LOAD – DEFORMATION RESPONSES OF SLENDER STRUCTURAL STEEL REINFORCED CONCRETE WALLS

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ABSTRACT

Reinforced concrete structural walls are commonly used to provide lateral strength and stiffness to resist lateral loads from wind or earthquake. Slender walls usually require large amounts of longitudinal reinforcement at wall boundaries to satisfy code lateral strength requirements; therefore, use of structural steel reinforced concrete (SRC) walls, where the boundary longitudinal reinforcement is replaced by a structural steel section, is a potentially attractive alternative. This study presents results of an experimental program involving tests under combined gravity and lateral loading for three, heavily instrumented, moderate-scale SRC slender walls. Primary test variables included varying the size of the embedded structural steel section and the quantity of transverse reinforcement provided at the wall boundaries. Although slip of the structural steel was observed for all specimens, test results reveal good behavior up to and beyond 2% lateral drift, even for a specimen with only modest confinement at the wall boundaries.

Keywords: strength; slender; shear wall; structural wall, structural steel; reinforced concrete; experiment, test

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1. INTRODUCTION

Reinforced concrete structural walls are commonly used to resist the actions imposed on buildings due to earthquake ground motions because they provide substantial lateral strength and stiffness. The strength and stiffness are required to limit damage in more frequent earthquakes and the wall must possess sufficient wall deformation capacity to ensure the lateral load capacity is maintained during the inelastic response expected during stronger, less frequent, earthquakes.

Flexural reinforcement for structural walls is typically concentrated at wall edges, commonly referred to as boundary elements. The boundary elements carry the majority of the tension and compression forces that develop under the applied lateral loads, with lighter flexural reinforcement typically used within the wall web between the boundary zones. The boundary longitudinal reinforcement, which typically consists of a relatively large number of large-diameter reinforcing bars, must be enclosed within transverse reinforcement (hoops and crossties) to confine the core concrete and to restrain buckling of longitudinal reinforcement [1]. The relatively large quantities of longitudinal and transverse reinforcement at wall boundaries, as well as the anchorage of horizontal web reinforcement at the wall boundary, creates substantial congestion and complicates construction. A promising alternative construction approach is to replace a majority of the boundary vertical reinforcement (longitudinal reinforcement) with a structural steel shape commonly used for steel construction. This technique of embedding a structural shape within concrete is commonly referred to as steel reinforced concrete (SRC) construction.

Use of an SRC system may offer several advantages. For example, depending on the shape of the section used at the wall boundary, the embedded steel section may provide substantial confinement and requirements for transverse reinforcement might be relaxed over that required

by ACI 318-14 §18.10.6 [2] for reinforced concrete walls. As well, the embedded steel sections may not be as susceptible to buckling as reinforcing bars, and might also delay buckling of the supplemental deformed longitudinal boundary reinforcement (in addition to the structural steel section), leading to an improved behavior relative to RC construction. Use of SRC construction might also enable more rapid construction, as the structural steel boundary columns could be used to support formwork and construction loads for several levels above the level reached for concrete placement.

Although SRC systems offer advantages, observations have shown that several potential problems exist. The embedded steel section may slip due to lack of sufficient bond stress, or the encased steel section may act like a wedge and split the surrounding concrete [3], thereby reducing the effective nominal flexural strength. Also, it has been observed that the restraint against lateral expansion provided by encased steel sections can cause vertical web splitting along the interface between the embedded steel section and the wall web [4]. Given these problems, there is a need to conduct large-scale tests to assess requirements for lateral strength and boundary detailing, as well as assist in the development of design recommendations.

The use of composite sections has become more common for frame structures, e.g., use of structural steel encased columns, where the concrete around the structural steel section also provides fire protection, or alternatively, steel sections on the outside of concrete (e.g., concrete filled tube), where the casing also serves as formwork. The use of steel sections in shear walls has been included in short walls (i.e. [5, 6]), as well as, slender walls. Cho et al. [7] tested 3 specimens as cantilever walls with a point cyclic lateral load with a height-to-length aspect ratio ($AR = h_w/l_w$) of 3.75. A constant axial load of either $0.10f'_cA_g$ or $0.15f'_cA_g$ was applied, and longitudinal reinforcement was provided by either a steel section (2 specimens) or traditional

reinforcing (1 specimen), all with similar tensile strength. For the specimens with steel sections, either HSS (hollow structural section) or channels were placed at both wall boundaries (no cover concrete), and connected by web horizontal reinforcement. The experimental results indicate that similar stiffness and flexural strength values were observed for all three specimens, with slightly more deformation capacity prior to significant strength loss for the specimen with traditional longitudinal reinforcement. For specimens with steel sections, strength degradation was observed to coincide with the onset of buckling of the steel section, followed by significant concrete crushing, which was more severe for the specimen with steel channels. Zhou et al. [8] tested 16 SRC cantilever wall specimens subjected to a point cyclic lateral load and constant axial load between $0.09f_c A_g$ and $0.24f_c A_g$. Wall aspect ratios (AR) ranged from 0.8 to 3.75, with four, slender wall tests ($AR \geq 2$). Primary test variables were aspect ratio and axial load, with test specimens designed to match the amount of reinforcement that would exist for the same walls designed with traditional longitudinal reinforcement. Test results were consistent with expectations for specimens with traditional reinforcement, specimens with smaller aspect ratio or larger axial load yield reduced ductility. Dan et al. [9] tested 6 slender specimens ($AR = 2.6$), 4 with I-shaped steel sections (different section orientation and one specimen with the steel section not embedded), one with square (tube) sections, and a control specimen with traditional deformed longitudinal reinforcement. Test specimens were cantilever walls with a point cyclic lateral load applied at the top and a constant low axial load of about $0.02f_c A_g$ for all specimens. The five specimens with steel sections also included some longitudinal reinforcement enclosed in hoops at the wall boundaries (four $\phi 10$ longitudinal bars enclosed within $\phi 8$ hoops at 150mm vertical spacing – similar to the control specimen), except for the specimen with steel section not embedded into the boundary. All specimens reached lateral drift ratio exceeding 4% prior to

1 significant lateral strength degradation, with the lowest drift capacity observed for the control
2 specimen, where concrete crushing and rebar buckling was observed earlier than in SRC test
3 specimens. The test results indicate that the boundary detailing provided by widely-spaced hoops
4 ($s/d_b=150/10=15$) is less effective than providing a steel section. Qian et al. [10] tested 7
5 cantilever wall specimens with $AR = 2.1$ with a point cyclic lateral load applied at the top of the
6 wall and constant (large) axial load ($\sim 0.3f_c A_g$). Primary test variables were boundary element
7 detailing (configuration and spacing) and boundary element geometry (barbell section). Steel
8 tubes were used for primary longitudinal reinforcement for six SRC specimens. The
9 displacement ductility measured at 15% strength loss from peak load was 3 to 4 for SRC
10 rectangular walls, ~ 8 for the barbell wall, and ~ 5 for the specimen without steel section
11 (traditional). In the case barbell wall, the smaller neutral axis depth relative to walls with
12 rectangular cross-sections, results in a smaller yield displacement (stiffer specimen) and smaller
13 extreme fiber compressive strains, for the same drift level, yielding crushing at larger drift levels.
14 In the case of the RC specimen, the yield displacement was much smaller ($\sim 5\text{mm}=0.002\%$ drift)
15 compared to the SRC specimens ($\sim 8\text{mm}=0.003\%$ drift) given the smaller amount of total
16 longitudinal reinforcement (no steel section) placed at the boundaries. Therefore, even though
17 strength loss for the RC specimen occurred at a smaller lateral drift (1%, compared to the SRC
18 specimens – no barbell – as 1.2% in average), the displacement ductility ratio was larger. Similar
19 variation in displacement ductility between the RC and SRC specimens is observed for
20 specimens with similar strength, provided that the spacing of transverse reinforcement is low
21 (e.g., $s/d_b=8$ [11]). Lu and Yang [12] tested three, cantilever T-shaped walls with a point cyclic
22 lateral load applied in the plane of the wall web. Specimens were reinforced with I-shaped steel
23 sections at the flange and web edges (including the intersection). Identical detailing was used for

all specimens even though the axial load level was varied between $\sim 0.1f_c A_g$ to $\sim 0.28f_c A_g$. Test results confirmed that higher axial load levels result in smaller deformation capacity at strength loss, although variation of detailing with the level of axial load would help establish the adequacy of the design approach.

The previous work has shown the capability of SRC walls to obtain similar structural behavior as RC wall construction; however, little attention has been given to boundary detailing of SRC walls, as primary test variables have been axial load, aspect ratio or cross-sectional shape; however, different detailing was not provided in the test specimens despite the variation in the applied axial load or the imposed lateral displacement demands, which influence the depth of the neutral axis and the magnitude of the axial tension and compressive strains, respectively. In the present work, experimental results are presented for three moderate-scale, slender walls to examine behavior for combined gravity and lateral loading as well as to assess the validity of common design assumptions and provisions, e.g., for nominal moment capacity and for boundary element detailing used for reinforced concrete walls (such as ACI 318-14) and often applied to SRC walls. Special attention is given to detailing of boundary transverse and web horizontal reinforcement at wall boundaries.

2. EXPERIMENTAL PROGRAM

a. Prototype and Specimen Description

A typical 15-story office building located in UBC-97 Zone 4 on a stiff soil served as a prototype building to help guide the design of the test specimens [13]. The prototype building plan extends 180 ft (54.9 m) in one direction and 110 ft (33.5 m) in the other direction. Three reinforced concrete structural walls, arranged in a C-I-C-shaped configuration (placed in the

shortest building direction), are located at the center of the building. This system of core walls is connected at each level by a set of 4 coupling beams separating the walls by 8 ft (2.4 m). Floor heights are assumed to be 12 ft (3.7 m). A dead load of 125 psf (0.006 MPa) and a live load of 50 psf (0.0024 MPa) were assumed for each level of the structure. The prototype C-shaped wall had a flange dimension of 11 ft (3.4 m), a web dimension of 30 ft (9.1 m) and a thickness of 18 in (0.46 m). Half of the C-shaped section could be tested as an equivalent T-shaped, but due to testing limitations, only a rectangular section was considered for testing as the scaled specimen using the flange of the C section as the web of the rectangular section. It was determined that an 18 in (0.46 m) by 24 in (0.61 m) concrete coupling beam yielded a level of coupling that was appropriate for the prototype building, which is consistent with common used coupling beam depths. The beam size selection resulted in relatively high coupling between walls, resulting in a moment-to-shear length ratio of $M_u/V_u l_w = 3.4$. Design was based on $R_w = 8.0$ and an estimated fundamental period of 0.98 seconds. The average axial load was estimated as $0.15A_g f'_c$. The section analysis for axial-bending behavior, assuming Grade 60 reinforcing steel (60 ksi – 414 MPa, yield strength), resulted in a longitudinal reinforcement area of 37.2 in² (240 cm²) at the flange-web boundary and 7.3 in² (47 cm²) at the web boundary, for each leg of the C-section. Based on this analysis, a W12x106 section surrounded by 8 - #11 bars was selected for longitudinal reinforcement at the flange-web boundary. A W12x40 section surrounded by 8 - #8 bars was used at the web boundary. In both cases, about 2/3 of the force is taken by the steel section (assumed to be of Grade 50 steel – 50 ksi, 345 MPa yield strength).

The test specimens were approximately one-third scale replicas of the prototype walls due to testing limitations (height, maximum loads). The final test geometry, materials, and reinforcement for the test walls were selected to examine important design parameters, such as

the quantity of longitudinal reinforcement used at the wall boundaries, axial stress due to gravity load, neutral axis depth as a fraction of total wall length, and the concrete compressive strain at the wall boundary for an assumed design displacement equal to 1.5% lateral drift. Wall detailing requirements were assessed using the displacement-based approach provided in ACI 318-99 [14] for reinforced concrete walls. The ACI 318-99 provisions were unchanged until the ACI 318-14 [2] code, which includes a provision for minimum wall thickness (which would not impact the design in this case), and a provision to design for 1.5 times the design displacement (which would require a larger boundary element depth).

Three specimens, with 6 in. (152 mm) thick and 48 in. (1.22 m) long cross-sections, were constructed and tested. The height between the application of lateral load near the top of the wall and the critical section at the base of the wall was 192 in. (4.88 m); therefore, the ratio of the wall height to wall length (h_w/l_w) = 4 (**Figs. 1 and 2**). Relatively slender walls were tested because the use of SRC wall construction is more likely to be economical for taller buildings. Specimen SRCW1 and SRCW2 were subjected to an axial load of $0.10A_gf'_c$ (641 kN). SRCW3 was identical to specimen SRCW1, except that the axial compressive load was increased to $0.18A_gf'_c$ (1184 kN), but maintained at $0.17A_gf'_c$ (1068 kN) for a majority of the test and the spacing of special transverse reinforcement at one boundary of the wall was decreased from 4 in. (102 mm) to 3 in. (76 mm). **Fig. 1** shows plan views for the three specimens at two heights and **Fig. 2** shows elevation views of the specimens.

The primary longitudinal reinforcement at each wall boundary consisted of a W6x9 section ($A_{bl} = 1729 \text{ mm}^2$) for specimens SRCW1 and SRCW3, and a W6x16 ($A_{bl} = 3058 \text{ mm}^2$) for SRCW2. The W sections were surrounded by 8 - #4 ($A_{b2} = 1032 \text{ mm}^2$) longitudinal bars that served as secondary vertical boundary reinforcement and provided a means to support the

transverse reinforcement used at the wall boundaries. A 25 mm thick, 127 mm by 203 mm base plate was attached to the base of each steel section and embedded in a support block at the base of the wall used to anchor the test specimen to the strong floor for testing.

The web reinforcement for each wall consisted of two curtains of #3 ($d_b = 9.5\text{mm}$) deformed bars spaced at 152 mm on center in both the vertical and horizontal directions, terminated with 90 degree hooks. Holes were drilled in the web of the structural steel boundary columns for crossties (**Figs. 1, 3**).

Hoops and crossties at wall boundaries were made using deformed #2 bars ($d_b = 6.35\text{ mm}$). The vertical spacing of the hoops/crossties was 51 mm and 102 mm for SRCW1 and SRCW2, for each side (Fig. 2a and 2b), and 51 mm for one boundary zone and 76 mm for the other boundary zone for SRCW3 (Fig. 2c). The vertical spacing of the perimeter hoop and the crosstie was continued from the wall base to a height of 1.27 m above the base based on an assumed upper-bound plastic hinge length equal to the wall length ($l_p^{max} = l_w = 1.2\text{ m}$). The vertical spacing of the hoops above this region was increased to 152 mm. Crossties were provided along the web over the bottom 0.61 m of the wall, as shown in **Fig. 1a**, to aid construction and to avoid potential vertical web splitting observed in the Loma Prieta Earthquake in this type of wall [4].

b. Materials

The structural steel sections (W6x9 and W6x16) were A572, Grade 50. Stress vs. strain relations for steel coupons obtained from the steel sections were tested according to ASTM A370-97 (**Fig. 4a**). The design concrete compressive strength for each SRC shear wall specimens was 34.5 MPa; however, actual average concrete compressive strengths of 45.4, 22.7

and 45.0 MPa for foundation, first and second stories, and third and fourth stories (see **Fig. 1**), respectively, were obtained (concrete was placed in three lifts). The variation of the design and actual concrete strengths did not significantly impact the expected test results, since behavior is governed by flexure and actual material properties could be used in analytical studies. Three types of Grade 60 deformed reinforcing bars were used for the study: #4 ($d_b = 12.7$ mm) bars as boundary element longitudinal reinforcement, #3 ($d_b = 9.5$ mm) bars as web longitudinal and transverse reinforcement, and #2 ($d_b = 6.4$ mm) bars as boundary element transverse reinforcement. Representative stress-strain curves for all bars are shown in **Fig. 4b**.

c. Wall Boundary Detailing

Transverse boundary reinforcement was selected using the displacement-based approach of ACI 318-99 [14] assuming a design drift level of 1.5%, which is a reasonable maximum value for typical US buildings [1, 15]. Based on the prescribed levels of axial load for each wall, section analyses were performed ($\varepsilon_c = 0.003$) and the depth of the neutral axis depth c , measured from the extreme compression face of the wall, was found to be 302 mm, 320 mm, and 411 mm for specimens SRCW1, SRCW2, and SRCW3, respectively. Using the approximately relationship presented by Wallace and Orakcal [1] the maximum concrete compressive strain at the wall boundary is:

$$\varepsilon_{c,\max} = 2 \left[\frac{\delta_u}{h_w} \right] \left[\frac{c}{l_w} \right] = 2[0.015] \left[\frac{c}{1.22 \text{ m}} \right] \quad (1)$$

The expression assumes that all flexural deformation (constant curvature of $\varepsilon_{c,\max}/c$) is concentrated at the wall base over an assumed plastic hinge length of $l_w/2$. Note that the previous expression does not include the 1.5 amplification of δ_u added to ACI 318-14 to address

uncertainty (dispersion) in the design displacement [16]. Given the calculated values of neutral axis depth noted above, the maximum expected compressive strains at the wall boundary at the design drift are 0.0074, 0.0079, and 0.0101 for specimens SRCW1, SRCW2, and SRCW3, respectively. Since these values are beyond the strain at which concrete is assumed to degrade in compression ($\varepsilon_c = 0.003$), special transverse reinforcement is provided to confine the concrete and to restrain rebar buckling. For SRC walls, it is noted that the presence of the structural steel section at the boundary zone may suppress rebar buckling, since the embedded structural steel section will restrain the shortening needed to produce rebar buckling. The depth of required confinement zone is estimated using the following expression [1]:

$$c'' = c - \left[\varepsilon_{c,lim} / 2 \right] \left[\frac{l_w}{\delta_u / h_w} \right] = c - \left[\frac{0.003}{2} \right] \left[\frac{1.22 \text{ m}}{0.015} \right] = c - 0.1l_w \quad (2)$$

where c'' is the length of over which transverse reinforcement is required and $\varepsilon_{c,lim}$ is the limiting compression strain and c'' is not taken less than $c/2$ (values selected to be consistent with ACI 318-99 and later versions). Based on **Eq. 2**, transverse reinforcement is required over a depth of $c'' = 180$ mm for specimen SRCW1, 198 mm for SRCW2, and 290 mm for SRCW3, extending from the outermost compression fiber of the wall. For practical reasons, transverse reinforcement was provided over a depth of 241 mm for all three test specimens, although for SRCW3, this depth is slightly less than that required.

The quantity of transverse reinforcement required is determined using ACI 318-14 Table 18.10.6.4(f) [2] and is the same as would be required by ACI 318-99 [14]. The required spacing to satisfy ACI 318-14 cannot exceed 69 mm within the critical section. Additionally, it is required that the hoop/tie spacing not exceed $t_w/3 = 51$ mm per ACI 318-14 18.10.6.4(e). To restrain buckling of deformed boundary longitudinal reinforcement, a spacing limit of $6d_b$

(18.7.5.3(b)), or 76 mm given the #4 longitudinal bar size, is required. Given all these requirements, hoops and ties were spaced at 51 mm ($4d_b$) for one special boundary element, which satisfies the ACI requirements. Larger spacing values were used at the other wall boundary because the ACI provisions are conservative [17], as well as to investigate the potential benefit of the embedded structural steel section on reducing required transverse reinforcement. The required transverse reinforcement was provided over the bottom 1.3m based on ACI 318-99 (and -14) requirements (upper-bound plastic hinge length estimate). The eastern boundary element includes less transverse reinforcement (**Figs. 1 and 2**). The vertical spacing of boundary element transverse reinforcement was 102 mm for SRCW1 and SRCW2, and 76 mm for SRCW3, whereas the western boundary element was design with a spacing of 51 mm.

d. Shear strength

A capacity design approach was used to select web shear reinforcement. The nominal moment determined from the section analysis was divided by the height to the applied lateral load (4.9 m) to determine the shear at nominal moment; the design shear was taken as 1.25 times this value, similar to the approach used for special moment frames. A 90 degree hook was used on the horizontal web reinforcement at each wall boundary, with the hook angled slightly from the horizontal plane.

e. Steel Section Anchorage

It is important that the steel section be able to develop fully into the foundation to prevent premature anchorage failure at the critical section (wall-foundation interface). The anchorage capacity is typically provided through bond stress and end plate bearing. Although a pullout

(cone) failure should be checked in actual construction, the high-strength rods used to anchor the specimen foundation block to the strong floor precludes pullout failure for the test specimens.

The average bond stress capacity, derived by Roeder et al. [3] based on a series of tests on structural steel sections embedded in concrete columns, is calculated as:

$$f_B = 2.91 - 0.3(L/d) - 14.97\rho \text{ (MPa)} \quad (3)$$

where L is the embedded length of the steel section, d is the depth of the steel section, and ρ corresponds to the structural steel cross-section area to the composite area ratio. In the case of the wall specimens $L = 584$ mm for all specimens (pedestal height minus the plate thickness) and the steel ratio (ρ) is calculated using the same approach as given in ACI 318-14 with $h = 152$ mm (thickness of the wall section) and $(a+2x) = 254$ mm (steel section length plus twice the concrete cover). Based on these values, average bond stresses over the 584 mm embedment of 1.09 MPa and 0.58 MPa are calculated for SRCW1 and SRCW3 (W6x9 section) and for SRCW2 (W6x16 section), respectively. These bond stresses, multiplied by the section perimeter and the embedment of 0.58 m correspond to forces of 422 kN (W6x9) and 222 kN (W6x16), which is insufficient to develop the yield forces associated with the sections (595 kN and 1052 kN, respectively). Therefore, the bond strength given by embedment represents 71% and 21% of the required strength, respectively. The presence of cross-ties passing through the structural steel web inside the pedestal might improve the bond strength between the concrete and the structural steel section. For spacing of transverse reinforcement at the boundary elements of 102 mm, the number of cross-ties inside the pedestal was three (3), which would increase the bond strength by 40 kN ($2 \cdot 3A_{s,\phi 6.4} f_y / 2$), whereas for 51 mm spacing the number of stirrups was six (6) increasing the bond strength by 80 kN ($2 \cdot 6A_{s,\phi 6.4} f_y / 2$), assuming a shear yield strength of half

the tensile yield strength. Given the low strength of the crossties, they are not expected to significantly alter the observed test behavior.

The concrete bearing capacity is estimated as $1.3f'_c$ given that the foundation block is well-confined by transverse reinforcement. Based on this assumption, the plate (127 mm x 203 mm x 25 mm) bearing capacity is $P_b = A_{pl}(1.3f'_c) = 1502 \text{ kN}$. It is noted that the bond stress along the embedment length may deteriorate after numerous cycles as has been noted for headed bars anchored within beam-column joints [18]. However, the bearing capacity alone is sufficient to develop the yield strength of the embedded steel sections (596 kN and 1053 kN). Therefore, adequate anchorage capacity exists, although slip along the 584 mm embedment length is expected, especially for SRCW2 (W6x16).

f. Test Setup and Instrumentation

The wall specimens were tested in an upright position (**Fig. 5**) and anchored to the strong floor with steel tie-down rods. A specially fabricated steel load transfer assembly was used to transfer both axial and lateral loads to the wall specimen. An axial load of approximately $0.10A_gf'_c$ (641 kN) for all specimens except for SRCW3 was applied at the top of the wall by hydraulic jacks mounted on top of the load transfer assembly. SRCW3 was loaded with an axial force of approximately $0.18A_gf'_c$ (1184 kN), but was reduced to $0.17A_gf'_c$ (1068 kN) after a few cycles because one strand of one of the prestressing cables used to apply axial load was damaged during testing. The axial stress was maintained constant throughout the duration of each test. Cyclic lateral displacements were applied to the walls by a hydraulic actuator mounted horizontally to a reaction wall 4.88 m above the wall. Out-of-plane support was provided to prevent twisting of the wall specimen during testing. The walls were subjected to two complete

(push and pull) cycles of the following drift levels: 0.10%, 0.25%, 0.50%, 0.75%, 1.00%, 1.50%, 2.00%, 2.50%, 3.00%, and 4.00% (when possible).

Instrumentation was used to measure displacements, loads, and strains at critical locations for each wall specimen (**Fig. 5**). As well, instrumentation was provided to measure any horizontal slip between the foundation block and the strong floor, or rotation (uplift) of the foundation block (**Fig. 5**). Shear deformations were measured through the use of wire potentiometers mounted on the bottom three stories (in an “X” configuration) of each specimen (**Fig. 5**). Axial (vertical) displacements at the wall boundaries were measured using two wire potentiometers mounted directly to the wall ends and were used to calculate wall base rotations (by dividing the difference in relative axial displacements by the distance between the potentiometers). Similar sensors were located to assess rotation up to third floor. In addition, in specimen SRCW3, three additional vertical sensors were included in the first story at both edges to estimate the distribution of the deformations over the first story height. Axial displacements were measured along the length of each wall specimen near the base of the wall using linear variable differential transducers (LVDT's).

The strains in the reinforcing steel were (rebar and structural steel) measured through the use of strain gauges. All of the different types of reinforcement were monitored (longitudinal steel, uniformly distributed web steel, and transverse boundary steel) at various locations.

3. EXPERIMENTAL TEST RESULTS

Lateral force versus deformation responses for overall lateral displacement, flexure and shear are studied in this paper. General damage observations are included as well. Detail experimental information can be found elsewhere [19].

a. Observed Damage

The following section provides a description of the experimentally observed damage and behavior of each wall specimen based on visible damage (cracks, spalling, crushing, buckling, fracture, etc.).

SRCW1: At drift levels less than 2.0%, primary specimen cracking consisted of hairline horizontal cracks at the wall boundaries and hairline diagonal cracks in the wall web. Vertical splitting at the base of the SRCW1 was observed at 2.5% drift at the east end and 4.0% drift at the west end (**Figures 6a** and **6b**, respectively). At 2.5% drift, buckling of vertical bars at the edge of the wall was observed between 0.36 m and 0.51 m above the pedestal. At the 3.0% and 4.0% drift levels, the lateral strength of the wall degraded substantially for pull cycles (negative lateral displacement, i.e., compression in the eastern boundary element); however, lateral strength loss was not observed for positive lateral displacement, even at 4.0% drift. The loss of lateral strength occurred due to buckling (at 3%) and subsequent fracture (at 4%) of the #4 ($d_b = 12.7$ mm) vertical reinforcement at the eastern boundary element (**Figures 7b** and **7a**, respectively). Buckling of vertical reinforcement at the western boundary element initiated at 4% lateral drift for positive loading.

SRCW2: At approximately 1.0% lateral drift, strain gauge readings on the #4 ($d_b = 12.7$ mm) vertical reinforcing bars exceeded the yield strain. Yielding of the structural steel section in tension was observed after yielding of longitudinal reinforcement at the initiation of buckling of longitudinal bars at the eastern boundary element. Buckling initiated in the #4 ($d_b = 12.7$ mm)

vertical reinforcing bars around the eastern boundary element ($s = 102$ mm, boundary hoop spacing) 1.07 m (**Fig. 8b**) above the pedestal during the second cycle to 2.0% lateral drift. The strains on the western boundary steel section were higher on average than in the eastern steel section, likely due to increased slip of the steel section at the eastern end due to the lower quantity of boundary transverse reinforcement (and the crosstie through the web of the structural steel section). As well, the slip caused large cracks to form at the wall-foundation interface, which likely led strain concentrations in reinforcement in compression, and eventually buckling of reinforcement at this location. At the 3.0% and 4.0% drift levels, the lateral force dropped noticeably for the pull cycles (eastern boundary in compression). During the first 3.0% drift cycle, loud noises indicated fracture of reinforcement followed by noticeable slip (**Figure 8**) between the embedded structural steel section and the surrounding concrete (likely due to fracture of the crossties passing through the W6x16 web). At 4.0% lateral drift, several of the #4 ($d_b = 12.7$ mm) vertical reinforcement in the eastern boundary element fractured in tension, as was observed for SRCW1. Several #4 ($d_b = 12.7$ mm) reinforcement bars started buckling on west boundary element during the second cycle of 3.0% drift at around 1.17 m above the pedestal, where the spacing of transverse reinforcement was increased to 152 mm.

SRCW3: At 2.5% drift, buckling of steel reinforcing bars started occurring visibly in the east boundary element at about 1.1 m above the pedestal; although no strength degradation was observed on the wall. After that, no complete cycles were achieved during the 3.0% drift level because of an unanticipated out-of-plane failure, attributed to a construction defect. This failure is probably affected by the higher level of axial load of this specimen. **Figure 9** shows the bent reinforcement and wall profile.

b. Wall Nominal Moment and Shear Capacities

The wall nominal moment capacities are computed using stress – strain relations developed for 34.5 MPa concrete (**Fig. 10a**) and for Grade 60 (413 MPa) reinforcement and for Grade 50 (345 MPa) structural steel (**Fig. 10b**). Confined concrete is incorporated using the model proposed by Saatcioglu and Razvi [20]. The lateral loads required to reach the nominal moment capacity for each specimen are given in **Table 1**. The nominal shear capacities (V_n) are computed using Eq. (18.10.4.1) of ACI 318-14 [2] and also are provided in **Table 1**. The results indicate that the shear capacity is about twice the force required to reach the nominal flexural strength; therefore, shear failure is not anticipated.

c. Load – Displacement Response

A bilinear approximation to the lateral load – top displacement relation was computed by the moment and displacement associated with first yield of the boundary vertical reinforcement (initial slope) and the nominal moment (moment strength). The material relations plotted in **Fig. 10** were used. A section analysis was used to find the moment at first yield of reinforcement and the yield displacement was computed by integrating the curvature over the wall height. The nominal moment capacity was computed for an extreme fiber concrete compressive strain of 0.003.

The bilinear predictions are plotted on **Fig. 11** to **Fig. 13** along with the experimental measured lateral load versus top displacement (corrected by pedestal movement) relations for all three specimens. The elastic stiffness is based on first yield of the boundary reinforcement at the extreme wall face; a lateral displacement of approximately 1.5 times first yield displacement is

1 required to achieve yield of the majority of the boundary longitudinal reinforcement. The
2 positive lateral top displacement is associated with compression in the western boundary element
3 (more confined boundary element). The plots reveal that lateral load capacities of the tested
4 walls are slightly smaller than nominal values for SRCW1 and SRCW3. However, the lateral
5 load capacity resulted much smaller than its nominal value for SRCW2, as it can be seen in
6 **Table 1**, where expected nominal load relative to maximum experimentally applied lateral loads
7 are listed, resulting as small as 0.79 for this case. Specimens SRCW1 and SRCW3 reached
8 values close to 1.0 resulting in better agreement between the nominal and experimental forces.
9 From this global behavior the influence of confinement can be observed. In all three specimens
10 no drop in lateral load was observed when loading in the positive displacement direction
11 (western boundary element in compression) which indicates good performance up to the applied
12 lateral displacement. However, loading in the negative displacement direction (eastern boundary
13 element in compression) presented load decrements at large displacements (larger than the
14 design lateral displacement, 1.5% drift). These load decrements are associated with a reduced
15 confinement in the eastern boundary element. According to the sectional analysis, the nominal
16 moment is reached for tensile strain (in the extreme tension bar) values close to 1% (for SRCW1
17 and SRCW2), which is consistent with a lateral drift at the top of the wall of about 0.6% (see Eq.
18 1). In the case of SRCW3, a tensile strain of about 0.6% is reached by the extreme rebar in
19 tension at the nominal moment for a lateral drift of about 0.45%. When the nominal moment is
20 reached, significant plastic strain is anticipated, especially for specimens with lower axial load
21 (SRCW1 and SRCW2). Test results reveal that nominal moment is reached at a larger drift level
22 (close to 1%), which is, in part, caused by strain penetration (strain distribution of reinforcing bar

1 and steel sections within the pedestal that result in rigid body rotation of the specimens at the
2 wall-pedestal interface). More discussion on this topic is provided in the following section.

3 The wall aspect ratio and the wall shear strength were selected to ensure that elastic and
4 inelastic deformations would be dominated by flexure. As well, the wall vertical reinforcement
5 was continuous over the wall height and favorable conditions were provided for the anchorage of
6 vertical reinforcement (web and boundary) extending into the base pedestal. An examination of
7 the test results revealed that top displacement due to strain penetration was relevant, and in the
8 case of SRCW2, slip at the interface of the concrete and structural steel section, which is
9 explained in later in this paper.

10 Although slip between the concrete and the structural steel section was observed,
11 specimens SRCW1 and SRCW3 were able to reach their nominal capacity. Specimen SRCW2
12 possessed smaller bond strength (222 kN) and larger bond demand (1052 kN) relative to other
13 specimens. Anchorage failure was not observed (within the foundation block), which is
14 consistent with calculations presented earlier; therefore, yielding of the embedded structural steel
15 section, as well as the traditional longitudinal reinforcement, is expected for specimen SRCW2.
16 The lower capacity observed for specimen SRCW2 is due to delayed yielding of the structural
17 steel section caused by slip. Observations indicate that buckling of longitudinal reinforcement
18 initiates prior to yielding of the structural steel section (in tension), reducing the overall capacity.
19 The boundary longitudinal reinforcement accounts for about 25% of the section moment
20 capacity, which is consistent with the reduced capacity observed in the wall at large
21 deformations.

22 The initial stiffness values for all three specimens for both positive and negative loading,
23 is not significantly different, indicating that boundary element cross-ties did not significantly

1 impact the bond strength between the concrete and the embedded steel section for loads less than
2 one-half of the yield load); however, the crossties did reduce slip between the concrete and the
3 embedded steel section at larger deformations.

4 5 **d. Shear and flexural deformations: Evaluation of test data**

6 Instrumentation was provided on the test specimens to allow the determination of flexural
7 and shear deformations over the first three stories of the specimens (**Fig. 5**). Shear deformations
8 were determined using measurements from wire potentiometers placed diagonally, as well as
9 vertically along the wall boundary at three levels, based on the approach reported by Massone
10 and Wallace [21], which includes a higher order correction term to account for the location of the
11 centroid of the flexural deformations over the gage length of the diagonal sensors.

12 The contribution of the flexural deformations (curvature) over the first story is
13 determined as a rigid rotation at an assumed centroid, where the rotation is determined by
14 integrating the curvature over the first story. Similarly, this procedure may be used to estimate
15 the flexural deformations at upper stories.

16 **Figure 14** plots the story shear force (lateral load) versus the flexural and shear
17 deformations within the first, second and third stories for specimen SRCW1. To ease the
18 comparison only the enveloped of the cyclic positive response is plotted in **Fig. 14**. The flexural
19 component of deformation is normalized to the relative deformation at each level that causes first
20 yielding of the vertical boundary reinforcement in order to have comparable curves at each level.

21 Plots of story shear force (lateral load) versus flexural deformation (**Fig. 14a**) reveal that
22 the nonlinear deformation curvature and strain penetration were important for the first story. This
23 is consistent with experimental observations, where damage location and reinforcement strain

gauges indicate that yielding was primarily limited to the first story. Strain penetration appears to have contributed significantly to a loss of stiffness in the first story level relative to the stiffness obtained for a section analysis based on plane sections and no slip. An estimation of displacement due to strain penetration at yield for the first story was calculated assuming linear strain distribution in the structural steel section inside the pedestal from zero at the end plate (584 mm below the pedestal-wall interface) to a maximum strain (yield strain) of 0.00172 at the pedestal-wall interface. Integration of the strain distribution (inside the pedestal) results in a rotation around the neutral axis that produces a rigid body rotation at the top of the first story level (1219 mm above the wall-foundation interface). The strain penetration contribution was then subtracted from the measured lateral flexural displacement and the results plotted (**Fig. 14a**) reveal that stiffness values are similar for all story levels.

Regarding the shear deformations for specimen SRCW1 (**Fig. 14b**), the plot reveals findings similar to other authors (e.g., [21]) where: (1) inelastic shear behavior occurred in the first story despite a nominal shear capacity of approximately twice the applied story shear, and (2) inelastic deformations were limited to essentially the first story. Experimental results indicate that shear deformations account for only about 5% of the total lateral displacement at the top of the wall.

e. Effect of confinement at boundary elements

Vertical splitting at the interface of the boundary element and the wall web was observed in all specimens. Splitting was more pronounced for specimen SRCW1 because less boundary transverse reinforcement was provided. Improving the shear transfer mechanism at this interface would also delay splitting and delay buckling of boundary longitudinal reinforcement. For

example, specimens SRCW1 and SRCW2 used the same configuration of transverse reinforcement; however, spacing of transverse reinforcement was increased from 51 mm to 102 mm (at both boundary elements), which are equivalent to $4d_b$ and $8d_b$, where d_b is the longitudinal bar diameter. Earlier and more substantial buckling of boundary longitudinal reinforcement was observed at wall boundaries with less transverse reinforcement, as expected. In particular, specimen SRCW2 presented earlier degradation compared to SRCW1 given the larger lateral strength provided by the larger steel section used at the wall boundary. In specimen SRCW1 degradation started at about 2% drift in the less confined boundary, whereas SRCW3, with larger axial load, displayed degradation at a similar drift level due to out-of-plane instability, but the boundary element was provided with a spacing of $6d_b$. This indicates that the spacing of transverse reinforcement was relevant regarding the displacement capacity of the specimens. Experimental and analytical studies reported by Massone and Moroder [22] also show that peak tensile stresses developed in longitudinal reinforcement reduce by about 25% at a tensile strain of 0.05 as the spacing of transverse reinforcement increased from $4d_b$ to $8d_b$, helping to explain the lower strength achieved.

4. SUMMARY AND CONCLUSIONS

Test results for structural steel reinforced concrete walls (SRC) were examined to establish force deformation relations and failure modes. Special attention was offered to the behavior of both boundaries due to different level of confinement and any premature slipping in steel section. Based on these studies, the following conclusions were reached:

- (1) SRC walls presented good behavior, i.e., were able to reach the design drift level (1.5%) without strength loss and with modest strength loss (about 45% for SRCW1 and 20% for

SRCW2) beyond 2% lateral drift when the less confined boundary element ($s = 102 \text{ mm} = 8d_b$) was in compression. SRCW3 displayed almost no strength loss until about 2.5% lateral drift once out-of-plane failure is observed.

(2) Load versus displacement curves for flexure showed how the slipping of steel section influences the general behavior of the wall, reducing its stiffness by a factor of two (approximately) essentially for the first story and also reflected at the top of the wall. Although, cross-ties going through the steel section inside the pedestal might have increase bond strength, the number of them seems no sufficient to avoid slip.

(3) Specimens SRCW1 and SRCW3 were able to reach their nominal capacity, where specimen SRCW3 reached 97% of the nominal strength in average. Specimen SRCW2 presented much smaller bond strength, and also larger bond demand. The bond strength provided by embedment of the structural steel section represented 71% of the required capacity to reach yielding for specimens SRCW1 and SRCW3, whereas for specimen SRCW2 it was only 21%. Considering this, it is recommended to provide at least 70% of the bond strength by structural steel embedment in order to promote yielding.

(4) Force versus deformation relations for flexure and shear were derived from SRCW1 for the first story level, where inelastic deformations were concentrated. Inelastic shear and flexural deformations initiated at essentially the same lateral displacement level, despite nominal shear strengths of approximately twice the lateral force required to initiate flexural yielding.

(5) Tight spacing seems to delay vertical splitting (e.g., SRCW1), favored by later initiation of rebar buckling.

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REFERENCES

- [1] Wallace JW, Orakcal K. ACI 318-99 Provisions for Seismic Design of Structural Walls. ACI Structural Journal 2002; 99(4):499-508.
- [2] ACI 318-14. Building code requirements for structural concrete and commentary. 2014 building code and commentary. American Concrete Institute (ACI).
- [3] Roeder ChW, Chmielowski R, Brown CB. Shear Connector Requirements For Embedded Steel Sections. Journal of Structural Engineering 1999; 125(2):142-151.
- [4] Wallace JW, Moehle JP, Martinez-Cruzado J. Implications for the Design of Shear Wall Buildings Using Data from Recent Earthquakes. Proceedings, Fourth U. S. National Conference on Earthquake Engineering 1990, Palm Springs, California, Vol. 2, 359-368.
- [5] Esaki F, Ono M. Effect of loading rate on mechanical behavior of SRC shearwalls. Steel and Composite Structures 2001; 1(2):201-212.
- [6] Liao F-Y, Han L-H, Tao Z. Performance of reinforced concrete shear walls with steel reinforced concrete boundary columns. Engineering Structures 2012; 44:186-209.

- [7] Cho SH, Tupper B, Cook WD, Mitchell D. Structural steel boundary elements for ductile concrete walls. *Journal of Structural Engineering* 2004; 130(5):762-768.
- [8] Zhou Y, Lu X, Dong Y. Seismic behaviour of composite shear walls with multi-embedded steel sections. Part I: experiment. *Struct. Design Tall Spec. Build.* 2010; 19:618-636.
- [9] Dan D, Fabian A, Stoian V. Theoretical and experimental study on composite steel-concrete shear walls with vertical steel encased profiles. *Journal of Constructional Steel Research* 2011; 67:800-813.
- [10] Qian J, Jiang Z, Ji X. Behavior of steel tube-reinforced concrete composite walls subjected to high axial force and cyclic loading. *Engineering Structures* 2012; 36:173-184.
- [11] Ji X, Sun Y, Qian J, Lu X. Seismic behavior and modeling of steel reinforced concrete (SRC) walls. *Earthquake Engng Struct. Dyn.* 2015; 44:955-972.
- [12] Lu X, Yang J. Seismic behavior of T-shaped steel reinforced concrete shear walls in tall buildings under cyclic loading. *Struct. Design Tall Spec. Build.* 2015; 24:141-157.
- [13] Cherlin ME. Design of Steel Reinforced Concrete Shear Walls Subjected to Seismic Loading. MSCE Thesis, University of California, Los Angeles, Dept. of Civil Engineering, April 2003, 204 pp.
- [14] ACI 318-99. Building code requirements for structural concrete and commentary. 1999 building code and commentary. American Concrete Institute (ACI).
- [15] Massone LM. Fundamental Principles Of The Reinforced Concrete Design Code Changes In Chile Following The Mw 8.8 Earthquake In 2010. *Engineering Structures* 2013; 56:1335-1345.

- [16] Wallace JW. Behavior, Design, and Modeling of Structural Walls and Coupling Beams - Lessons from Recent Laboratory Tests and Earthquakes. International Journal Of Concrete Structures And Materials 2012; 6(1):3-18.
- [17] Thomsen IV JH, Wallace JW. Displacement-Based Design of Slender RC Structural Walls – Experimental Verification. J. Struct. Eng. 2004; 130(4):618-630.
- [18] Wallace JW, McConnell SW, Gupta P, Cote PA. Use of headed reinforcement in beam-column joints subjected to earthquake loads. ACI Structural Journal 1998; 95(5):590-606.
- [19] Sayre BL. Performance Evaluation of Steel Reinforced Shear Walls, MSCE Thesis, University of California, Los Angeles, Dept. of Civil Engineering, 2003, 265 pp.
- [20] Saatcioglu M, Razvi SR. Strength and ductility of confined concrete. Journal of Structural Engineering 1992; 118(6):1590-1607.
- [21] Massone LM, Wallace JW. Load – Deformation Responses of Slender Reinforced Concrete Walls. ACI Structural Journal 2004; 101(1):103-113.
- [22] Massone LM, Moroder D. Buckling Modeling Of Reinforcing Bars With Imperfections. Engineering Structures 2009; 31(3):758-767.

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- 18

Table 1 – Test load capacity: analytical and experimental results

Specimen	Reinforcement		F_N^3	V_N	$F_{MAX} \equiv V_{MAX}^4$	V_{MAX}/V_N	F_{MAX}/F_N
ID	Boundary ¹	Web ²	(kN)	(kN)	(kN)	(6)/(5)	(6)/(4)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
SRCW1	8 - 12.7 $A_s = 1729$	9.53 @ 152.4	304	651	307 -290	0.47 0.45	1.01 0.95
SRCW2	8 - 12.7 $A_s = 3058$	9.53 @ 152.4	392	651	311 -329	0.48 0.51	0.79 0.84
SRCW3	8 - 12.7 $A_s = 1729$	9.53 @ 152.4	344	651	337 -331	0.52 0.51	0.98 0.96

1 # of vertical bars - bar diameter in mm; A_s = W-steel-section area [mm²]

2 bar diameter in mm @ bar spacing in mm

3 Lateral force required to reach the M_n at the wall base

4 Maximum applied lateral force (positive/negative)

Fig. 1

Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6

(a)

(b)

Fig. 7

(a)

(b)

Fig. 8

(a)

(b)

Fig. 9

(a)

(b)

Fig. 10

Fig. 11

Fig. 12

Fig. 13

Fig. 14