the base of the 2438-mm long walls, a successful repair was not expected to improve the post-yield performance of the specimens, as damage for the flexure-controlled walls was expected to concentrate at the base of the walls. During prior testing of the SRC coupling beams, demands at the base of the wall produced flexural cracking, but generally did not cause yielding of wall boundary longitudinal reinforcement.



Fig. 1 - Wall Cross-Sections (Dimensions in millimeters)



Fig. 2 – Photo of Repairs to Walls Prior to Testing (Dimensions in millimeters)

3. Material Properties

A summary of material properties, including the tested yield stress ($f_{y,test}$), ultimate stress ($f_{u,test}$) and % elongation (% elong.), for the reinforcement used in the test specimens is provided in Table 2. All reinforcement 9.5-mm in diameter or larger was U.S. Grade 60 (specified yield stress of 60-ksi = 414-MPa), with the values in Table 2 coming from the provided mill certificates. The wall boundary transverse reinforcement consisted of 6.4-mm diameter U.S. Grade A36 undeformed bars (minimum specified yield stress of 36-ksi = 248-MPa), with the values in Table 1 taken as the average from three tensile tests.

	W1					W2			W1 & W2
Bar Size	#7	#6	#5	#4	#3	#5	#4	#3	#2
$f_{y,test}$ (MPa)	490	534	474	497	441	426	497	469	351
$f_{u,test}$ (MPa)	659	717	720	752	707	577	739	724	499
% elong.	19	14	15	13	14	19	15	14	21

Table 1 – Material Properties

Prior to testing W1 and W2, high-strength (~69.0-MPa = 10-ksi) hydro-stone grout was used to repair wall damage that occurred during coupling beam testing (Motter et al, 2014). Each test specimen included a construction joint that was located 1905-mm above the base of the wall, i.e., at the top of the coupling beams that were tested by Motter et al (2014). Based on average values from tests on 152-mm x 305-mm concrete cylinders, the tested concrete strength, $f'_{c,test}$, for the $f'_c = 31.0$ -MPa mix was 50.1-MPa and 47.7-MPa for concrete below and above the construction joint, respectively, for W1 and 40.7-MPa and 39.7-MPa for concrete below and above the construction joint, respectively, for W2.

4. Test Set-Up and Testing Protocol

Using the test set-up shown in Figure 3, both specimens were tested at the University of California, Los Angeles Structural/Earthquake Engineering Testing Laboratory. The specimens were subjected to a constant axial (gravity) load of 1220-kN ($0.053A_gf'_c$, corresponding to $0.032A_gf'_{c,test}$ for W1 and $0.040A_gf'_{c,test}$ for W2) and quasi-static, reversed-cyclic lateral loading and overturning moment applied at the top of the wall. Two 1780-



kN (max.) actuators with +/- 457-mm stroke were used to apply wall overturning moment. The wall lateral shear force (and additional moment) was applied by a 1335-kN (max.) actuator with +/- 305-mm stroke and was reacted by the reaction blocks. The reaction blocks were stacked, grouted at the interfaces, and post-tensioned to the laboratory strong-floor with 32-mm diameter high-strength post-tensioning rod. The "concrete top beam" refers to a thickened portion at the top of the shear wall that was poured continuously with the upper wall. The concrete top beam facilitated anchorage to the steel loading beam that was positioned across the top of the specimen. The concrete top beam was post-tensioned to the steel loading beam, and the footing was post-tensioned to the laboratory strong floor. To prevent out-of-plane displacement or torsion of the specimen during testing, a steel brace (not shown) was attached to the steel loading beam. Loading jacks at the top of the steel loading beam were used to apply axial compressive load to the top of the wall. Additional axial load was applied by the two vertical actuators.



Fig. 3 – Test Set-Up (Dimensions in millimeters)

The applied moment gradient in the walls was constant and linear throughout testing, with the ratio of wall top moment to wall base moment held at 0.79. The selection of this ratio was determined by analyzing an idealized 10-story structure with equal story heights and story masses using the equivalent lateral force procedure in accordance with ASCE 7-10 Section 12.8 [8]. Using this procedure the lateral force at story *x*, denoted F_x , is determined as $F_x = C_{vx}V$ (Equation 12.8-11), where *V* is the base shear and the C_{vx} is the vertical distribution factor, computed as (Equation 12.8-12):

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \tag{1}$$

where w_i and w_x are the portion of the total effective seismic weight of the structure (W) located or assigned to Level *i* or *x*, and h_i and h_x are the height from the base to level *i* or *x*. *k* is an exponent related to the structure period, with k = 1 for structures having a period of 0.5 seconds or less, k = 2 for structures having a period of 2.5 seconds or more, and *k* linearly interpolated for structures having a period between 0.5 and 2.5 seconds. Based on Eq. (1), and assuming that the prototype structure had equal story heights and story seismic weight, the relative proportioning of story lateral forces and the corresponding relative moment demand over the height of the structure becomes a function of the period of the structure. Although the period influences the overall shape of the moment diagram, minimal variation in the moment gradient is evident at the lower stories (Figure 4). The test specimen, 2.74-m (9-feet) tall at approximately one-half-scale, was assumed to represent the lowermost 1.5



stories within an actual structure. For the idealized 10-story prototype, the ratio of the overturning moment at story level 1.5 to that at the base varies between 0.79 and 0.81 for all possible structural periods (Figure 4). Given the minimal variation in this parameter as a function of period, a value of 0.79 was selected for testing. In order to maintain this ratio, the ratio between the overturning moment applied by the vertical actuators and the applied wall shear load was held constant at 30.8', as the applied cyclic load in each vertical actuator was held at 1.4 times the magnitude of the load in the horizontal actuator. As the lateral load was applied 11.6' above the base of the wall, the effective cantilever height was 42.4', corresponding to a shear span ratio of 42.4'/(lw=8') =5.3. The 42.4' effective cantilever height is 0.71 times the building height, assuming equal story heights of 6' for the one-half scale 10-story building. Note that lateral load was not applied at story level 1 (6' above the base), as this lateral load imparts no significant change to the moment diagram, which is nearly linear at the lower stories (Figure 4).



Fig. 5 – Loading Protocol

-0.05

-0.1

-3

-3.5

Both tests were displacement-controlled using the plastic hinge rotation measured by a pair of LVDTs, located near opposite ends of the wall, spanning from the footing to the assumed plastic hinge length. The assumed plastic hinge length was taken as $l_w/2=1219$ -mm (where l_w is the wall length) for the first specimen. For the second specimen, this length was taken to the base of the steel plates used for repair, as the presence of the steel plates was expected to prevent the spread of plasticity into the repaired region. The bottom of each steel plate was located $3l_w/8=914$ -mm above the base of the wall. The loading protocols for both tests are shown in Figure 5. Relative to W1, additional low-level cycles were carried out for W2, as yielding was expected to occur at a smaller rotation due to the lower yielding moment associated with the reduced quantity of wall boundary longitudinal reinforcement. The loading protocols were identical between 0.125% and 1.5% rotation, except for the inadvertent loading from -0.25% rotation to -1.15% rotation (at a rate of roughly 0.07% rotation per second) during the first loading cycle at 0.25% rotation for W2. Three cycles were carried out at all loading increments



through 1.5% rotation. Beyond 1.5% rotation the testing protocol was based on the observed damage and response of the test specimens. Failure initially occurred at one wall boundary for both specimens. In an effort to reduce the likelihood of fracture of the longitudinal bars at this boundary, asymmetric loading was then used in order to limit the level of compression applied to the boundary that had failed. This was done in an effort to generate failure at the other boundary of the specimen, noting that fractured bars would reduce the compression demand on the boundary that had not failed and reduce the likelihood that this failure could be achieved.

5. Test Results

5.1 Observed Damage

For both walls, damage concentrated beneath the repaired region of the wall (Figure 6 and Figure 7) rather than at the base. This was due to pre-existing damage, noting that the applied wall demands were largest at the base. For W1, damage concentrated at the wall boundary with a lower s/d_b ratio (i.e., at the wall boundary with 22-mm diameter longitudinal reinforcement), which was likely the result of pre-existing damage, noting that this wall boundary was damaged more heavily during previous testing (Motter et al, 2014). For W2, damage predominantly occurred at the wall boundary with a higher s/d_b ratio (i.e., with 9.5-mm diameter longitudinal reinforcement). With reference to the loading protocol shown in Figure 5, positive loading put the damaged wall boundary into compression for both W1 and W2.



Fig. 6 – Damage photos for W1 at: Boundary with $s/d_b = 5.3$ at: a) +1.5% rotation, 1st cycle (north face); b) +1.5% rotation, 1st cycle (south face); c) +1.5% rotation, 3rd cycle. Boundary with s/db = 4.6 at: d) -2.25% rotation

For W1, damage at the wall boundary with the lower s/d_b ratio was observed to occur at one face prior to the other, likely due to pre-existing damage. At one face of this boundary, minor damage was evident at 0.375% rotation with longitudinal bar bucking initiating during the first loading cycle at 1.0% rotation. Bar buckling at this face had become extensive after the positive excursion of the first loading cycle at 1.5% rotation (Figure 6a) but was not observed at the other face of this boundary element (Figure 6b). After the 3rd cyclic excursion to 1.5% rotation in the positive direction, significant bar buckling and spalling and crushing of some core concrete was observed (Figure 6c). At the other wall boundary, damage was much less extensive. Large splitting tensile cracks (~5-10-mm crack widths) and some spalling of cover concrete was observed after the first loading cycle

at 1.5% rotation. After the completion of the negative excursion to 2.25% rotation, significant spalling of cover concrete was observed but bar buckling was not evident (Figure 6d). Bar buckling at this boundary was eventually observed during a final negative excursion to 4.5% full-height rotation. During this final excursion, the corresponding plastic hinge rotation was unknown and not indicated on the plot in Figure 5, as some sensors were removed to avoid damage from spalling concrete.

For W2, no significant damage was observed through loading cycles at 1.0% rotation. After the first loading cycle to 1.5% rotation in the positive direction, some splitting tensile cracks, indicative of the onset of spalling of concrete cover, were observed at the wall boundary with the larger s/d_b ratio. After the first loading cycle at 1.5% rotation in the negative direction, large tensile cracks (~6-8-mm) were observed at this boundary (Figure 7a). After load reversal, longitudinal bar buckling occurred during the second cycle at 1.5% rotation in the positive direction. More extensive spalling of cover concrete and buckling of longitudinal boundary reinforcement was observed after the third loading cycle at 1.5% rotation (Figure 7b). Fracture of longitudinal reinforcement was observed after loading at 2.0% rotation in the negative direction (Figure 7c) with bar buckling of unfractured bars evident over multiple spacing intervals of transverse reinforcement after unloading (Figure 7d). At the other wall boundary, i.e., the wall boundary with smaller s/d_b ratio, a large splitting tension crack (~10-mm) formed during the second loading cycle at 1.5% rotation in the negative direction with bar buckling clearly evident after the third loading cycle at 1.5% rotation in the negative direction with bar buckling clearly evident after the third loading cycle at 1.5% rotation in the negative direction with bar buckling clearly evident after the third loading cycle at 1.5% rotation in the negative direction with bar buckling clearly evident after the third loading cycle (Figure 7e). Significant propagation of damage at this boundary was not observed during the remainder of the test.

Fig. 7 – Damage Photos for W2 at: Boundary with $s/d_b = 16.0$ at: a) -1.5% rotation, 1st cycle; b) +1.5% rotation, 3rd cycle; c) -2.0% rotation; d) zero load after -2.0% rotation. Boundary with $s/d_b = 6.4$ at: e) -1.5% rotation, 3rd cycle

5.2 Load-Deformation

Behavior in the repaired (i.e., upper) portions of the walls is likely unrepresentative of typical wall behavior due to the significant alteration of the wall section properties associated with the steel plate and post-tensioning system used to make the repairs prior to testing. Therefore, assessment of load-deformation response will focus on the non-repaired portion of the walls, i.e., the assumed plastic hinge region. For W1, minimal pinching and strength degradation is evident in the moment-rotation response (Figure 8) until failure occurred during the third loading cycle at 1.5% plastic hinge rotation. In this paper, failure is defined to have occurred when the load at the peak of a cycle initially drops below 80% of the peak load and does not return to this value. Failure occurred in the negative loading direction at a plastic (i.e., post-yield) rotation of roughly three times the yield rotation. In the negative loading direction, no significant strength loss was observed up to a plastic rotation of roughly six times the yield rotation. It is noted that the performance of an undamaged wall could potentially exceed that observed in this test.

Fig. 8 – Moment-Rotation

For W2, modest pinching and minimal strength degradation was evident in the moment-rotation response (Figure 8) through loading cycles at 1.5% rotation (neglecting strength degradation relative to the peak strength observed during inadvertent loading to -1.15% rotation), despite bar buckling at the wall boundary at 1.5% rotation (Figure 7b). The degree of pinching in the moment-rotation response was larger when loading from negative rotation to positive rotation, even for loading cycles at 1.0% rotation, which was prior to observation of significant damage. During the first loading cycle to 2.0% rotation (after completion of three cycles at 1.5% rotation), failure occurred at roughly 1.25% rotation in the plastic hinge. For W2, which had an intermediate level of boundary transverse reinforcement (ACI 318-11 Section 21.9.6.5(a)) at one end and a minimum level of boundary transverse reinforcement (ACI 318-11 Section 21.9.6.5(b)) at the other end, failure occurred at the wall boundary with minimum transverse reinforcement. The yield moment and yield rotation for W2 were lower than for W1. For W2, a plastic rotation in excess of six times the yield rotation was reached prior to failure, noting that the ductility of a wall with no prior damage may exceed that observed for W2. The deformation capacities for the two test specimens are similar to that observed in recent tests of thin walls with ACI 318-11 Special Boundary Elements [9]. Relative to the tests reported in [9], the thicker wall sections and lower axial loads for W1 and W2 likely reduced the compressive strains at a given plastic hinge rotation and reduced the likelihood of lateral instability.

6. Conclusions

Two large-scale walls with rectangular cross-sections and asymmetric boundary longitudinal reinforcement were tested to failure under a constant axial load of 0.053Agf'c and quasi-static, reversed-cyclic lateral loading and overturning moment applied at the top of the wall. The moment gradient in the test specimens was consistent with the lower 1.5-stories of a 10-story building. The primary test variables were the amount of boundary longitudinal and transverse reinforcement, influencing the ratio of transverse reinforcement spacing to longitudinal bar diameter (s/d_b), which varied from 4.6 to 16.0 at the four wall boundaries. The test specimens did not include ACI 318-11 (Section 21.9.6.4) Special Boundary Elements. An intermediate level of boundary transverse reinforcement was provided in accordance with ACI 318-11 21.9.6.5(a) for one of the test specimens. The other test specimen had an intermediate level of boundary transverse reinforcement at the other end, provided in accordance with ACI 318-11 21.9.6.5(b).

The observed failure sequence in these walls initiated with spalling of cover concrete and buckling of wall longitudinal reinforcement followed by eventual fracture of wall longitudinal reinforcement and crushing of the concrete core. Significant damage due to bar buckling was observed prior to failure, with failure defined to have occurred when the load at the peak of a cycle initially drops below 80% of the peak load and does not return to this value. For W1, failure occurred during the third loading cycle at 1.5% rotation, where the plastic rotation was roughly three times the yield rotation. For W2, failure occurred after the completion of three loading cycles at 1.5% rotation and during the first loading cycle to 2.0% rotation. At 2.0% rotation for the failure cycle, the plastic rotation was in excess of six times the yield rotation, noting that W2 had a lower yield moment and yield

rotation than W1. Had the two test specimens been tested in an undamaged state, the deformation capacity in the plastic hinge region would be expected to meet or exceed that of the walls tested in this study. The deformation capacities for W1 and W2 are similar to that obtained in recent tests of thin walls with ACI 318-11 Special Boundary Elements. This is likely due to the thicker wall sections and lower axial loads reducing the compressive strains at a given plastic hinge rotation and reducing the likelihood of lateral instability.

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