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INFLUENCE OF MECHANICAL PROPERTIES OF HIGH-STRENGTH STEEL ON DEFORMATION CAPACITY OF REINFORCED CONCRETE WALLS

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Abstract

Results are reported from tests of two large-scale reinforced concrete "T-shaped" slender walls under reversed cyclic loading. The specimens are part of a series of four specimens designed to study the effect of reinforcing bar mechanical properties on wall deformation capacity (defined by a drift cycle completed before a 25% loss of strength). Primary variables included reinforcement yield strength and the ratio of tensile-to-yield strength of the reinforcement. An additional aim of the tests is to determine the minimum uniform elongation required of high-strength reinforcing bars for use in earthquake-resistant structures.

The walls, 300-in. (762-cm) tall and 10-in. (25.4-cm) thick had a 100-in. (254-cm) long stem joining a 100-in. (254-cm) long flange at one end (resulting in a shear span-to-depth ratio of 3). The control specimen had Grade 60 (420) reinforcement with a nominal tensile-to-yield strength ratio of 1.35. The second specimen had Grade 100 (690) reinforcement with a nominal tensile-to-yield strength ratio of 1.25. Future test specimens will have Grade 100 (690) reinforcement with a nominal tensile-to-yield strength ratio of 1.15 and 1.4. The walls were designed so that flexural yielding would limit lateral strength, resulting in an expected web shear stress of approximately $3.5\sqrt{f'_c}$, psi ($0.3\sqrt{f'_c}$, MPa). Design of the walls complied with applicable ACI 318 Building Code [1] requirements for special structural walls and additional detailing requirements identified in ATC 115 (2014) [2]. A key feature of the design was to limit the neutral axis depth to ensure strain demands approached fracture of the longitudinal reinforcement. The tested walls followed the same loading protocol and experienced bar fracture during the first cycle to a drift ratio of 4%. The similar hysteresis behavior observed from testing of the two walls supports the use of high-strength steel as concrete reinforcement for earthquake-resistant construction.

Keywords: Cyclic loading, earthquake-resistant construction, high-strength steel, seismic performance, structural walls.

1. Introduction

Use of high-strength steel bars with a nominal yield strength greater than 80 ksi (550 MPa) as longitudinal reinforcement in concrete is not currently permitted by U.S. building codes. This is mostly due to insufficient test data and, until recently, the limited availability of affordable high-strength steel. Use of high-strength steel bars could, however, allow designers to reduce the amount of reinforcement, resulting in less reinforcement congestion, simpler construction, and lower cost.

It is now possible to produce high-strength steel reinforcement at a cost that is competitive with Grade 60 (420 MPa) reinforcement. The various methods of doing so, however, result in post-yield behaviors that range from a bar having a clearly defined yield point and limited strain-hardening (tensile-to-yield strength ratio, T/Y, greater than 1.0) to a bar that exhibits no yield point and pronounced strain hardening. The 0.2%-offset method is commonly used to define the yield strength. Values of T/Y typically range between 1.1 and 1.5.

The main objective of this research is to study the effects of mechanical properties of reinforcement on the behavior and deformation capacity of T-shaped concrete walls. In particular, this investigation aims to define the minimum elongation required for high-strength reinforcing bars to be used in earthquake-resistant slender walls and to examine whether T/Y affects that requirement by altering the spread of plasticity in the plastic hinge region. It is important to study these variables through tests of T-shaped walls because the shape of the cross-section leads to larger longitudinal reinforcement tensile strain demands than in most other members. Reported results will also be evaluated to determine whether the performance of T-shaped concrete walls under lateral loads is affected by using reduced amounts of high-strength steel Grade 100 (690) in place of conventional Grade 60 (420) reinforcement.

The test program, summarized in Table 1, is comprised of four large-scale T-shaped slender wall specimens. This paper summarizes preliminary findings from tests of two of the walls (T1 and T3). The T-shaped walls have a height of 300 in. (762 cm), thickness of 10 in. (25.4 cm) and length of 100 in. (254 cm) along both flange and stem (Fig. 1 and 2). The control specimen, wall T1, was reinforced with conventional Grade 60 (420) reinforcement. Wall T3 had Grade 100 (690) longitudinal and transverse reinforcement. The primary longitudinal reinforcement (vertical reinforcement in the confined area) for wall T1 and T3 had a nominal tensile-to-yield strength ratio of 1.35 and 1.25 respectively. For both walls to attain nearly identical flexural strength, the wall constructed with Grade 100 reinforcement had a lower reinforcement ratio, ρ, so the product ρf_y was nearly the same. Both specimens were subjected to the same loading protocol patterned after FEMA 461 [3].

2. Research Significance

This study includes the first large-scale tests of slender walls reinforced with Grade 100 steel reinforcing bars. The available literature on the use of high-strength reinforcement has focused predominantly on beams and columns. Prior studies have not investigated the cyclic response of concrete walls with unsymmetrical cross section reinforced with Grade 100 (690) steel bars for use in high-seismic regions.

The test program is aimed at investigating the effects of uniform elongation (strain at peak stress) and the ratio of tensile-to-yield strength of reinforcing bars on the deformation capacity of reinforced concrete walls. These were identified as high-priority items in the "Roadmap for the Use of High-Strength Reinforcement in Reinforced Concrete Design" (ATC 115, 2014) [2], which outlines the research effort needed to incorporate the use of high-strength reinforcement into building design and construction practice.

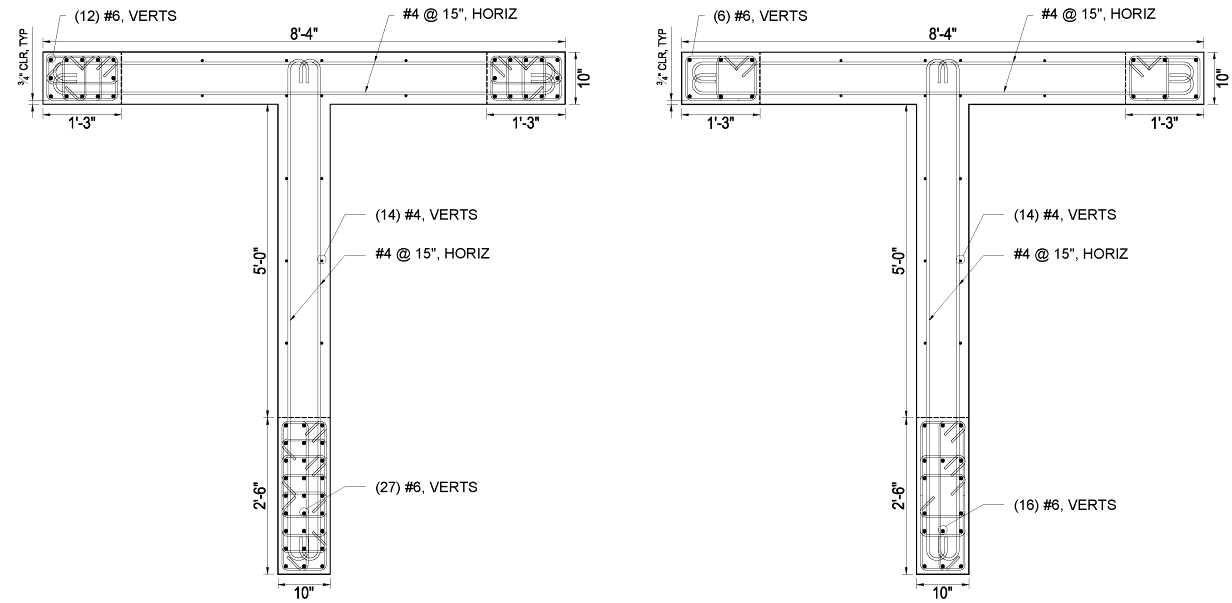
3. Experimental Investigation

The test program includes the four large-scale T-shaped walls summarized in Table 1. Results from tests of walls T1 and T3 are reported herein. The cross sections of the two specimens are shown in Fig. 1. A typical wall elevation is shown in Fig. 2. The walls were loaded at 300 in. (762 cm) above the base block for a shear span-to-depth ratio of 3.0. Wall T1 had Grade 60 (420) reinforcement with a nominal tensile-to-yield strength ratio of 1.35. Wall T3 had Grade 100 (690) reinforcement with the same nominal tensile-to-yield strength ratio of 1.25. The design and proportioning of these specimens is similar to previously tested walls with Grade 60 (420) reinforcement [4, 5].

Table 1 – Summary of test program

Wall	Yield Strength (f_y) [1] ksi (MPa)	Tensile-to-Yield Strength Ratio (f_t/f_y) [1]	Concrete Compressive Strength [1] ksi (MPa)
T1 [2]	60 (420)	1.3	8 (55)
T2 [3]	100 (690)	1.2	8 (55)
T3 [2]	100 (690)	1.3	8 (55)
T4 [3]	100 (690)	1.4	8 (55)

[1] Target values
[2] Tested in fall of 2015; preliminary results are presented herein
[3] Tested in summer of 2016



(a) Wall T1 (b) Wall T3

Fig. 1 – Reinforcement layout (Note: 1 in. = 25.4 mm.)

The walls were designed to have their lateral strength limited by flexural yielding. The shear stress associated with the probable flexural strength (calculated using 1.25 times the specified yield strength of reinforcement) was limited to a maximum of $4\sqrt{f'_c}$, psi ($0.33\sqrt{f'_c}$, MPa). For this level of shear stress, the required shear reinforcement is approximately equal to the minimum prescribed in Chapter 18 of the ACI Building Code (318-14) [1] for special structural walls. The walls complied with detailing requirements in Chapter 18 of ACI 318-14 and the additional special requirements identified in ATC 115 (2014) [2] for walls reinforced with Grade 100 (690) steel bars; particularly, the spacing of the confining reinforcement in the boundary elements did not exceed 4 times the diameter of the longitudinal reinforcement. A key design feature was to minimize the neutral axis depth to ensure large tensile strain demands (measured from the extreme compression fiber at nominal flexural strength when the flange is in compression, the calculated neutral axis depth was less than 2% of the wall length).

3.1 Materials

Ready-mixed concrete was provided by a local supplier to cast both specimens. The target concrete compressive strength was 8,000 psi (55 MPa). The concrete compressive strength, measured following ASTM C39/C39M-15a (2015) [6], was 7.2 ksi for wall T1 and 7.3 ksi for wall T3 (for the first lift of the wall).

Mill certifications of reinforcing bars used in wall T1 showed compliance with ASTM A706/A706M-15 (2015) [7] Grade 60 (420) requirements. Mill certifications of reinforcing bars used in wall T3 showed compliance with ASTM A615/A615M-15a (2015) [8] Grade 100 (690). Table 2 lists the measured properties. The Grade 60 (420) bars in the boundary elements of wall T1 had actual yield strength of 70 ksi (483 MPa), whereas the Grade 100 (690) bars in the boundary elements of wall T3 had actual yield strength of 99 ksi (683 MPa).

Table 2 – Properties of reinforcing steel

Wall	Bar Size No.	Nominal Bar Diameter in. (mm)	Yield Strength f_y ksi (MPa)	Tensile Strength f_t ksi (MPa)	f_t/f_y	Uniform Elongation ε_{su}	Fracture Elongation [a] ε_{sf}
	6 (19)	0.75 (19)	70 (483) [b]	94 (648) [b]	1.34	12.1% [b]	15.0% [c]
T1	4 (13)	0.50 (13)	76 (524) [b]	106 (731) [b]	1.39	10.7% [b]	14.0% [c]
	3 (10)	0.375 (10)	60 (414) [c]	91 (627) [c]	1.52	–	16.5% [c]
	6 (19)	0.75 (19)	99 (683) [b]	122 (841) [b]	1.23	9.2% [b]	12.5% [c]
T3	4 (13)	0.50 (13)	101 (696) [b]	122 (841) [b]	1.21	5.9% [b]	12.5% [c]
	3 (10)	0.375 (10)	109 (752) [c]	134 (924) [c]	1.23	–	11.3% [c]

[a] Based on 8-in. (203 mm) gauge length
[b] Measured from laboratory tests of coupons
[c] Reported on manufacturer mill certification

3.2 Specimen Construction

As indicated in Fig. 2, each wall specimen was constructed in four phases (base block, two wall lifts, and a top block) with three construction joints. Each phase consisted of assembling reinforcing bar cages and wooden formwork, and then casting the concrete. Formwork was typically removed three to four days after casting. The wall vertical reinforcement was lap spliced near mid-height.

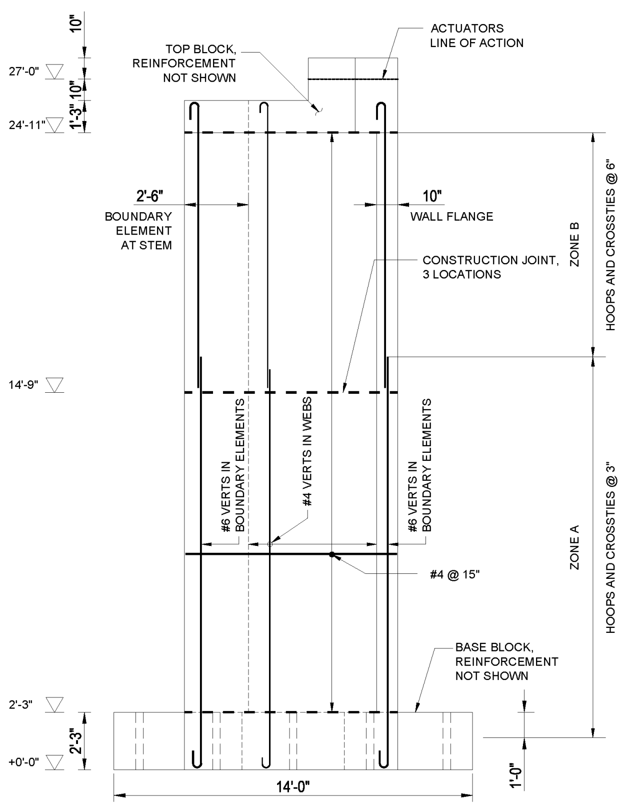


Fig. 2 – Typical wall elevation
(Note: 1 in. = 25.4 mm.)

Fig. 3 – Test setup

3.3 Test Setup

For testing, the specimens were bolted to the laboratory strong floor with fourteen 1.7-in. (43-mm) diameter high-strength threaded bars passing through the base block. The top of each wall was connected to two MTS 201.70 Hydraulic Actuators installed in parallel. The actuators each have a stroke length of 40 in. (102 cm) and a capacity of 220 kip (979 kN). Placing the actuators at the same elevation and on opposite sides of the wall stem prevented twisting of the top of the wall during testing (Fig. 3).

Additional steel fixtures were provided to brace the wall near mid-height as shown in Fig. 3. Two separate bracing systems were assembled: 1) internal bracing near mid-height to prevent instability of the stem or flange tips, and 2) external bracing just above mid-height to prevent global twisting of the wall. The diagonal braces in the internal bracing system were pinned at their ends to allow relative vertical displacements between brace points. The connection between the external bracing and the wall specimen consisted of a nylon pad at the end of each diagonal brace and mirror-finished steel plates attached to the wall stem.

3.3.1 Instrumentation

Lateral deflection of the top of the specimens, relative to the strong wall, was measured with three string potentiometers installed at a height of 317-in. (805-cm) above the strong floor. Two of the potentiometers, with a 50-in. (127-cm) stroke, were spaced 80 in. (203 cm) apart to record twisting of the wall in addition to lateral displacement. The third potentiometer, with a 20-in. (51-cm) stroke, was centered on the stem as a redundant measure of the wall deflection. In addition, two potentiometers, with a 4-in. (10-cm) stroke, were mounted 19 in. (48 cm) above the strong floor to measure twisting and sliding of the base block relative to the laboratory floor. To allow for calculation of wall elongation and flexural rotation, string potentiometers were mounted vertically along the tip of the stem and along the center of the flange. These vertical potentiometers measured elongation of the wall boundaries within 100 in. (254 cm) of the top of the base block and between 100 and 290 in. (254 and 737 cm) from the top of the base block.

In addition to potentiometers, an infrared-based non-contact position measurement system was used to record the position of 86 markers attached directly to the surface of the specimens (the markers emit infrared light pulses that are detected by cameras, allowing for their spatial coordinates to be triangulated). The markers were arranged in an approximately 14-in. (36-cm) square grid over one face of the stem and half of one face of the flange throughout the lower 100 in. (254 cm) of the wall. Data from this system will be analyzed to better understand the distribution of deformations near the base of the wall. Other measurements are based on twenty-eight electrical resistance strain gauges placed on longitudinal and transverse reinforcing bars.

3.3.2 Loading Protocol

The sequence of displacements imposed on each wall specimen followed the pattern shown in Fig. 4, which is similar to the protocol recommended in FEMA 461 [3]. The drift ratio is defined as the top displacement (measured from the top of the base block to the horizontal plane passing through the center axis of the actuators, corrected for translation and rotation of the base block) divided by the height of the wall. Two cycles were imposed at each target drift up to a drift ratio of 3%. As shown in Fig. 4, the loading protocol ended with a single cycle to a drift ratio of 4%. A final monotonic push was added after reaching a drift ratio of –4%.

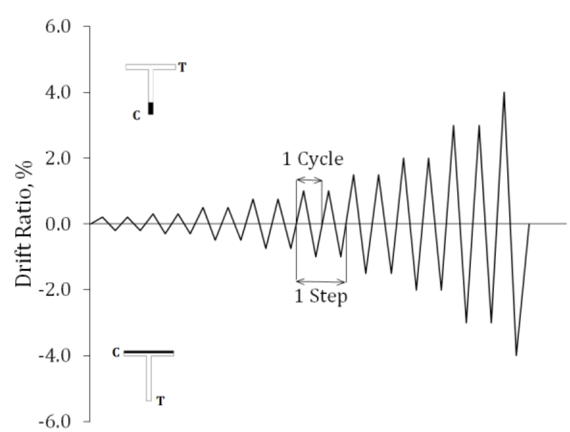


Fig. 4 – Loading protocol

4. Test Results

4.1 Measured Shear versus Drift Ratio

The measured shear versus drift response for walls T1 and T3 are shown in Fig. 5 and 6. The specimens exhibited similar behavior with a stable hysteretic response. Both walls sustained two cycles at 3% drift ratio without a significant reduction in load carrying capacity. In the first cycle to 4% drift ratio, both walls exhibited fracture of vertical bars in the confined stem. Unlike T3, T1 did not exhibit a significant strength reduction after fracture of the stem reinforcing bars near a drift ratio of –3%.

The maximum shear force resisted by the walls was 303 and 275 kip (1348 and 1223 kN) for walls T1 and T3, which corresponds to shear stresses of $3.6\sqrt{f'_c}$ psi ($0.30\sqrt{f'_c}$ MPa) and $3.2\sqrt{f'_c}$ ($0.27\sqrt{f'_c}$ MPa), respectively. Both walls exhibited a maximum shearing force at 3% drift ratio. The difference in lateral strength was attributed to the difference between the actual yield strength and the nominal yield strength of the longitudinal reinforcement.

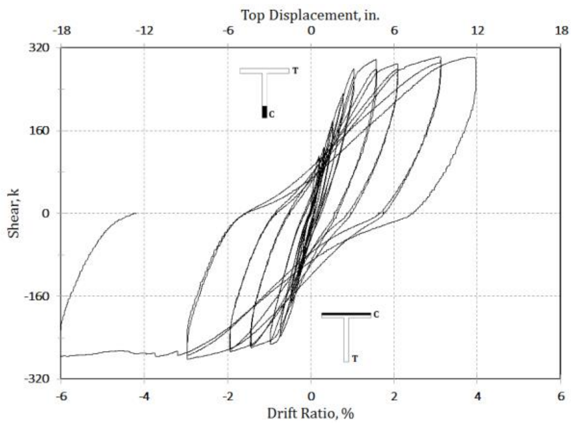


Fig. 5 – Shear versus drift ratio for T1

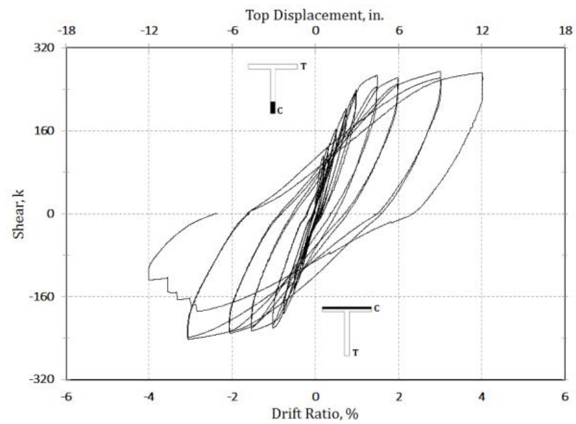
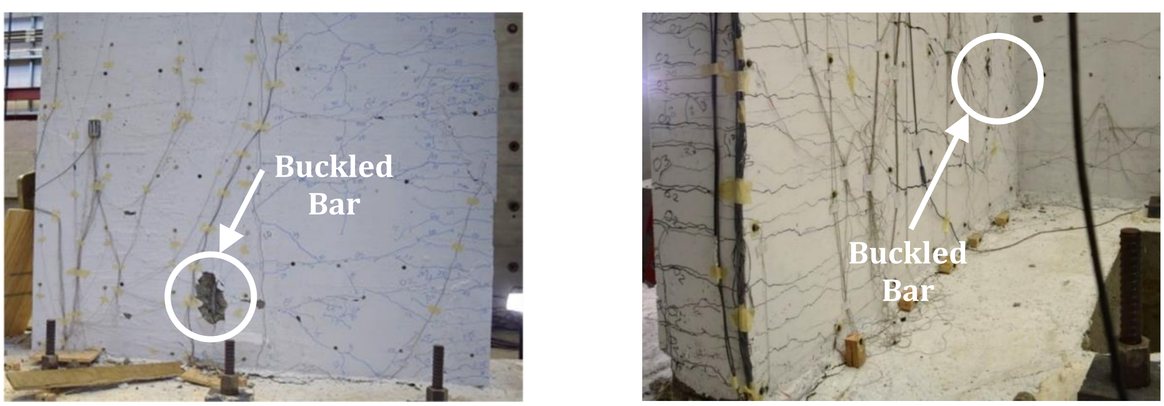


Fig. 6 – Shear versus drift ratio for T3

4.2 Progression of Damage

Horizontal cracking associated with flexure was observed along the confined stem and flanges at a spacing that was approximately equal to the 3 in. spacing (76 mm) of the confining hoops within the boundary elements. Flexural cracks in the flange between the flange boundary elements and the intersection with the stem were inclined such that the cracks met the stem-to-flange intersection at a lower elevation than they met the boundary elements at the tips of the flange. Inclined shear cracks within the stem were observed along the entire height of the walls at a spacing of approximately 15 in. (38 cm).



(a) Wall T1, flange in compression (b) Wall T3, flange in compression

Fig. 7 – Bar buckling at unconfined zone during 2% drift cycle

In T1, buckling of the longitudinal (vertical) bars was first observed at the unconfined flange-stem intersection during the first excursion to a drift ratio of –2% (stem in tension), as shown in Fig. 7(a). Buckling of the longitudinal reinforcement in the unconfined stem was first observed during the first excursion to a drift ratio of –4%. In T3, bar buckling was first observed in the unconfined stem during the second excursion to a drift ratio of –2%, as shown in Fig. 7(b). Longitudinal reinforcement in the unconfined flange-stem intersection was first observed to buckle during the first excursion to a drift ratio of –4%. It is important to emphasize that vertical reinforcement in the unconfined stem and flange was located outside the horizontal bars (i.e., closer to the face of the wall), as permitted in ACI 318-14. Buckling of the bars at unconfined zones did not severely affect the measured force-displacement curves (Fig. 5 and 6).

(a) Wall T1

(b) Wall T3

Fig. 8 – Bar fracture at confined stem during 4% drift cycle

For both specimens, concrete cover began to exhibit compression-related damage (flaking and minor spalling) at the tip of the stem during the cycle to +1% drift ratio (stem in compression). Buckling of the bars near the tip of the confined stem was first observed during the second excursion to +3% drift ratio for T1 and during the first excursion to +4% drift ratio for T3. Bar fracture first occurred in both specimens during the first excursion to –4% drift ratio (Fig. 8).

5. Concluding Remarks

Main findings from the test results presented for T1 with conventional Grade 60 (420) reinforcement and for T3 with Grade 100 (690) reinforcement are as follows:

1) Walls T1 and T3 exhibited similar deformation capacity and overall hysteresis. Both specimens completed two cycles to 3% drift ratio without a major reduction in load carrying capacity.
2) Buckling of longitudinal (vertical) reinforcement in both tests was first observed in the unconfined stem and flanges near the base of the wall during cycles to 2% drift ratio.
3) Spalling of the concrete cover was observed at the confined stem during the first excursion to a drift ratio of 1%. Buckling of longitudinal bars in the confined stem was first observed during the second cycle to 3% drift ratio in wall T1 and during the first cycle to 4% drift ratio in wall T3.
4) Although the reinforcement in wall T1 had 20% greater fracture elongation than the reinforcement in wall T3 (15% versus 12.5%), both walls exhibited fracture of reinforcement in the stem boundary element during the first cycle to a drift ratio of 4%.

6. Acknowledgements:

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7. References

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